EARTH AND ROCKFILL DAMS
BARRAGES EN TERRE ET EN ENROCHEMENT

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INTRODUCTION

This paper is a report on the state-of-the-art of the design and construction of earth and rockfill dams. As such, it is intended to reflect current design and construction practise throughout the world, but the writers are well aware that neither they nor anyone else could possibly accomplish this solely on the basis of personal experience and knowledge. A detailed review of the published literature plus extensive conversations and correspondence with colleagues provides the basis for most of this report, but inevitably there are errors of omission, particularly with regard to countries other than those in North America.

For most large dams a considerable period of time elapses between preliminary design and first filling of the reservoir; hence professional papers giving details of design, construction and performance reflect design practices which antedate by at least 3 to 5 years the publication date. For this same reason, however, changes in design practices develop slowly since engineers are reluctant to adopt unproven changes. Perhaps in no other type of major construction does the engineer rely so heavily upon experience and precedent.

A dam, whether it be for power production, flood control, pumped storage, irrigation, recreation, lowstream augmentation, or multipurpose, contains many elements. This report is concerned primarily with the embankment itself and not with such elements as powerhouses, spillways, diversion features and outlet works except insofar as they interact with and thereby affect the embankment performance. Abutments and foundation treatment are considered essential elements of the embankment although grouting per se is not covered in detail.

A brief chapter on the development of earth and rockfill dams during the past century is presented to show the overall development of trends in design and construction. This is followed by chapters on current practise with respect to field explorations, laboratory testing, embankment design, construction, instrumentation and performance. The paper concludes with a brief chapter on present problems and areas of future concern.

The greatest emphasis is placed upon the developments of the past decade which have led to current practise in the design and construction of earth and rockfill dams.

1. HISTORICAL

1.1 Failures

"There is probably no type of structure built by man which offers so great a potential hazard to life and property as a large dam and reservoir upstream from a heavily populated area." This statement was made three decades ago by Morris (1939) but its significance is even greater today than it was on the afternoon of May 31, 1889 when a dam across the Conemaugh River in Pennsylvania burst after being overtopped. In the resulting flood, many thousands of people lost their lives and about $40,000,000 worth of property was destroyed (Ferris 1889) in the town of Johnstown and elsewhere in the Conemaugh Valley.

Tragic as the Johnstown Flood was, it is significant that the cause of the failure was overtopping resulting from insufficient spillway capacity. The dam itself, as described by Mr. H. W. Brinkerhoff to the Engineering and Building Record was about 80 feet high with upstream slopes of 2:1 and downstream slopes of 1.5:1, apparently built in layers of "puddle of very good quality". It was apparent early that fateful morning that overtopping was imminent, but shouted warnings from a man on horse back went unheeded because of frequent prior warnings that had proven uneventful. Overtopping took place at 11:00 A.M., but final failure did not develop until about 3:00 P.M.; this is an excellent example of the resistance of an earth dam containing cohesive soil to transient overtopping.

Middlebrooks (1953) tabulates approximately 200 earth dams which have failed or whose performance has been unsatisfactory. Less than 20 of these took place in dams that were built after 1925. Only 25 were over 100 feet in height. The most common causes of failure were either overtopping or piping.

Babb and Mermel, (1968) have prepared a list of some 600 entries of dam disasters, failures and accidents. The authors point out, however, that the term "failure" may have many meanings when applied to dams and that the entries can generally be classified into groups described as follows:

> " (a) Major disasters. The sudden and complete failure of a dam while in service. Usually with total destruction and loss of the dam and of life and property.
>
> (b) Failures and washouts of minor dams constructed without benefit of professional skill and having little engineering significance.
>
> (c) Failures to gates, valves, piers, and appurtenant structures, including cracks, leaks, and erosion where the dam as such did not fail but received publicity associated with its name.
>
> (d) Accidents and failures during construction and before the dam was ready for service. Usually corrected and not inimical to the satisfactory functioning of the dam after completion.
>
> (e) Ancient dams about which little is known, and predating any modern area of dam building which might be arbitrarily selected."

Obviously group (a) is of major concern to the engineering profession, but while a failure of a minor dam a decade ago may be of little significance, a failure of the same dam a decade from now could fall into the category of a major disaster. The volume of water impounded by the Baldwin Hills Reservoir in Los Angeles, for example, was small yet the property damage was relatively large because of the commercial and residential developments just downslope which were serviced by the reservoir.

1.2 Ancient Dams

Earth dams comprise some of man's oldest works, being used initially to store or divert water for irrigation. Rao (1951) describes ancient earth dams of India constructed in the period 800 A.D. to 1600 A.D. Not many details are known of these early dams, but even then certain fundamentals of good site selection and construction were recognized, including the following:

> "2. A learned man in the science of Pathas and Sastras (i.e.) Hydrology.
>
> 3. A ground of hard soil.
>
> 5. Two projected portions of hills in contact with it (the river), for the site of the tank (dam).
>
> 12. A gang of men skilled in the art of tank (dam) construction.

It is known that these early dams were often breeched, primarily by overtopping because of insufficient "waterway" capacity, but also by piping or seepage instability. The number one fault to be avoided was stated as "water oozing from the dam".

1.3 Nineteenth Century Dams

The period 1800 to 1900 A.D. marked the development of the zoned earthfill dam, the hydraulic-fill dam and the rockfill dam. Unfortunately, the literature of this period is full of vague and conflicting descriptions of the soil types used for construction. Bassell (1907) in a study of earth dams attempted to define and clarify some of these terms, but stated:

> "The writer had intended to present a table of physical properties of different materials, giving their specific gravity, weight, coefficient of friction, angle of repose, percentage of imbibition, percentage of voids, etc., but found it impossible to harmonize the various classifications of materials given by different authorities."

This confusion was not fully resolved until the U.S. Corps of Engineers officially adopted the Unified Soil Classification based upon the Airfield Classification System originally developed by Casagrande (1948).

Schuyler (1912) describes the six ways in which earthen dams were usually constructed:

> "(1) A homogeneous embankment of earth, in which all of the material is alike throughout;
>
> (2) An embankment in which there is a central core of puddle consisting either of specially selected natural materials found on the site, or of a concrete of clay, sand, and gravel, mixed together in a pug-mill and rammed or rolled into position;

(3) An embankment in which the central core is a wall of masonry or concrete;

(4) An embankment having puddle or selected material placed upon its water-face;

(5) An embankment of earth resting against an embankment of loose rock;

(6) An embankment of earth, sand, and gravel, sluiced into position by flowing water."

The use of puddle, either as a central impervious core or as a homogeneous dam (as in the Conemaugh River dam) deserves clarification. Bassell (1907) defines it in the following manner:

> "Puddle without qualification may be defined as clayey and gravelly earth thoroughly wetted and mixed, having a consistency of stiff mud or mortar. Puddle in which the predominating ingredient of the mixture is pure clay, is clay puddle. Gravel puddle contains a mugh higher percentage of grit and gravel than the last named and yet is supposed to have enough clayey material to bind the matrix together and to fill all the voids in the gravel."

It appears that the European engineers favored the central puddle-cores whereas the American engineers preferred the masonry core-wall, or as an alternate, a puddle facing on the inner slope of the embankment. Glossop (1968), in the Eighth Rankine Lecture, comments:

> "British practise differed in that, following Telford, a core wall of puddle clay was carried up to the full height of the dam to form an impermeable membrane, which was considered economical by limiting the amount of careful construction!
>
> There was much argument about this point, the British engineers maintaining that since the quantity of puddle was relatively small it would be 'heeled in' under very careful control, and was completely impermeable; so much so that some believed that drains in the downstream side of the embankment were unnecessary, and might be positively dangerous, one speaker saying that 'he did not follow the author's meaning as to leakages having to be dealt with, because the endeavour in England was to stop all leakage.' However, the author very wisely maintained the importance of thorough draining of the embankment on the downstream side of the puddle core."

In general, very little attention was paid to the control of moisture contents in embankments prior to the work of Proctor (1933), although Schuyler (1912) clearly sets forth the basic requirements for compaction and water content control:

> "In building earth dams of any type it is essential that the earth should be moist in order to pack solidly, and if not naturally moist it must be sprinkled slightly until it acquires the proper consistency. An excess of moisture is detrimental. It should be placed in thin layers, and thoroughly rolled or tamped, and the surface of each layer should be roughened by harrowing or plowing before the next layer is applied. Droves of cattle, sheep, or goats are often used with success as tamping-machines for earth embankments. They are led or driven across the fresh made ground, and the innumerable blows of their sharp hoofs pack the soil very thoroughly."

1.4 Hydraulic Fill Dams

Extensive hydraulic mining on the Pacific Coast of the United States, which followed the discovery of gold in California in 1849, led to the development of the hydraulic-fill dam. It soon became recognized as the most economical method of handling earth, and in comparison with the then accepted procedures for placing earth without compaction control, it was considered the most positive and satisfactory means of compacting it in a solid and immovable mass.

The principles of hydraulic-fill dam construction are well described by Schuyler (1912), as follows:

> "To secure stability and perfect drainage it is essential that the outer slopes of the embankment, composing about one third of the mass, equally divided between the up-stream and down-stream slopes, shall be composed of rock, gravel, or sand, which will afford friction and stability, as well as drainage to the interior of the dam. This interior two thirds is the water-tight section, composed of the finest clay or silt as segregated from the other material. It is at first thoroughly unstable and would slide from its position if unsupported by the stable sections on either side. With the slow settlement following gradual drainage and the pressing of the water from its voids, it becomes more and more dense, and finally assumes a condition or mature solidity and impermeability that fits it to resist the pressure of the water in the reservoir behind it. The best practice in hydraulic-fill dam construction is to test the structure at all stages of its growth, by permitting the reservoir to follow up the rising dam, always 10 to 15 feet below the top (at the same time maintaining a pond of water on top of the embankment), although this is not always feasible. When such a water-level is maintained against the dam as near the top as possible, the principal drainage of the interior mass of the dam will be out through the downstream zone of permeable material, permitting of partial filling of the voids in the up-stream permeable zones with fine silt."

Although there were many failures of early hydraulic-fill dams, mostly during or immediately after construction, there were also many successful dams built by this method which are still serving a useful purpose.

Stringent safety requirements now being applied to these older dams by State and Federal Agencies have forced a detailed re-examination of their stability. In some cases these studies have revealed marginal or sub-standard factors-of-safety against seismic loading with the result that buttressing, slope flattening and improved drainage are required prior to re-licensing. Although the owners may understandably object to improving the stability of a dam whose performance has been satisfactory for 50 or more years, age itself is not proof of resistance to unusual events such as severe earthquakes.

During construction of the Fort Peck Dam, the world's largest hydraulic-fill dam, a major failure developed in the foundation sands and in the saturated upstream portion of the dam. Casagrande (1950) states that a weakness in the underlying Bearpaw shale developed strains in the sand which transferred load to the pore water, thus increasing shearing stresses in the foundation which ultimately culminated in both foundation shear failure and liquefaction of the sands. The resulting thorough investigations are well described by Middlebrooks (1942).

The Fort Peck failure was subsequently repaired, also by hydraulic methods, but this time compaction procedures were incorporated. However at about this time the development of large earth-moving and compaction equipment, and the systematic use of compaction control procedures led to the abandonment of the hydraulic fill dam, except where used to retain deposited waste piles, such as from ore processing plants. However, there are many engineers today who believe that with adequate design and construction control, hydraulic fill dams can be constructed both economically and safely.

1.5 Rockfill Dams

The true rockfill dam also had its origin in California in the middle of the 19th Century at about the same time as the hydraulic-fill dam. Up until about 1940, a rock-fill dam was defined as being composed of three elements: A loose rock-fill forming the mass of the dam; an impervious face next to the water; and a rubble cushion between the two. Galloway (1939) further describes the action of rockfill dams:

> "The characteristic that differentiates rock-fill dams from other types is that the element resisting the thrust of the water pressure is of loose rock of varying sizes, placed as a fill at an appropriate site. In almost every case, the rock is dumped loosely in position and there is no attempt at orderly arrangement of the individual rocks; nor is there any other material introduced to bind the rocks together. The mass of rock is somewhat consolidated when placed in position and further consolidation takes place by settlement under load and the action of the weather.
>
> Resistance to the forces imposed by water is obtained from the weight of the mass of rock in the dam. There can be no arch action; nor can there be any action such as the cantilever offset of a gravity masonry dam."

Unusual combinations of earth and rock-fill were built in the late 1800's, and many of these are still serving a useful function. Waialua Dam, Hawaii (Fig. 1), is an interesting combination of upstream hydraulic fill, central wooden diaphragm with concrete cut-off wall, and downstream loose rock fill with hand-placed rock facing on a 3V on 2H slope (Schuyler 1912). The dam was completed in 1905 and there appears to have been no settlement of the embankment whatever after its completion although there was some "bulging" of the downstream stone facing during construction. In 1966, an inflatable rubber dam was placed across the spillway to increase the storage capacity and as a result downstream springs developed. Although subsequent investigation revealed that the wooden diaphragm was still effective in reducing seepage and that the dam was adequately safe for the design criteria, it was decided to buttress the downstream slope with a compacted earth fill in order to increase the resistance to earthquake action.

In the early rock-fill dams both upstream and downstream faces were built on slopes of 1:1 or steeper, which made it necessary to protect each face with rubble in order to retain the loose rock fill. The term rubble is not an apt description since it refers to the hand-placed layer of rock placed between the impervious facing of the dam and the loose rock of the main fill. Its function was to act as a semi-rigid member between the rigid facing and the loose fill which was subject to settlement. Even so, it was recognized that the proper application of this type of dam was to storage reservoirs which are emptied at intervals in connection with their use, thus exposing the face for repairs.

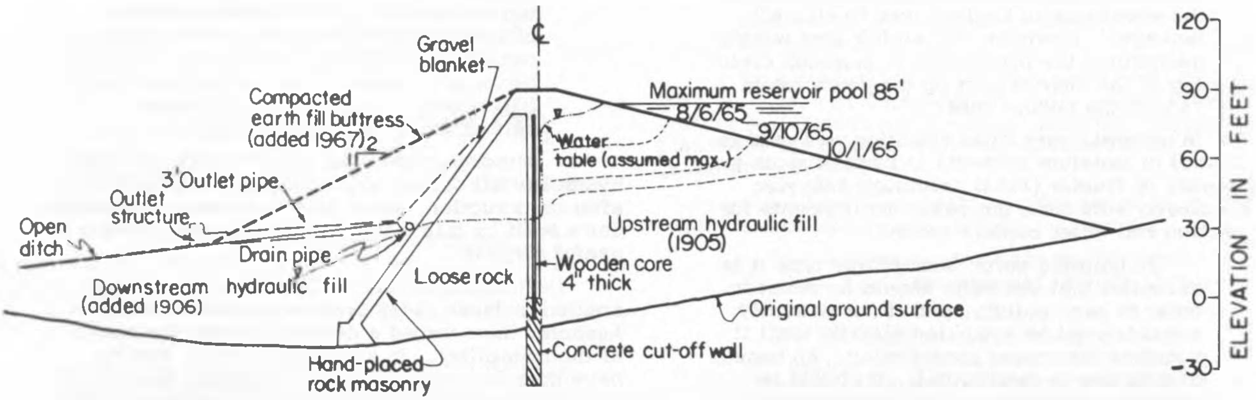


Fig. 1 Typical Section of Wahiawa Dam, Hawaii

The proper gradation, methods of placing (including sluicing) and compaction have been controversial items of discussion from the start. Galloway (1939) stated:

> "The nature of the rock-fill is one upon which difference of opinion will develop. It is believed that the fill should be composed of individual rocks of fairly uniform size, one rock bearing directly upon another, usually expressed as "rock to rock." Any wide divergence in size will cause excessive and unequal settlements, something to be avoided wherever possible. To illustrate the idea, if one selects a sample from a mass of broken stone of fairly uniform size, it will be found extremely difficult to force the sample into the mass by added pressure. On the contrary, if a large rock, which may rest upon small gravel or sand, is given a load, it will settle into the mass by displacing the smaller grains. In the usual method of dumping rock into the fill the larger ones roll to the bottom and the smaller ones remain at the top. A graded fill results."

The opposite viewpoint and one that conforms to present thinking was taken by Peterson in his discussion of Galloway's paper (1939):

> "In the writer's opinion an unsegregated quarry-run mass, with excess fines wasted, will give the highest degree of contact, rock to rock, and the greatest resulting fill density. Such a fill will have ample rock-to-rock contact and will still have sufficient voids to provide adequate drainage."

Most authorities, even today, agree that plentiful sluicing of the fill during construction will facilitate and accelerate settlement. There is less agreement on why this is this case. One author states that the purpose of sluicing is "to wash the fines into the fill", while another states with equal conviction that the purpose is "to wash the fines out of the fill". A more commonly accepted explanation today is that wetting weakens the rock at the points of contact, permitting them to break down and redistribute the load. The late K. Terzaghi once confided to the author that although he was not certain of the mechanism, he would not dare to build a dumped rockfill dam without sluicing. However many compacted rockfills have been built without watering or sluicing, with apparently good results.

1.6 Modern Earth and Rockfill Dams

The year 1940 was the start of a gradual revolution in the design and construction of earth and rockfill dams. The failure of the Fort Peck Dam marked the end of the large hydraulic fill dam, although this type of construction was already marked for obsolescence because of the development of improved earth moving and compaction equipment.

The rockfill dam was modified by replacing the rigid upstream facing with a more flexible type. The 250-foot-high Nantahala Dam in North Carolina was the first major rockfill dam with a sloping clay core. One significant feature of this dam was the placement of protective filters of varying thickness and gradation on each side of the sloping core. This type of design increased in popularity until the late 1950's, Brownlee Dam on the Snake River in Idaho (1958) being one of the last major dams of this type. Brownlee also was unique in that the downstream shell was constructed on a 110 foot thick alluvial deposit of dense sand, gravel and cobbles. It is interesting to note that because of the lack of large, uniform-size quarry rock the basic design of Brownlee was modified during construction to include a zone of well graded rock compacted in layers directly behind the core.

The United States Bureau of Reclamation first applied the principles of Classical Soil Mechanics to the design of earth dams in about 1940 and from 1940 on dams have been built to greater and greater heights (Fig. 2), with an increase in the ratio of the number of earth dams built to those of masonry and concrete as shown in Table I, (Johnson 1968).

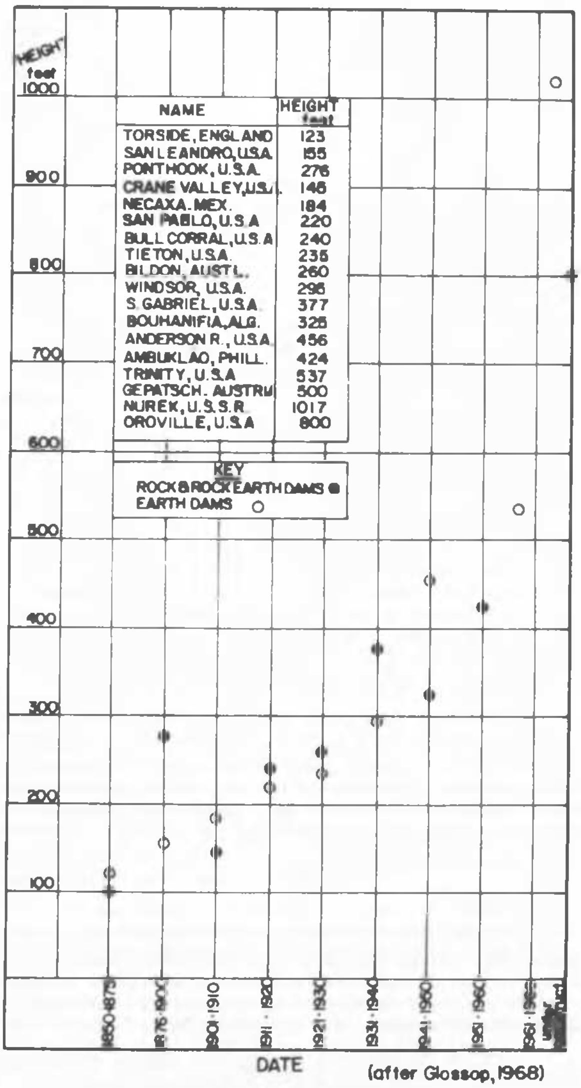


Fig. 2 Maximum Heights of Earth Dams From 1850 to 1968

Table I Tabulation of Dams in the United States

Dates	Concrete Gravity	Concrete Arch	Earth-fill	Rock-fill	Earth and Rock	Combination Concrete Gravity and Earth and Rockfill	Unclassified and Other	Totals
Pre-1940	255	136	521	34	40	58	296	1,340
1940-1949	51	16	179	6	10	9	27	298
1950-1959	88	16	495	12	18	27	20	676
1960-1965	22	12	516	15	23	19	11	621
Under con. Jan. 1, 1966	26	9	149	18	16	18	5	238
Proposed Jan. 1, 1966	14	4	161	20	20	27	15	261
Totals	456	193	2,021	105	127	158	374	3,434

(Reprinted from Water Power, Nov. 1968)

Stability analyses and laboratory testing techniques likewise progressed rapidly, with the U. S. Corps of Engineers sponsoring a cooperative triaxial research program with Harvard University (Waterways Experiment Station, 1947).

As in all branches of engineering and major construction, there have been failures along the way, some catastrophic. The overtopping of Vaiont Dam in Italy in 1963 brought to the attention of the profession the need for surveillance of reservoir slopes. The failure of Baldwin Hills Reservoir also in 1963 re-emphasized the problems of piping and internal erosion of dams built of or on erodable materials.

Remarkably few earthquake-induced dam failures have occurred, although there have been liquefaction of tailings dams such as in Chile on March 28, 1965 (Seed 1968). Seed also describes the partial failure of Sheffield Dam near Santa Barbara, California, 1925. Hegben Dam in Montana was severely shaken and damaged by an earthquake in 1959 (Sherard 1963), but failure did not occur.

With increasing State and Federal control and licensing of dams as well as increases in population downstream from dams, greater emphasis than ever before is being placed upon hydrological studies and the construction of reserve or emergency spillways, together with increased freeboard.

2. FIELD EXPLORATIONS

2.1 General

For the design of earth and rockfill dams, extensive field investigations, laboratory studies and office studies are required. These are at first general in nature but then become more detailed as specific questions arise during the course of the studies.

During the preliminary design phase, data are accumulated for an evaluation of project feasibility at one or several sites and for an estimate of project costs. Once this phase is completed and the type and locations of the dam and appurtenant works selected, further and more detailed studies are necessary to complete design. These studies are designed to fill specific gaps in the information available on subsurface conditions and to define to a greater extent the engineering properties of the proposed embankment materials. In both the preliminary and the final design phases of the study various types of investigations and studies are made which, for the purposes of discussion, can be grouped into the following: (1) Field Explorations (2) Laboratory Testing and (3) Embankment Design.

2.2 Geological and Foundation Investigations

As an initial step in this phase, all the available information concerning the proposed site or sites is obtained such as geologic, soil and topographic maps. These data are usually supplemented by aerial photographs from which more detailed topographic information can be obtained. Aerial photographs, particularly stero-pairs, provide broad coverage of land forms, including landslides, surface drainage, rock and soil outcrops and major structural features such as folds or faults. Experience has shown that many of these features can be more readily identified from aerial photographs than from the ground. Increasing use is being made of color photography and of special techniques such as infra-red and other filters.

From studies of the available information, programs of field work are planned which broaden or add to the existing knowledge of site conditions. Such field programs consist of a comprehensive field reconnaissance by engineers and engineering geologists and a scheme of subsurface explorations usually consisting of a series of drill holes. The drill holes are located for the most part within the limits of the embankment and in the areas of the spillways, powerhouse and other major works. However, other locations are often selected to provide specific information on such features as groundwater conditions, fault zones, buried channels and other features of a similar nature.

At times, various devices, such as optical, photographic or television probes are inserted in the drill holes to examine the in-situ characteristics of the foundation rock, such as the orientation of fracture patterns and seams, soft zones, rock contacts and voids. An interesting example of the use of borehole and television devices at the Manicouagan 5 project in Canada was described by Baribeau (1967). These devices were used to examine the extent and the characteristics of numerous sand seams and widespread glacial rebound fractures in the rock underneath the main dam. Lundgren, et al (1968) summarizes the state-of-the-art of borehole cameras and television devices.

As design progresses, shafts, tunnels and trenches may also be excavated to permit more detailed examination of subsurface features. At El Infiernillo Dam in Mexico, exploratory tunnels under the river were used subsequently for grouting and finally for drainage. Abutment adits were later converted to drainage galleries. Large-diameter calyx drill holes permit visual inspection of subsurface materials.

Geophysical surveys may be used to broaden the information obtained in the drill holes, tunnels, and shafts. These surveys help delineate the depth of overburden, zones of weathered rock and, in some cases, the quality of the rock, i.e., the intensity of jointing, fracturing and bedding. Geophysical programs may include refraction surveys as well as resistivity, interhole, uphole and sonic-logger surveys. Both P and S-wave velocity determinations are often made. The geophysical measurements of rock properties in-situ have been described by Wantland (1963).

At Mossyrock Dam in Washington, geophysical soundings were made in a series of holes prior to excavation, after excavation, and several times during concreting and reservoir filling. The holes were located beneath the dam and were extended through the concrete to the foundation gallery.

Probably the most interesting aspect of the geophysical program at Mossyrock was the change of the amplitude of the micro-seismogram signal from one reading period to another. An increase in the amplitude of the wave indicates a closing of joints and fractures in the rock. By visually comparing changes in the amplitude, it was possible to qualitatively determine the effects of excavation, the dead load of the dam, and reservoir loading. As expected, the interpretation showed that, in general, joints and fractures opened during excavation and closed during subsequent loading. Indicated deformations, however, were not uniform but varied with rock type. In some instances, the observed amplitude changes indicated the opposite of the anticipated result. Much of the data collected during reservoir filling has not been evaluated as yet and none of the results have been published (Fucik, 1969).

The most important end result of all the geological investigations is the preparation of a geologic map that shows the character and distribution of the exposed surface materials in the project area, including rock outcrops and soil overburden, and the various structural features of the rock such as faults, folds and stratigraphy. The locations of springs and marshy areas are also carefully noted along with other significant items such as areas of existing or potential instability in the reservoir rim and the spillway and powerhouse areas. On the geologic sections, the depth to bedrock is outlined as well as the pertinent characteristics of the overburden and the rock.

2.3 Reservoir Studies

The Vaiont catastrophe in Italy (Kiersch, 1964) and other failures such as the Baldwin Hills failure in California (Jessup, 1964) have drawn increased attention to the hazards inherent in the instability of reservoir slopes and bottoms. It is presently recognized that geologic investigations plus other studies must be devoted to an assessment of the geologic features observed in the reservoir rim and how they may react or otherwise change as a result of filling the reservoir.

Since the Vaiont landslide, other instances have occurred wherein reservoir filling has triggered mass movement in the reservoir slopes. Both Breth (1967) and Lauffer, et al (1967) describe the movements of a large mass of material into the reservoir of Gepatsch dam during the first and subsequent periods of reservoir filling. Involved was about 20 million m^3 of material over a length of about 1000m, composed of moraine and talus materials. As noted by Lauffer, et al, extensive explorations were conducted, including drill holes, seismic surveys and the excavation of tunnels. Analytical and then model studies were conducted, the latter to evaluate the effect of rapid failure on the generation of flood waves in the reservoir. The studies revealed that the movements were triggered by uplift hydrostatic pressures, but that catastrophic movements were unlikely because the movements toward the reservoir tended to restore equilibrium.

Mizukoshi, et al (1967) discusses the extensive geologic studies conducted on some reservoir banks in Japan. They attribute much of the existing instability to the extensive cracking and consequent loosening of the bedrock in the reservoir from prior tetonic movements and the formation of deeply eroded valleys at the toes of the slopes. All these features combined with heavy rainfalls, earthquakes and submergence tended to result in slope movements.

Pre-existing landslides are likely to be reactivated by reservoir filling and drawdown. Numerous such examples occurred along the reservoir rim upstream from Santa Rosa Dam in Mexico on relatively gentle slopes of volcanic tuffs and rhyolites. Installation of precise horizontal extensometers across the upper scarp permitted continuous monitoring of the rate of movement. Movements gradually diminished with repeated filling of the reservoir, Fig. 3 (Marsal 1969) and no corrective treatment is presently contemplated.

Along the Eel River in Northern California, U.S.A., an extensive landslide study is being undertaken by the Department of Water Resources well in advance of dam construction. The study includes detailed geologic mapping, precise triangulation surveys, and installation and reading of piezometers and inclinometers with the intent of identifying and stabilizing critical areas prior to filling of the reservoir.

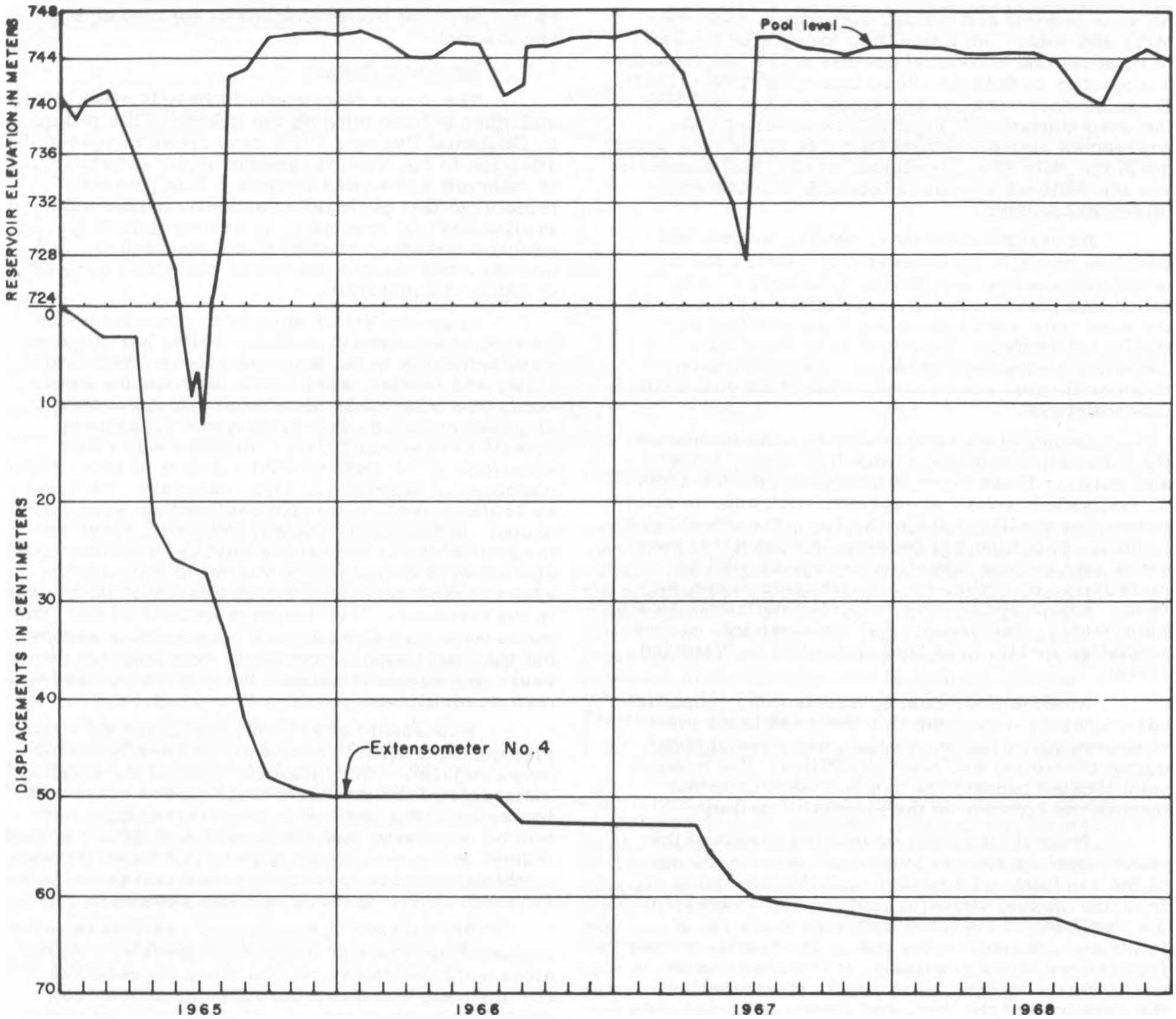


Fig. 3 Slide Movements in Reservoir Above Santa Rosa Dam, Mexico

Various governmental agencies in the United States have taken positive steps to prevent a catastrophe similar to that at Vaiont. The California law in 1965 was strengthened to require that special attention be paid to the margins of reservoirs (Jansen, 1967). The Bureau of Reclamation has adopted a program aimed at decreasing the possibility of destructive landslides in reservoirs. At existing reservoirs, the program calls for field officials at dams to examine reservoir rims in potential slide areas when they have been subjected to abnormal conditions, such as unusually heavy rains or an exceptionally long rainy season, heavy spring runoff, rapid drawdown or long continued wave action. At new reservoirs, and as reservoir filling commences, periodic examinations are made, which are continued through at least the first several seasons of filling and drawdown until at least the maximum water level and the critical drawdown have been experienced (Dominy, 1967).

Another problem associated with reservoirs is the effect of the first filling, and also subsequent operation, on the stability of the foundations for and the embankments of newly relocated highways and railroads. This has been an especially severe problem in connection with relocations around the reservoirs of the Columbia River dams in Washington, U.S.A., where there are thick deposits of talus and wind-blown silts and sands that are loose and have never been saturated. Around the John Day Dam, where approximately 175 miles of relocations were required particular attention was paid by the Corps of Engineers to excavation of loose foundation materials and to compaction control of embankments. The resulting performance has been excellent with only modest settlements (of the order of several inches) and no major landslides.

Increasing attention is also being given to the effects of the reservoir on the development or increased frequency of earthquake tremors. In a personal communication, J. L. Sherard notes that there appears to be an increasing amount of evidence to support the concept that the creation of a reservoir and the significant changes in stress that result could in some instances be sufficient to trigger crustal movements along old faults or new ones. Sherard notes that the filling of the reservoirs of such dams as Kariba Dam in Southern Rhodesia and Kremasta Dam in Greece among several others could be related to the commencement or increased activity of earth tremors. He concludes that the effects of possible crustal movements on the performance of the dam must be considered by a designer.

2.4 Groundwater Investigations

The nature of the groundwater, i.e., whether it is normal, perched or artesian and the variation of one form to another in the reservoir, foundations and abutments is important in design. For example, whether the groundwater observed in the abutments is perched or normal is significant with respect to the characteristics of seepage in and around the abutments. The rise in groundwater table away from the river is significant with respect to potential leakage from the future reservoir. In addition, the stability of abutment slopes and potential settlements of foundations upon first filling are directly related to the position and nature of the existing groundwater table. Changes as a result of first filling are also important to consider, such as the effects of an upward readjustment of the regional groundwater table on the stability of reservoir slopes.

Groundwater observations are made during the drilling operations as well as afterwards. The degree of permeability of bedrock formations and other deposits is often obtained in a qualitative sense from pumping tests or from bail-out tests wherein the drill hole is bailed out and the rate at which the water level returns to a static condition observed. In addition, observations of the groundwater level in a number of drill holes are usually made over a period of time to record fluctuations with the seasons or to observe relationships with the rise and fall of river levels.

Contours of the groundwater levels may be drawn which reveal the characteristics of flow toward the valley. At times such contours may reveal subsurface anomalies caused, for example, by faults or by intrusive bodies. In other instances, the characteristics of groundwater seepage can be investigated by the use of electrolytes or radioactive isotopes. In some cases sensitive velocity meters are lowered down the hole to detect zones and directions of inflow and outflow.

2.5 Borrow Areas

Investigations in borrow areas are undertaken to determine the quality and quantity of the available materials. Routine procedures of investigation usually include (1) trenches excavated by bulldozers (2) pits opened by backhoe or dragline and (3) drill holes, sometimes of large diameter with bucket augers, supplemented by geophysical surveys. From these explorations, samples are obtained for laboratory and at times field testing.

O'Neill and Nutting (1963) describe the extensive borrow investigations that were undertaken for Oroville Dam. To investigate a 7000 acre tailing deposit for some 65 million cu. yds. of pervious material, about 71 dragline pits and 129 bulldozer trenches were made. To obtain representative samples of loose gravels and sands below the static water level, a special piece of equipment called the "hole excavator" was devised which consisted of a hydraulically operated clam shell on a boom attached to a 2-1/2 ton truck. For the impervious borrow located elsewhere, investigations were made with 30-in. dia. bucket auger holes: about 139 holes were required.

As part of the borrow investigations, testing was accomplished in the field. Such tests included gradation and in-place densities.

For fine-grained soils, determination of the natural water content for comparison with the optimum water content for compaction, is of great importance. In wet climates, the difficulties connected with drying out borrow materials that are too wet can increase costs and time significantly.

2.6 Field Tests

2.6.1 General

Field tests provide a means of obtaining more reliable information than can be obtained from laboratory tests, either because of the size or mass of the sample to be tested or because, from a technical standpoint, field tests afford the only satisfactory means of obtaining the required data. Examples of the latter are pumping tests in alluvium, grouting tests, quarry blasting, and large scale shear tests on bedded materials (such as clay shales or bedrock with weak layers of shale, lignite or mylonite). Of equal importance, field tests provide contractors with useful pre-bid information.

2.6.2 Tests on Embankment Materials

In some instances, direct shear tests are conducted on shell materials in the field when the materials are composed of pieces which are too large from a practical standpoint to test in the laboratory. At Muddy Run Dam, the shear strength parameters of the various shell materials were determined in a shear box with plan dimensions of 7.5 x 7.5 feet and a height of 2.8 feet. The test materials were composed of fragments of mica schist, weathered to different degrees and consisting of various shapes and gradations. With the application of the normal loads, measurements were also made of percent compression for an evaluation of material compressibility (Wilson and Marano, 1968). A similar type of test was conducted at Lewis Smith Dam on compacted broken sandstone, although in this instance, the dimensions of the shear box were 6 x 6 x 3 feet (Sowers and Gore, 1961). An interesting method of determining the angle of shearing resistance of rockfill composed of very large pieces of rock (40 to 60 cm) was reported by Kany and Becker (1967). In this approach the rockfill to be tested was placed in a circular-shaped embankment with a diameter in the order of 25m and a height of 3.5m. In the center of the fill, a cylindrical enclosure was constructed using four concrete wall segments, each 3m in height. In performing the test, the four segments were forced apart, developing passive pressures in the fill. On the basis of a passive pressure relationship derived from a laboratory model the angle of shearing resistance of the fill materials was calculated.

In some instances the permeability of the embankment materials is also determined in large scale field tests (Sowers and Gore, 1961).

2.6.3 Tests on Materials In-Situ

The in-situ shear strength of foundation materials is often obtained from field tests, particularly when the material exhibits preferred planes or zones of weakness, such as interbedded rock with layers or seams of shale, lignite or mylonite or, similarly, on bedded materials such as clay shales. To name a few, direct shear tests on rock with lignite seams (Schultze, 1957) and with shale seams (Pigot and Mackenzie, 1964) have been described in the literature. The inadequacy of laboratory tests in properly revealing the shear strength characteristics of clay shales and overconsolidated clays have been frequently noted in the literature by engineers of the PFRA, Canada (Peterson, 1968) and(Ringheim, 1964), although in this instance, greater reliance is placed on the shear strength computed from existing slopes, cuts and slides rather than on field shear tests per se.

Torsion vane shear tests are also useful in the determination of in-situ shear strength properties of soils and weak rocks. Wilson and Marano (1968) describe a torsion shear device (12-in dia.) used to obtain the angle of shearing resistance of a mica schist formation underlying the Muddy Run embankment. A much larger device (6 ft. dia.) has been developed in Mexico by the CFE, (Marsal, et al, 1965) to measure the shear strength of bentonitic and layered clay deposits, Fig. 4. Where sampling disturbance may adversely affect the results of laboratory strength

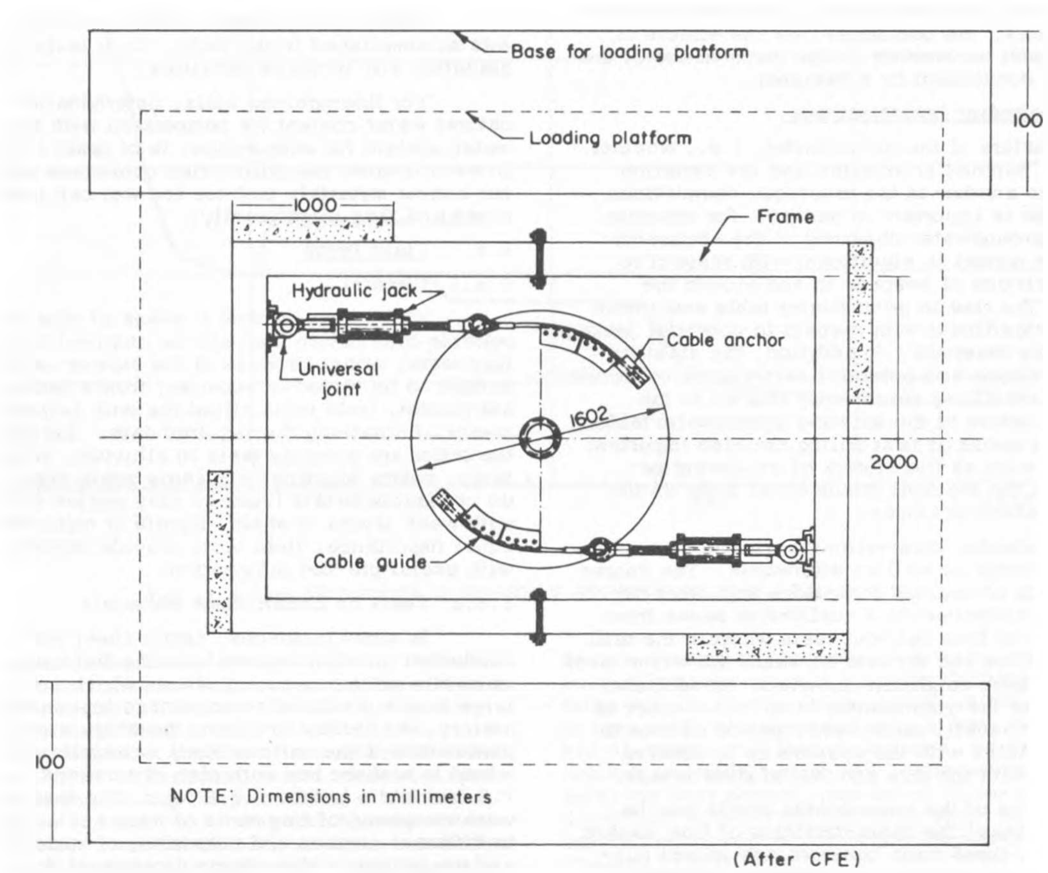
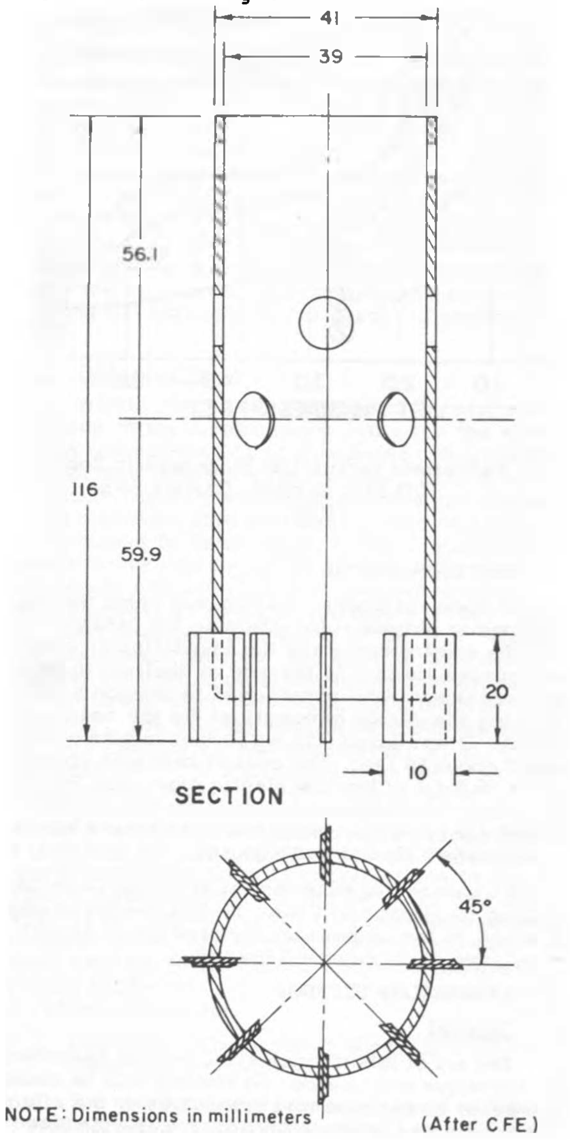


Fig. 4 Large-Diameter Torsion Shear Device (CFE, Mexico)

tests on clay, field vane shear tests have proved useful. Marsal (1969) describes a high-strength vane shear device developed by the CFE to investigate the in-situ shear strength of volcanic tuffs and stiff bentonitic clays (Fig. 5). This device has been used successfully in such materials having shear strengths of up to 10 k_g per cm^2.



PLAN
Fig. 5 High-Strength Vane Shear Device (CFE, Mexico)

Vane shear devices may also be valuable as a means for construction control. Esmiol (1967) describes the use of vane shear equipment in controlling the rate of construction of the 35 foot high Willard earthfill dam in Utah.

Wilson, et al (1962) describe horizontal jack tests used to evaluate bearing capacity and stress-deformation characteristics of glacial tills and slightly cemented sands and gravels. Such tests provide useful information at minimum expense.

Determination of the foundation modulus of elasticity is not customarily made in connection with the design of earth or rockfill dams. Newer techniques, when required, include the use of the Goodman Borehole Jack (Goodman, et al 1968) the Menard Pressure Meter (Gibson and Anderson 1961), the borehole dilatometer developed by Rocha of Portugal, pressure chamber tests (Monahan and Sibley, 1965) stress-relief tests using over-coring techniques, and flat-jack tests. The flat-jack modulus tests conducted at Reza Shah Kabir Project in Iran are unusual in that an area 3 m x 1.5 m (approximately 45 ft^2) is tested. The slot was cut by means of a conventional diamond rock-cutting disk, 1 meter in diameter. The disk is mounted on a guide that follows a previously drilled hole. In this way, depths in excess of the disk diameter may be cut. Cuts are made adjacent to each other to the total width desired (Fucik, 1969).

2.6.4 Field Permeability Tests

Field permeability tests are conducted at many sites, particularly if the dam is underlain by pervious, coarse-grained alluvium, composed of a wide range of particle sizes. In these deposits, a representative sample for laboratory testing cannot be obtained by any practical means, but just as important, sampling invariably alters the natural structure and porosity of the deposit, thereby masking the true nature of the permeability. Pumping tests are most often used to determine the in-situ permeability characteristics of deep, pervious, valley fills. Recently, at an ASTM Symposium, "Permeability and Capillarity of Soils", Lang (1966) described various pumping test methods for determining the permeability characterstics of aquifers. In a companion paper, the field determination of permeability by the infiltration tests (pump-in test) was discussed by Schmid (1966).

Data from these tests are used to estimate losses-and to design seepage control measures, such as grout curtains, drain holes and various others. To provide data for the same purposes, pressure testing of drill holes in bedrock is often done. In this test, a rod and packer assembly is lowered into a drill hole to a predetermined depth. Water is pumped in until the desired pressure is indicated on the pressure gage. After the desired pressure is reached, pumping is continued for a given time, usually about 5 minutes. At the end of this time, the quantity of water pumped into the hole is recorded from a flow meter. Successive tests at different depths permit a graphical plot to be developed which shows water loss in gallons per minute or in Lugeon Units at different levels in the drill hole.

In accordance with the U.S. Corps of Engineers practice, the gage pressure at any depth should not exceed one pound per foot of depth of the packer and in no case should the pressure exceed a maximum of 100 lbs. per sq. in. (Corps of Engineers, 1966), in order to control the opening of existing fractures or creating new ones by the test.

Other field tests may be programmed to arrive at the selection or adequacy of proposed design or construction procedures: these may include such field tests as grouting, quarry blastings, rock rippability tests, methods of densification, i.e., vibroflotation or blasting, and test fills and test embankments.

2.6.5 Test Fills

Test fills of embankment materials are increasingly being used as an important phase of the design studies. They reveal the best procedures for the placement and compaction of the materials as well as the resultant characteristics of the completed fill for the various procedures that are used. Fig. 6 shows the layout of one of the test fills constructed at Carters Dam in Georgia and the thicknesses of the layers compacted with a 5-ton and 10-ton vibratory roller. The settlement of the various lifts was determined by taking level readings before and after compaction on numerous points on the surface of the fill, identified by spray paint. The resulting data (Fig. 7) shows the effectiveness of the two rollers for varying lift thicknesses and number of passes of the compactors. In addition, the trenches that were excavated in the test fills provided valuable information on the character of the compacted materials (Robeson and Crisp, 1966).

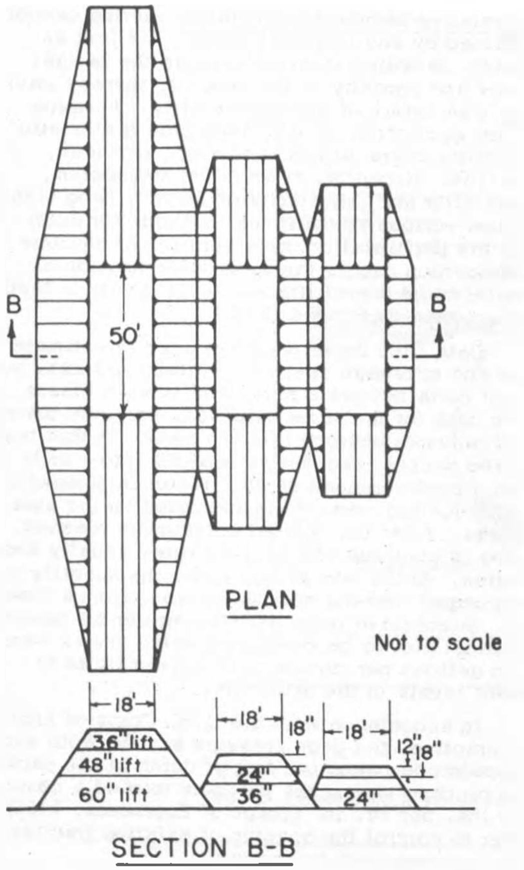


Fig. 6 Plan and Section of Rockfill Test Section, Carters Dam, USA

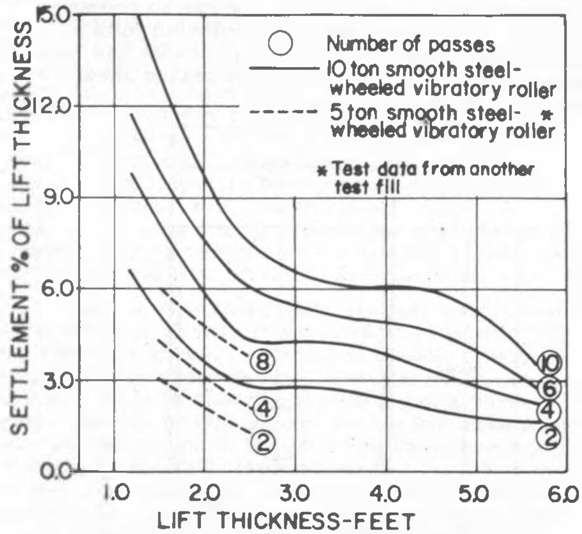


Fig. 7 Settlement Versus Lift Thickness in Rockfill Test Section, Carters Dam USA

2.6.6 Test Embankments

In some instances, test embankments may be constructed to provide information on the behavior of both the embankments and the foundation as a result of the imposed loads. At the site of Shellmouth Dam, a test embankment was constructed to provide a means of checking the design assumptions for the main embankment: it was about 250 x 850 feet in plan with a height of about 50 feet. The embankment was underlain by a deposit of medium plastic clay some 50 feet in thickness. Instrumentation consisted of horizontal movement gages, slope indicators, settlement markers and piezometers (Rivard and Kohuska, 1965).

An underwater embankment at Plover Cove Dam, Hong Kong (Guilford and Chan, 1969) provided valuable information on the underwater performance of an impervious core of decomposed granite.

3. LABORATORY TESTING

3.1 General

The art of laboratory testing is well described in the literature and, hence, no attempt will be made in this paper to describe test procedures or the effects of test procedures, sample preparation and the like on the physical properties or behavior of the tested materials. The discussion will review present practice of testing materials for use in embankments, including use of new apparatus but excluding discussion of such tests as Atterberg limits, mechanical analyses and others which are often of a routine nature.

Laboratory tests provide specific information on the properties of materials in order to evaluate their behavior under the loading and seepage conditions occurring during the life of the project. As noted by the Joint ASCE-USCOLD Committee on Current United States Practice in the Design and

Construction of Arch Dams, Embankment Dams, and Concrete Gravity Dams (1967) the determination of the strength of embankment and foundation materials, and their variation with time is a vital, yet often difficult, aspect of embankment dam design. At the present time, the shear strength of materials is usually determined by triaxial tests made under three different modes of load application and sample drainage. These are: (a) the consolidated-undrained (R) test; (b) the unconsolidated-undrained (Q) test; and (c) the drained (S) test. The latter test can also be conducted in the direct shear apparatus. The drained shear strength parameters of a sample can also be interpreted from a triaxial $\bar{R}$ test, wherein the pore pressures developed during an R test are measured. Normally, in both the S test and the $\bar{R}$ test on compacted soils, a back-pressure is applied to the porewater at some time in the consolidation phase of the test in order to achieve saturation of the sample prior to the application of the stress difference, $\sigma_1 - \sigma_3$. Lowe (1960) describes the application of anisotropically consolidated triaxial test data to the design of embankment slopes.

3.2 Cohesive Soils

Within the last decade extensive studies have been made of the factors which influence the shear strength characteristics of compacted cohesive soils. The effects of molding water content, method of compaction, density and structure on shear strength and, in addition, pore pressures were investigated and discussed by Seed, et al, (1960). Further comprehensive studies on the stress-deformation and strength characteristics of compacted clays were undertaken at Harvard University (1960-1964) in a research project sponsored by the U.S. Waterways Experiment Station. The studies, besides including the effects on shear strength of the factors mentioned, also considered the effects of time of loading. The studies were made under the direction of A. Casagrande and associates, R. C. Hirschfeld and S. J. Poulos and reported in Harvard Soil Mechanics Series 61, 65, 70 and 74.

As a result of the Harvard studies the following tentative conclusions were derived concerning strength testing for practical applications in the design of embankments. It may be noted that the usages of the various types of strength parameters, discussed below, are in general agreement with current practice in the United States as reported by the Joint ASCE-USCOLD Committee on Current Practice referenced earlier.

3.2.1 Strength of Compacted Clay for End-Of Construction Condition

For stability analysis on the basis of total stress the results from Q tests should be used provided certain requirements of testing are met. The requirements basically include checking on the influence of time of loading on shear strength; paying special attention to the control of leakage in long-term tests; performance of tests at constant temperature; and protection against vibrations. At the end of the test, the water content of the middle two-thirds portion of the specimen should be determined and correlated with the measured strength.

For stability analysis on the basis of measured pore pressures in the fill, usually the use of an S strength envelope will be slightly on the conservative side. Either triaxial or direct shear S tests may be used.

3.2.2 Strength of Compacted Clay for Long-Term Steady Seepage Condition

For an effective stress analysis the results of S tests should be employed. Either triaxial or direct shear S tests may be used provided certain requirements are met in the tests as follow:

If it is desired to correlate accurately the strength with the void ratio it is necessary to effect 100 percent saturation by means of a back pressure. In addition, the water content at the end of the test should be determined as mentioned above and correlated with the measured strength. If direct shear S tests are used, one should use thin large-diameter specimens, and in lieu of a confining perimeter wall, one should use a thin rubber membrane which is stretched over the sides of the discs, bridging across the edge of the specimen. The vertical dial readings should be accurately observed in such tests, preferably with more than one vertical dial so that any tilting will be noticed. In all long-term tests, care should be taken that metal parts of the apparatus are not in contact with the specimens.

3.2.3 Strength of Compacted Clay for Rapid Drawdown Condition:

For stability analysis the R or $\bar{R}$ envelope should be used together with a total or effective stress analysis, respectively, from R or $\bar{R}$ tests. Test requirements are similar to those outlined in Section 3.2.1 and in addition, a sufficient back pressure should be used to effect 100 percent saturation. During the consolidation phase of the test, ample time should be allowed for the completion of primary consolidation. Of additional importance, the pore pressure measuring system should consist of thin porous discs and of tubing of minimum diameter.

In addition to these tentative recommendations, a comprehensive discussion was made in the various series noted above, of areas requiring additiona research.

3.3 Granular Materials

The strength and volume change characteristics of gravel and rockfill materials under high confining pressures is rated a subject of high priority by the ASCE Committee on Earth and Rockfill dams (1967). In the United States reserach on these characteristics has been accomplished over the years by both the Bureau of Reclamation (Holtz and Gibbs, 1956) and the U.S. Corps of Engineers (Leslie, 1963) and to a degree by various private engineering firms and other governmental agencies, the most noteworthy of the latter being the Department of Water Resources, State of California. Outside the United States extensive research work on the behavior of granular materials is being accomplished, among others, by the Comision Federal de Electricidad and the Institute de Ingenieria, UNAM, Mexico under the direction of R. Marsal (Marsal, et al, 1965). In the laboratories in Mexico various unique pieces of equipment are available for large scale testing, including: (1) a high pressure (25 kg/cm^2) triaxial cell that tests specimens 113 cm in diameter and 250 cm high (max. particle size 20 cm); (2) a large scale compaction unit; and

(3) a plane strain unit, capable of testing dry samples of granular materials (max. particle size 15cm) under confining pressures up to 22 kg/cm^2 and maximum axial strains of 15 percent. In the latter unit, prismatic specimens, 70 x 75 cm in cross section and 180 cm high can be tested (Marsal, 1967) (Marsal et al, 1967).

In some instances, where large scale testing equipment is not available or not adaptable to the materials to be tested, the stress-strain characteristics and the strength parameters of coarse gravel and rockfill materials have been predicted from tests using a smaller particle size than the in situ material. At Goschenenalp Dam various gradations with a maximum particle size of 4-in. were tested and the relationships between gradation, porosity and shear strength developed for extrapolation to the shell materials, which contained a maximum particle size of about 24 in. (Zeller and Willimann, 1957). Lowe (1964) used a model approach, testing samples that contained sample grains 1/8 those of the prototype and which exhibited similar characteristics of shape in order to evaluate the shear strength characteristics of the coarse embankment materials (max. size 12 in.) for Shihmen Dam.

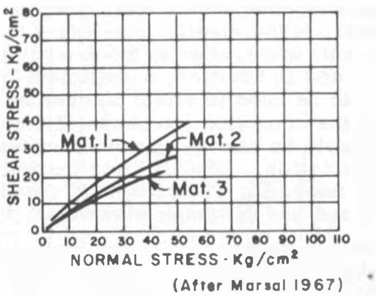


Fig. 8 Mohr Failure Envelopes for Three Rockfill Materials

Numerous programs of testing coarse-grained granular and rockfill materials have revealed a critical relationship between applied stress and the shear strength parameters in that the principal stress ratio at failure decreases with an increase in confining pressure (Hall and Gordon, 1963), (Bishop, et al, 1965), and (Nichiporovitch and Rasskazov,1967). Fig. 8 shows Mohr failure envelopes for three rockfill materials tested by Marsal (1967), which illustrate the drop in effective friction angle with increasing pressure. The properties of the materials tested are listed in Table II below.

The relationship between principal stress ratio at failure and confining pressure for the six materials listed in Table II is shown in Fig. 9.

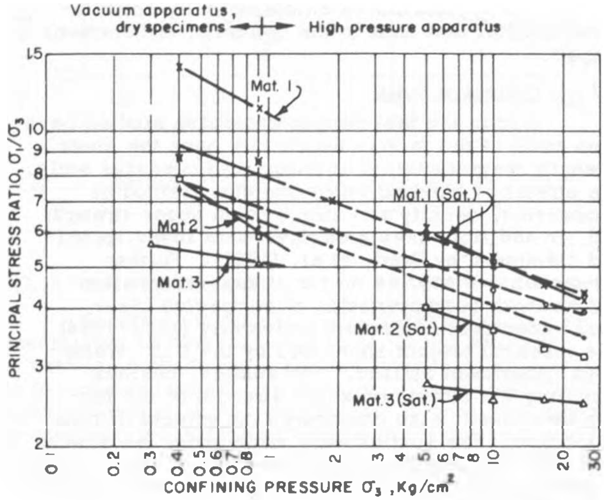


Fig. 9 Relationships Between Principal Stress Ratio at Failure and Confining Pressures

Table II Properties of Materials Tested

Material		Particle Shape	d_{10} mm	C_u	e*	P_a kg
Crushed, sound basalt	X (1)	Angular	1	19	0.30	860
Quarry-blasted, granitic gneiss	□ (2)	Sub-angular	6	14	0.32	130
Quarry-blasted, granitic gneiss	△ (3)	Sub-angular	53	2.5	0.62	130
Alluvial sand and gravel	✖ (P)	Sub-rounded	0.2	105	0.34	580
Quarry-blasted, sound silicified conglomerate	o (I)	Angular	5	15	0.45	230
Quarry-blasted, sound diorite	● (D)	Angular	20	5	0.56	170

*Void ratio, before testing

Hall and Gordon (1964) report the reduction in angle of internal friction for large-scale high pressure tests on minus 3-inch gravel dredger-tailings used in the construction of Oroville Dam as follows:

Table III Variation of Angle of Internal Friction for Pervious Materials

Sample No.	Major Principal Stress-psi 90	600	1200
	Angles - degrees		
Maximum Size	-3 inch	-1-1/2 inch	-1-1/2 inch
TP 15	43	40	37
TP 24	43	37	32
TP 28	45	42	39

Note: The angles reported in this table were measured from the origin and are tangent to the circles at the principal stress values noted

On the other hand, if the granular material contains an appreciable amount of fines, i.e., silt and clay, little if any decrease occurs as the confining pressures increase (Hall and Gordon, 1963), (Insley and Hillis, 1965). In the latter study, triaxial shear tests were conducted on 6-inch diameter specimens of glacial till material proposed to be used in the core of Mica Dam. The materials were compacted at three different moisture contents with a maximum particle size in the order of 1-1/2 inches. The maximum pressures that were applied reached 450 lbs. per sq. in. It was observed that only a very small decrease in the slope of the failure envelope occurred with an increase in confining pressure. The aforementioned relationship of a reduction in shear strength with an increase in confining pressure is significant with respect to the stability of embankments, particularly with the trend toward ever increasing heights.

As noted by Marsal (1967) and others the phenomenon of the reduction in shear strength with an increase in confining pressures is related to fragmentation or break-down of the grains comprising the specimen. This relationship is apparent from a study of Fig. 10. Of the factors influencing particle breakage, Marsal notes the following to be of significance: (1) confining pressure, σ_3; (2) grain-size distribution; (3) average size and shape of grains; (4) void ratio and others. For example, with respect to the effects of gradation, the studies by Marsal and Hall and Gordon (1963) showed that little degradation

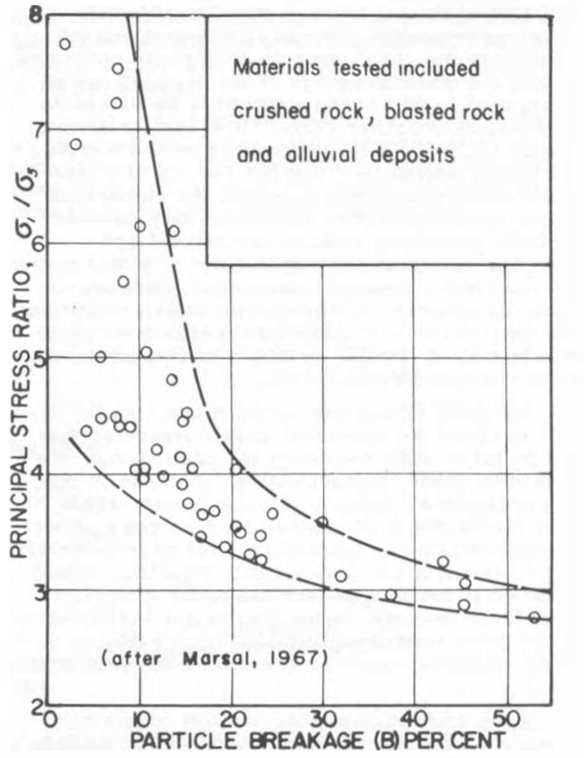


Fig. 10 Correlation of the Principal Stress Ratio At Failure and Particle Breakage

occurs in well-graded materials under large confining pressures and the application of shearing stresses and, hence, very little reduction in the principal stress ratio at failure occurs as compared with that occurring in uniformly graded material.

The reason for the particle breakdown and the decrease in principal stress ratio at failure with increasing confining pressure is believed to be the result of the high average intergranular forces at points of contact between individual grains. These forces increase with increasing partical size and with increasing coefficient of uniformity. Marsal (1963) has compared these intergranular forces for a typical sand and a rockfill, under an all-around confining pressure of 1 kg/cm^2 to be as shown in Table IV.

Table IV Computed Average Intergranular Forces

Material	Void Ratio	Equivalent Diameter cm	Average No. of Contacts per Particle	Number of Contacts per sq. meter	Average Intergranular Force - Kg
Sand	0.50	0.02	6	97×10^6	1.79×10^{-4}
Rockfill	0.70	20	7	39.7	400

(After Marsal 1963)

Note that the average intergranular forces at points of contact between the rockfill particles are of the order of two million times greater than for the sand particles.

With respect to the state-of-the-art in the evaluation of the behavior of granular materials from laboratory tests, Marsal (1969) cautions that the effect on the shear strength of both the specimen size and the gradation range of the samples tested as compared to that of the material to be placed in the dam demand further study. In a similar manner, the Joint ASCE-USCOLD Committee on Current Practices (1967) emphasized that the results of a laboratory test are appropriate only to the conditions existing in the laboratory specimen; they may vary from field conditions because the laboratory-compacted specimens may have quite different properties from field-compacted specimens, because of sample disturbance, or anisotropic stress conditions, or for other reasons. Laboratory research on plane strain testing of rockfill materials is now underway, see for example Marsal (1967).

The ASCE Committee on Earth and Rockfill dams (1967) outlined the defensive design measures that could be taken with respect to the phenomenon of a decrease in shear strength with an increase in confining pressure as follows: (1) use conservative values for the angle of internal friction and a careful consideration of a possible decrease in both shearing strength and permeability resulting from a breakdown in grain-size because of the high-loads imposed. In general, flatter slopes are indicated for high dams until investigations justify slopes comparable to those currently used on lower heights of dams.

Many soils laboratories are now programing and processing soils test data by computer methods to save time and manpower. An interesting report on this is published by the United States Bureau of Reclamation, (Knodel 1966).

4. EMBANKMENT DESIGN

4.1 Selection of Basic Dam Section

After the various preliminary studies such as the field investigations, laboratory tests, and engineering analyses are completed, a decision is made as to a site and to the basic type of dam to be founded at the site. Many factors govern the selection of the type of structure, but assuming adequate and comparable factors of safety the overriding consideration is economy. In recent years these factors have tended to favor the adoption of an embankment-type dam. The reasons for this are first that dams are being located where the foundations are not adequate for other types, and second that modern earth and rock-moving equipment utilizing locally available material often make such dams more economical even where foundation and abutment conditions are excellent. Often, however, the selection of type of dam is based upon the personal preference or past experience of the designing engineer.

World-wide preference for earth and rockfill dams relative to concrete dams is shown by Slichter (1967) in Fig. 11, based upon the World Register of Dams (1964).

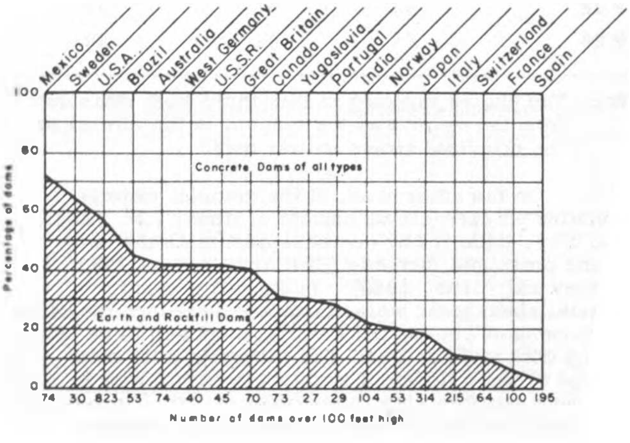


Fig. 11 World-wide Preference for Types of Dams

The term "embankment" in this paper includes all earth or earth-rockfill structures, ranging from simple homogeneous sections to multiple-zoned dams. Emphasis, however, is placed on the latter type wherein the dam is composed of an earthen impervious

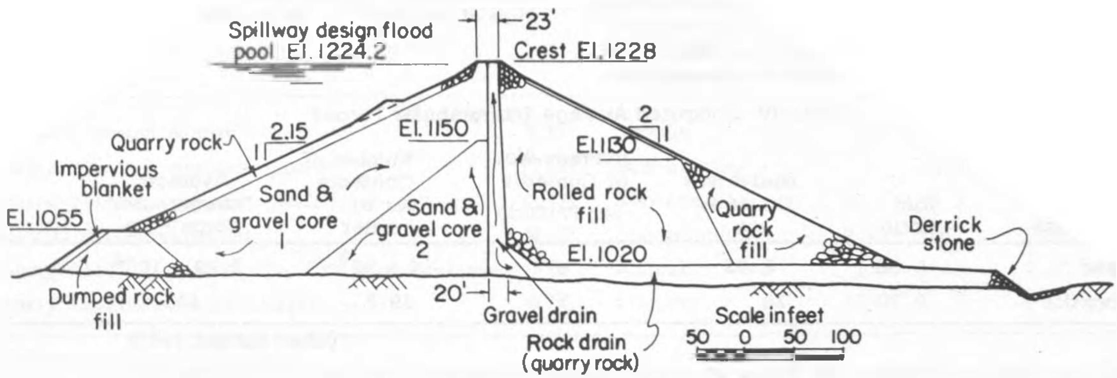


Fig. 12 Cross section of Howard A. Hanson Dam, Washington

core, bounded by transitions and shells of sand, gravel and rockfill or combinations of these materials.

An earth embankment may be single-zoned with internal drainage usually provided in the form of a vertical or inclined chimney drain, or the same basic embankment may be double-zoned. Fig. 12 shows a cross section of Howard A. Hanson Dam, Washington, a double-zoned dam, composed of an upstream relatively impervious section of sand and gravel and a downstream section of rockfill. A vertical chimney provides internal drainage. As noted by Bertram (1966) this type of design is particularly well-suited to glaciated regions where impervious glacial tills of high shear strength are found together with more pervious coarse-grained materials, i.e., sand and gravel or rockfill.

Fig. 13 shows several cross sections that can be classified from a standpoint of design as multiple-zoned. The shells of the sands are composed of rockfill or sand and gravel, whereas the cores are generally of highly plastic clay.

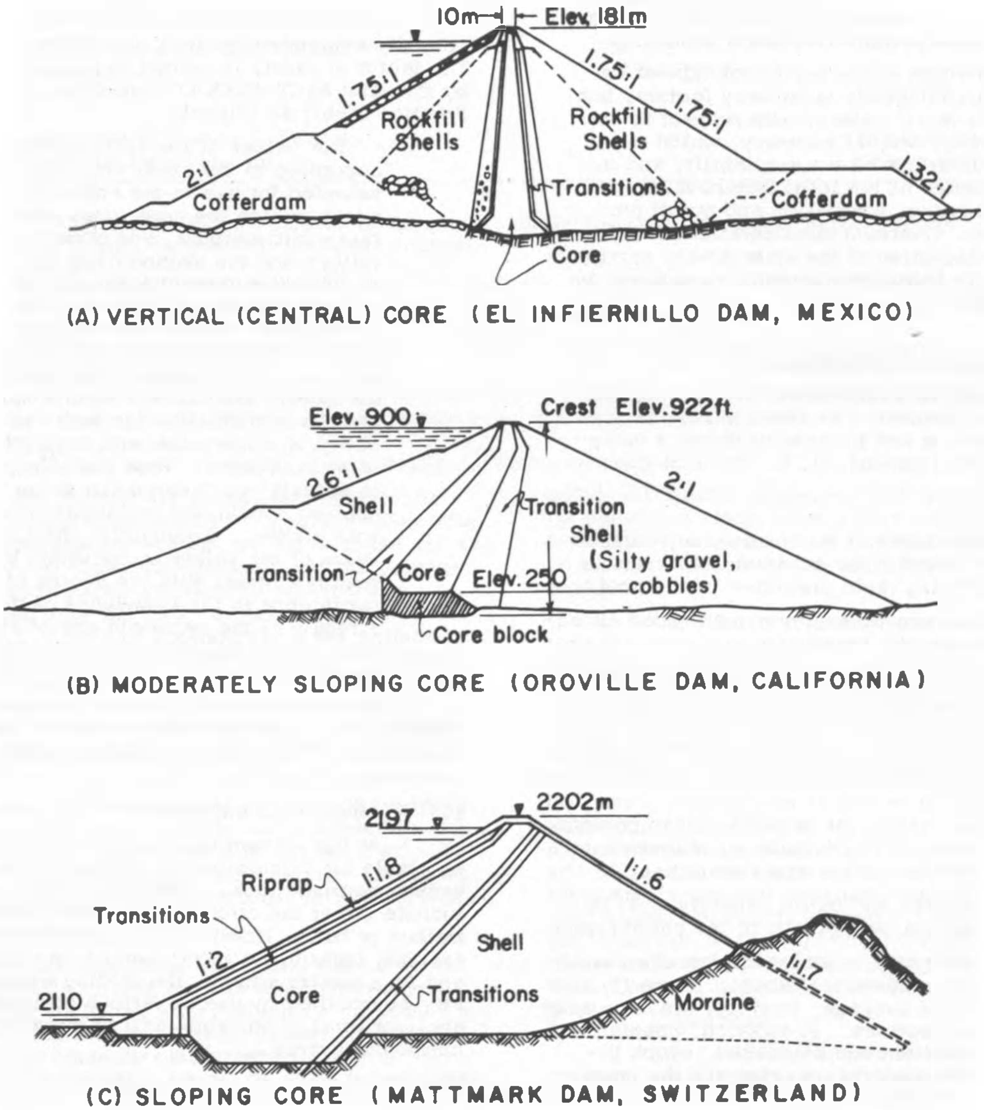


Fig. 13 Various Sections of Multi-Zoned Embankments

As observed by General Paper Committee of USCOLD (1964), a significant change has occurred in recent years in the location of the earth cores within the rockfill embankment section. The usual choice in the past was either a vertical core or a sloping core, with the former in the central portion of the dam and the latter laid on a dumped rockfill slope in the order of 1.35H : 1V. Recently, however, many of the cores have been located in an intermediate position, such that they can be called "moderately sloping." The terms "moderately sloping" can be applied to a core position when the downstream face of the core is on about a 0.5H : 1.0 V slope (Cooke, 1964). A moderately sloping core is often adopted after considering (1) the effect on the ease of construction, (2) the effect on stability and total volume, (3) the site topography, and (4) the possible development of tension cracks near the abutments. In Fig. 13, Diagram (A) shows a vertical core section (El Infiernillo Dam); Diagram (B) a moderately sloping core section (Oroville Dam); and Diagram (C) a sloping core section (Mattmark Dam).

The adoption of the particular type of embankment section depends upon many factors, but foremost would be (1) safety, with respect to stability and seepage and (2) economy, which would include considerations of the availability and the physical properties of the local materials, construction scheduling, foundation and cutoff problems and also, diversion considerations. In essence, the selection of the embankment section must satisfy the basic requirements as outlined in the next section.

4.2 Basic Design Requirements

The following criteria are set forth by the U. S. Corps of Engineers as basic design requirements that must be met in order to insure a satisfactory structure (Manual, U. S. Corps of Engineers, 1968):

(1) The slopes of the embankment must be stable under all conditions of construction and operation, including rapid drawdown of the pool.

(2) The embankment must not impose excessive stresses upon the foundation.

(3) Seepage flow through the embankment, foundation, and abutments must be controlled so that piping, sloughing, or removal of material by solution does not occur. In addition, the purpose of the project may impose a limitation on quantity of seepage flow.

(4) Freeboard must be sufficient to prevent overtopping by waves and include an allowance for settlement of the foundation and embankment.

(5) Spillway and outlet capacity must be sufficient to prevent overtopping of the embankment.

The approaches to an evaluation of embankment design with respect to Stability, items (1) and (2), and Control of Seepage, item (3), are discussed in the following sections. In addition, Special Design Considerations are discussed, which include the consideration of cracking and the phenomenon of load transfer.

4.3 Stability Analyses

4.3.1 General

Stability analyses provide a means of evaluating the margin of safety of different embankment sections under various loading and seepage conditions. In general, the analyses are made to evaluate the hazard of a shear failure in which a portion of an embankment or an embankment and foundation moves along a well-defined surface relative to the remainder of the mass. In addition to considerations of a failure by shear, possible failures as a result of excessive deformation or as a result of liquefaction must also be evaluated.

Lowe (1967) has capably summarized the state of the art with respect to stability analyses for earth and rockfill dams, and this paper will not go into the details of such analyses.

4.3.2 Factors of Safety

The results of the stability analyses are normally expressed in terms of a factor of safety. The factor of safety is placed in proper perspective by the Joint ASCE-USCOLD Committee on Current Practice (1967) as follows:

> "The values of the factor of safety differ according to the shear strength (Q,R,S) selected for use in the analysis, in the way in which the pore water pressures are taken into account, and other factors influencing the method used for computing it. Every method of analysis currently in use is based on highly significant simplifying assumptions which are frequently forgotten in the discussion and the use of the results. For the most part all current methods are semi-empirical and the justification for their use is based on experience with the performance of embankments. Thus the safety factor is actually an "experience factor" which enables a designer to compare one project with another. Frequently, the numerical value of the safety factor which a designer requires varies with the degree of his confidence in his knowledge of the shear strength of the materials and of site data, such as the influence of minor geologic details."

Minimum values of factors of safety in current usage correspond approximately to those suggested by the Corps of Engineers which are summarized in Table V.

4.3.3 Methods of Analysis

At the present time various procedures are available for calculating the factors of safety of embankment sections. Generally, the basic methods include either the circular arc or the sliding wedge method or both. However, in lieu of the circular arc, the logarithmic spiral method may be adopted and, in a similar manner, the sliding wedge method may be modified by use of various combinations of arcs, or lines of variable radii of curvature, or composite curves.

TABLE V

MINIMUM FACTORS OF SAFETY FOR EARTH AND ROCKFILL DAMS

Case No.	Design Condition	Minimum Factor of Safety	Shear Strength	Remarks
I	End of Construction	1.3 2)	Q or S 3	Upstream and down-stream slopes
II	Sudden drawdown from maximum pool	1.0 4)	R, S	Upstream slope only. Use composite envelope
III	Sudden drawdown from spillway crest	1.2 4)	R, S	Upstream slope only. Use composite envelope
IV	Partial pool with steady seepage	1.5	(R+S)÷ 2 for R < S S for R > S	Upstream slope only. Use intermediate envelope
V	Steady seepage with maximum storage pool	1.5	(R+S)÷ 2 for R<S S for R > S	Downstream slope only. Use intermediate envelope
VI	Earthquake (Cases I, IV & V with seismic loading)	1.0	-- 5	Upstream and down-stream slopes

Notes:

1) Not applicable to embankments on clay shale foundations; higher safety factors should be used for these conditions

2) For embankments over 50 feet high on relatively weak foundation use minimum factor of safety of 1.4

3) In zones where no excess pore water pressures are anticipated use S strength

4) The safety factor should not be less than 1.5 when drawdown rate and pore water pressures developed from flow nets are used in stability analyses

5) Use shear strength for case analyzed without earthquake

In the circular arc procedure, usually the method of slices is adopted in the calculations with the earth pressures acting on the sides of the slices included or, at times, not included. At the present time, the side forces are usually neglected in analyses made by the U.S. Bureau of Reclamation and the U.S. Corps of Engineers. If the surface of sliding is circular, the improvement in accuracy by considering the side forces acting on the slices is likely not to exceed 10 to 15 percent. On the other hand, if the surface of sliding is not circular the error may be significant (Terzaghi and Peck, 1967).

In any of the foregoing methods of analyses, the computations may be made using either the principle of total stress or the principle of effective stress. In most instances, the analyses are made, considering only a two-dimensional failure surface.

4.3.4 Conditions for Analysis

Various conditions of loading and/or states of seepage exist during the life of an embankment that must be evaluated with respect to the potential for a shear failure. These critical conditions or stages are normally classified as follows: (1) end-of-construction stage, or some intermediate stage; (2) steady seepage or full reservoir stage; and (3) drawdown stage, from either full or partial pool. In addition, the effects of earthquake ground motions on the performance of the embankment must be evaluated.

The shear strength parameters used to analyze the degree of stability of the embankment in each one of these stages are normally obtained from laboratory triaxial tests, conducted under various loading and drainage conditions as discussed in an earlier section. In connection with the selection of the appropriate shear strength parameters to be used in the analyses, consideration must be given to (a) possible variations in borrow materials, (b) natural water contents of borrow materials, (c) variations in placement rate and methods, (d) climatic conditions, and (e) inevitable variation in placement water contents and compacted densities that must be expected with normal construction control. In addition, some consideration must be given to the geologic details of the underlying foundation materials (Manual, U.S. Corps of Engineers, 1966).

4.3.5 Limitations

Most stability analyses involve limit procedures, which means that failure by sliding is fully developed along the entire failure surface. In reality, of course, slides involve some form of progressive failure, with consequent loosening of the structure in dense granular soil. Furthermore, the stress-distribution in earth and rockfill dams is not known, and certainly this distribution is affected by construction procedures and by deformations that develop during construction.

The mechanics of computing the stability of dams under various conditions of loading and with various zones of different materials have been greatly simplified and speeded up by the use of electronic computers. No method of calculation, however, can improve the reliability of the results unless the reliability of the assumptions is also improved. It is probably safe to generalize that laboratory testing lags behind the computations in this respect.

4.3.6 Evaluation of Seismic Effects

At the present time the overall stability of an embankment subjected to earthquake ground motions is analyzed in several different ways, which for the sake of the discussion, can be termed the conventional or the pseudo-static approach (Seed and Martin, 1966) or the dynamic response approach. The basic concepts in the latter were first proposed by Newmark (1963): In this method, the effects of earthquakes on the behavior of an embankment are evaluated on the basis of the deformations they produce rather than on a reduction in a numerical value of a factor of safety.

Pseudo-Static Approach

Most design agencies at the present time employ this type of analyses for an evaluation of embankment performance under a seismic occurrence In this approach, a minimum factor of safety against sliding along a well-defined failure is calculated using one or several of the methods of analyses outlined earlier, but in this instance, a static horizontal force of a certain magnitude is included in the analyses. The value of the horizontal force used in the calculations is expressed as the product of a seismic coefficient, k, and the weight, W, of the assumed sliding mass. Generally, if the computed factor of safety in this approach is equal to some predetermined value equal to or greater than 1, the embankment section is considered safe, although the acceptable value is largely based upon an evaluation of many factors, and ultimately, on the judgment of the engineer.

The choice of the proper value of the seismic coefficient, k, to be used in the computations, is a difficult one. The choice until recently was based more on convention than on theoretical considerations. Seed and Martin (1966) discussed the various methods that are available in order to arrive at a suitable value for the coefficient. These are essentially the use of (1) empirical rules, i.e., convention or past experience; (2) the consideration of the dam as a rigid body and, hence, the accelerations; and (3) the consideration of the embankment as an elastic medium with energy dissipation as a result of viscous damping Using the visco-elastic approach, Ambraseys (1960) developed several expressions for determining the magnitude of the seismic coefficient to be used in a pseudo-static analysis. Seed and Martin (1966) used a similar approach to analyze the time-history approach to analyze the time-history of dynamic forces acting on a potential sliding mass and, from this, calculated values of an average seismic coefficient at different levels.

The limitations of the pseudo-static approach were noted by Seed (1967). In the few cases of failure in which sufficient quantitative data were available for analyses, the pseudo-static approach failed to explain their occurrence. Of three major slope failures in Anchorage, sliding developed near the surface of a layer of soft, sensitive clay, whereas the analyses indicated that failure should have occurred at the base of the layer. Not only this, but the analyses could not explain why the failure did not develop at all until about two minutes after the start of the ground motions.

In addition, Seed (1967) demonstrated that the analytical details of the stability computations usually outweigh small variations in the values of the seismic coefficient. The following analytical details have a more predominant effect on the analyses in some cases than the choice of a seismic coefficient: (1) use of shear strength parameters either from triaxial or from plane strain tests, (2) use of similar parameters corresponding either to drained or undrained conditions, (3) the use of either inclined or horizontal forces between slices, (4) the use of the seismic force either at the centroid of each slice or at the base of each slice, and (5) the consideration that the seismic force affects both the overturning and resisting moments or only the overturning moment. All the variants to a solution, items (1) through (5) above, allow for at least 32 different possible analytical solutions. Computations during the preliminary design studies for Oroville Dam showed that varying the details of the analytical procedure in accordance with the above range of choices could easily change the value of the seismic coefficient from 0.10 to 0.23, retaining a constant value of the factor of safety of 1.1.

Hence, for the foregoing reasons and others, such as a complete disregard in the analyses of the influence of the time-history of stress applications on soil strength and pore water pressures Seed (1967), considers that the pseudo-static approach fails to provide a reasonable evaluation of the behavior of embankments during seismic events. As a result, their usefulness should be limited to ensuring a degree of conservatism in the selection of embankment slopes on a purely empirical basis.

As a result of the obvious limitations to the pseudo-static approach, other procedures have been developed within this decade which can be termed the dynamic response approach to evaluating the performance of an embankment subjected to transient and pulsating forces. In essence these new approaches relate the seismic event with embankment deformations.

Dynamic Response Approach

This approach has been pioneered by Newmark (1963) (1965). Essentially, it considers the embankment materials to be rigid-plastic in nature and for movements of the soil mass to occur along a well-defined surface. The method involves the determination of a yield acceleration, i.e., an acceleration at which sliding will begin and the computation of displacements that develop in the time intervals when this acceleration is exceeded. Goodman and Seed (1966) discuss the applicability of using the displacement analysis proposed by Newmark to calculate displacements in an embankment composed of dry, cohesionless soils. However, as noted by Seed (1967), for soils in which pore pressure changes develop as a result of the shear strains induced by the earthquake, determination of appropriate values of the yield acceleration becomes extremely difficult. Furthermore, for some types of soil, no well-defined yield acceleration exists and displacements take place over a wide range of accelerations.

In view of the foregoing an alternate approach has been suggested by Seed (1966). The alternate approach is based on (1) a determination of the stresses acting on soil elements within an embankment both before and during an earthquake, (2) subjecting typical soil samples in the laboratory to the same sequence of stress changes experienced by corresponding elements in the field and observing the resulting deformations, and (3) estimating the deformations of the slope from the observed deformations of the soil elements comprising it. The method thus gives consideration to (1) the time history of forces developed in the embankments or slopes during an earthquake, (2) the behavior of the soil under simulated earthquake loading conditions, and (3) the desirability of evaluating embankment deformations rather than a factor of safety. The procedure in reality has many limitations, particularly with regard to duplicating the effect of earthquake motion by means of tests on small specimens in the laboratory.

Seed (1967) discusses the use of the dynamic response approach in an evaluation of the behavior of Oroville Dam. A relatively concise discussion on earthquake considerations in earthdam design has also been presented by Chaturvedi and Sharma (1967).

It would appear that large scale field tests on dams, using shaking machines, such as those used on Bouquet Dam No. 1 in California (Hemborg and Keightley, 1964) combined with appropriate soil testing, would contribute toward an evaluation of the suitability of the analytical procedures presently proposed.

4.4 Seepage Control

4.4.1 General

Seepage control measures are required to protect a dam from any undesirable or dangerous effects of seepage, occurring through the dam itself or through the foundations or abutments. The measures adopted may or may not reduce seepage quantities, but foremost, the measures should minimize the risk of failure from instability of slopes, from foundation heave or from piping by erosion or by a combination of these.

For the purposes of discussion, current practices of the control of seepage through and underneath embankments are treated separately, but it is realized that effective treatment of seepage requires consideration of the embankment and its foundation as a unit.

Internally, most earth and rockfill embankments require some form of seepage control, either to improve stability or to control piping or both. In earth embankments, progressive zoning, horizontal drainage blankets, vertical (chimney) drains and toe drains provide seepage control in the downstream segment of the dam. A combination of inclined and horizontal filters is considered by some as the most positive solution for control of internal seepage (Bertram, 1967) (Cedergren, 1967). As an additional benefit, such a configuration of drains allows inclusion in the downstream shell of a wide variety of materials that otherwise might have been wasted. Mammoth Pool Dam, a zoned earth embankment with a crest located about 320 feet above the stream bed, was constructed on the San Joaquin River in California in 1958-1959. Fig. 14 shows that the dam contains a vertical chimney drain connected to a horizontal drainage blanket. Two layers comprise the chimney drain, a coarse downstream layer of crushed granite (tunnel muck) screened between 1/4 in. and 1-1/2 in. sieves and a finer layer composed of well-graded river sand. In addition to controlling the normal seepage through the dam, the drain system was designed to control concentrated leaks that might develop through transverse cracks in the relatively brittle impervious zone, composed of silty sand.

Various other dams with a similar approach to seepage control are shown in Figs. 15 and 16.

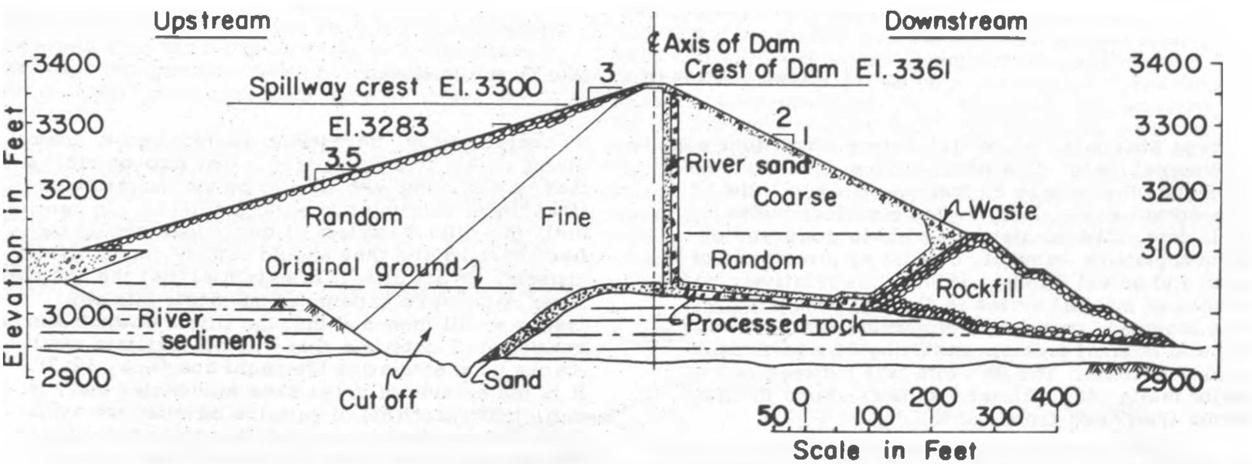


Fig. 14 Cross Section of Mammoth Pool Dam, California

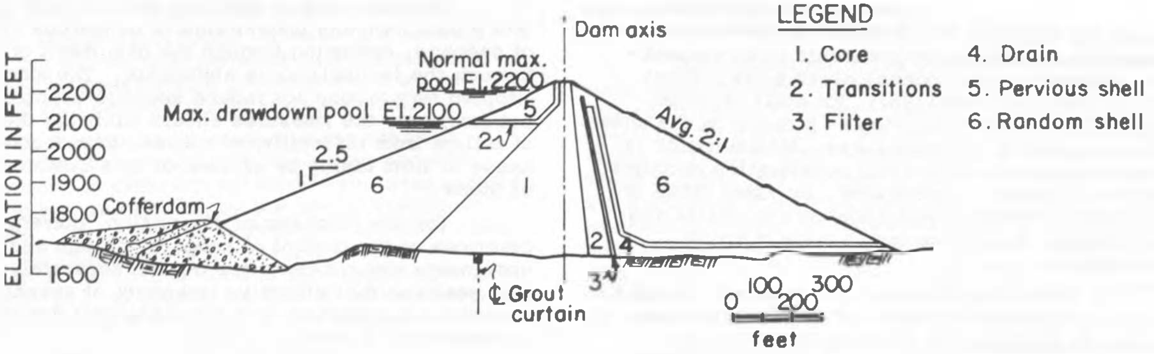


Fig. 15 Cross Section of Portage Mountain Dam (W.A.C. Bennett Dam), Canada

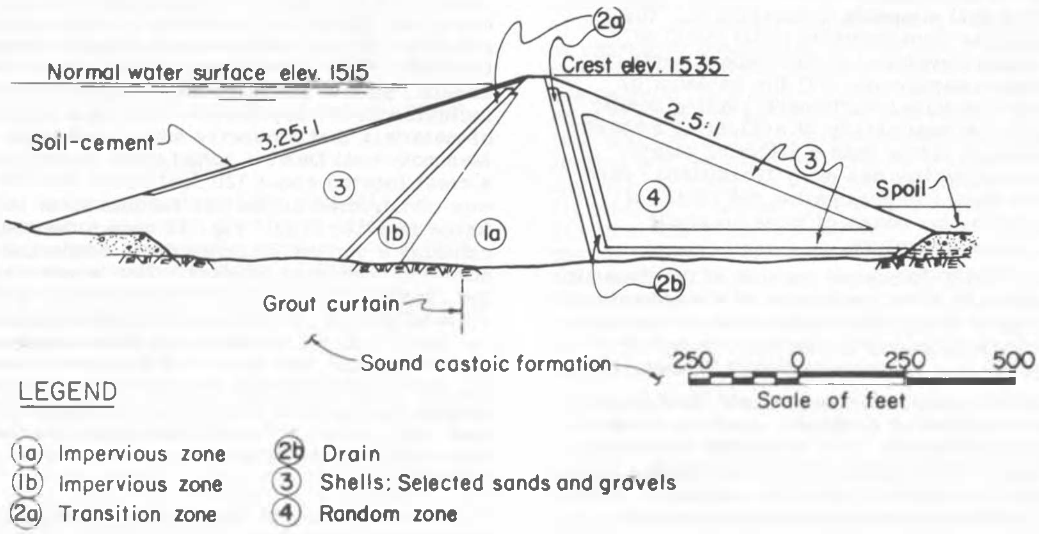


Fig. 16 Cross Section of Castaic Dam, California

Portage Mountain Dam contains wide transitions and an internal drain. The width of Zone 2 varies from 80 feet at the base to 33 feet near the top. The second transition, Zone 3, has a uniform width of 12 feet. The downstream shell is composed of random moraine material, consisting principally of sand and gravel (Ripley, 1967). The relatively wide system of internal drains in Castiac Dam (Fig. 16) was provided, in part, as protection against cracking and internal erosion, resulting from earthquake ground motions. The dam site lies between two major faults, San Gabriel and San Andreas in California (Perry and Kruse, 1968).

If the embankment section is primarily a rockfill section, composed of an impervious core bounded by coarse shells, transitions are required to control piping of the core materials, either through cracks that may develop with time or by the migration of fines under existing hydraulic gradients. In general, the filter materials or transitions should be free-draining and they should satisfy normal filter criteria. In addition it is essential that the upstream filter material be capable of migrating into any cracks to fill them and that the filters downstream be composed of materials that will not maintain open cracks upon saturation (Terzaghi and Peck, 1967). It is the opinion of these same authorities that, if sufficient quantities of suitable material are available, the smallest horizontal dimension that should be assigned to any layer be about 8 feet, equal to

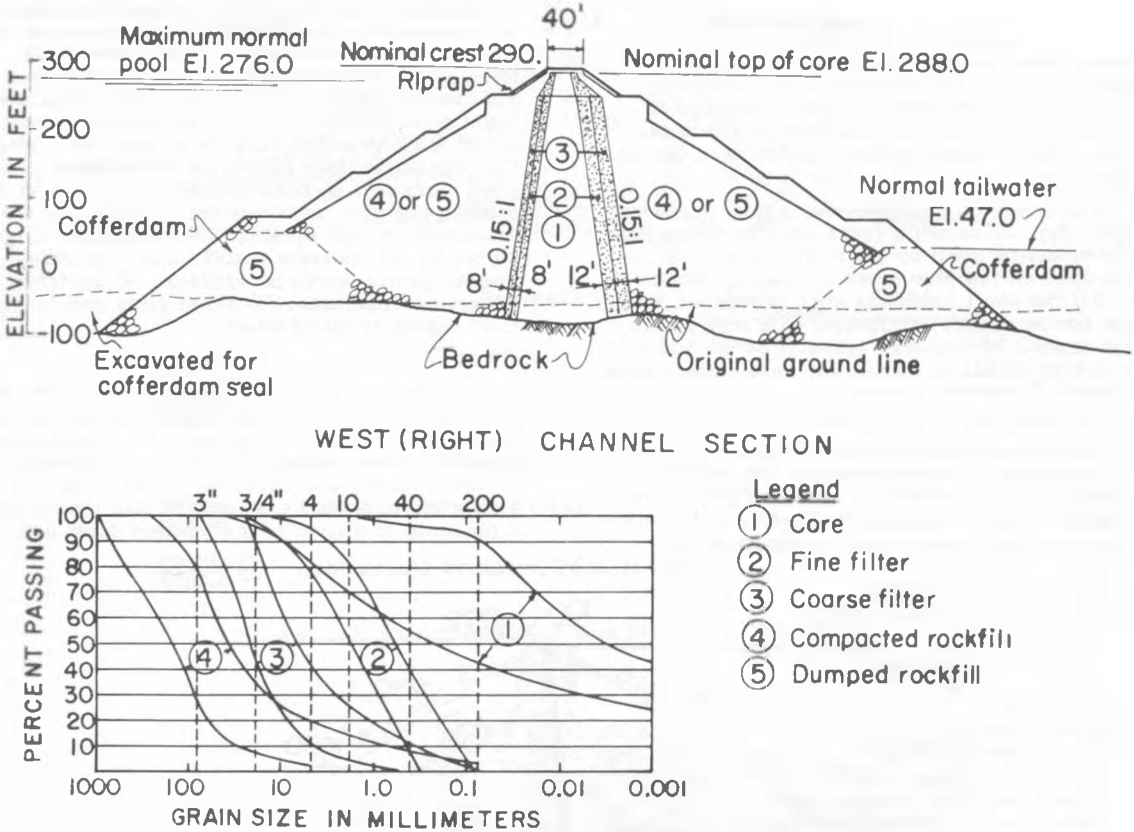


Fig. 17 Cross Section of Akosombo Dam, Ghana

the width of the hauling equipment, but that the downstream filter layers should be of considerably greater width to allow for a certain amount of segregation during placement. Because of the hazard of segregations, many engineers favor a relaxation of gradation requirements (with a consequent decrease in cost), but at the same time an increase in thickness.

Fig. 17 shows a cross-section of Akosombo Dam, a 370-foot high rockfill dam constructed on the Volta River, Ghana, Africa in 1963-1964. The dam contains a central clay core composed of CL materials (Unified Soil Classification System), compacted slightly dry of optimum in the lower portion and wet of optimum in the upper zones. The gradation curves shown were derived from tests made during the construction of the dam. As shown, the transition zones, both upstream and downstream, consist of two layers, with the inner layers composed of well-graded river sand and the outer layers composed of a well-graded crusher-run material. The cross section of Nurek Dam, Russia, is shown in Fig. 18. As shown, the embankment contains a vertical core, bounded both upstream and downstream by transitions. The outer slopes of the upstream and downstream shells are in the order of 1:2.25 and 1:2.2, respectively. This dam, now under construction (Engineering News-Record, 1968) will be the highest rockfill dam in the world when completed.

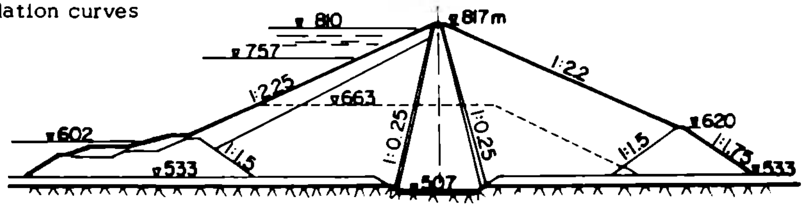


Fig. 18 Cross Section of Nurek Dam, Russia

In some instances relatively broad transitions of well graded sand and gravel material with cobbles afford ample protection against any undesirable effects of seepage and may be cheaper than multiple zones of tightly graded materials. Fig. 19 shows a cross-section of Gepatsch Dam, a 153m high rockfill dam completed in 1964 in the Kauner Valley in Austria. The broad transitions project a core composed of moraine and talus material, modified in a relatively narrow upstream segment with an additive of bentonite (Schober, 1967).

A dam may be progressively zoned to control seepage. Fig. 20 shows a cross section of the Blue River Dam, constructed by the U.S. Corps of Engineers in 1968 on the Blue River in Oregon (McCoy, 1968). All the shell materials were processed from a single borrow source, composed of a very heterogeneous deposit of cobbles, gravels, sands and silts. Such a design yields an absolutely safe embankment with respect to internal seepage.

4.4.2 Foundations and Abutments

Various methods for controlling seepage through pervious foundations exist: the measures best suited for any particular project depend upon many factors, but in general, the safety of the embankment must be insured and, in addition, the type of treatment must be justified on the basis of economic considerations. In many instances, consideration of all project requirements leads to the adoption of not one, but several types of seepage control measures. A positive cutoff, formed in an open excavation to an impervious stratum and which is backfilled with compacted impervious material is the most desirable form of cutoff (see Mammoth Pool Dam, Fig. 14). When this cannot be done from a practical standpoint, other measures must be considered. Some of the ones in common usage at the present time are discussed in this section: these include (1) grout curtains, (2) concrete cutoff walls, (3) slurry trench cutoffs (earth backfilled), (4) upstream impervious blankets, (5) sheet piles and (6) vertical drains or relief wells.

4.4.3 Grouting

The most satisfactory method for grouting deep deposits of alluvium at the present time is the tube a' manchettes or the sleeve pipe method of grouting, a process of injection invented by E. Ischy (Ischy and Glossop, 1962). Fig. 21 shows the essential elements of this procedure. A borehole is drilled to the desired depth and into

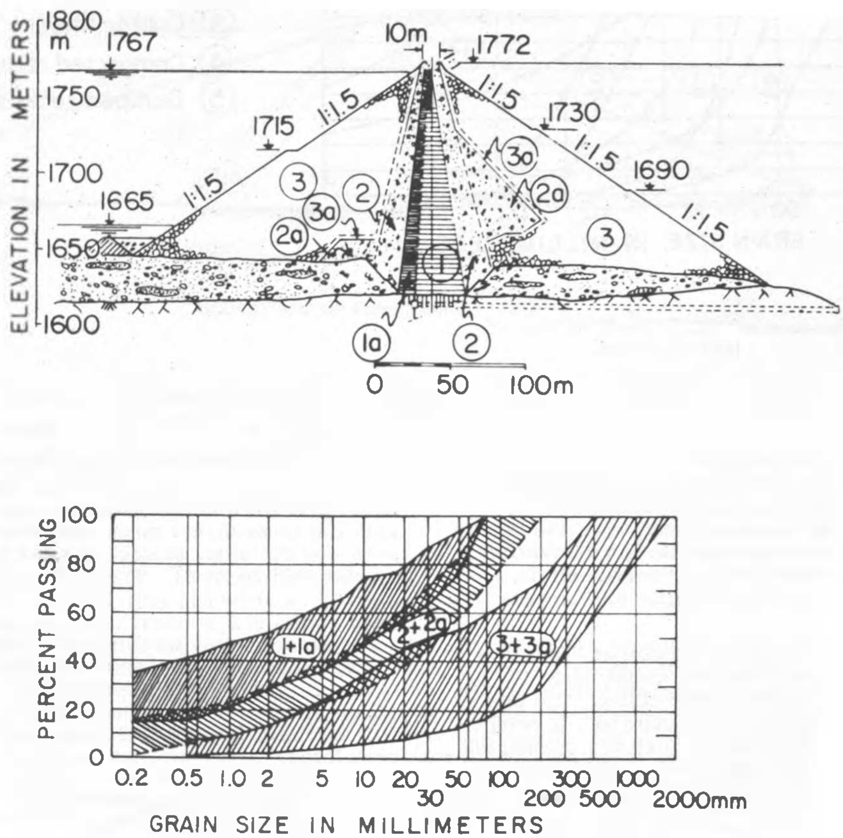


Fig. 19 Cross Section of Gepatsch Dam, Austria

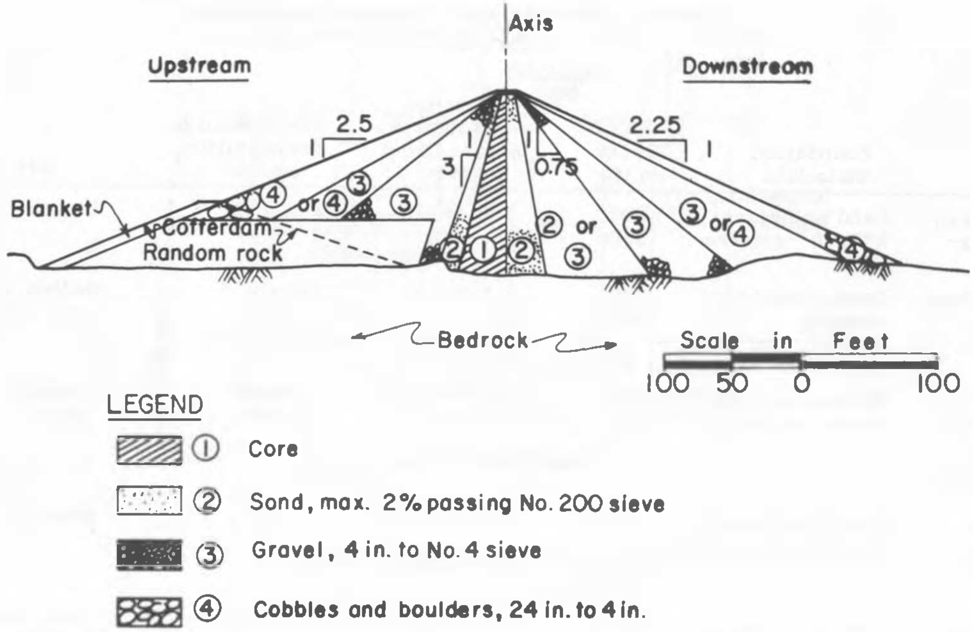


Fig. 20 Cross Section of Blue River Dam, Oregon

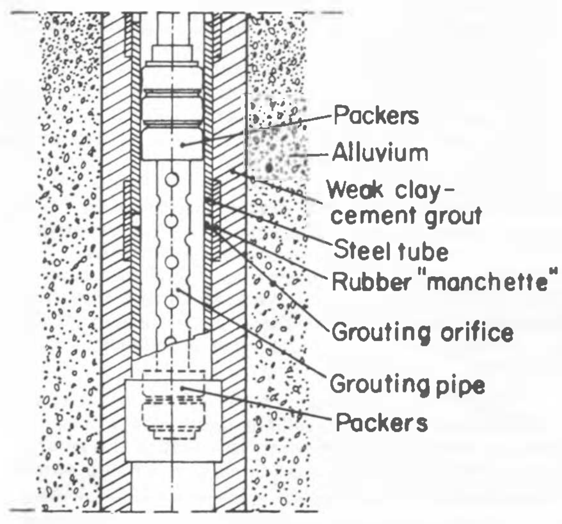


Fig. 21 Essential Details of the Tube A' Manchettes Grouting Pipe

the borehole a "manchette" is inserted, which consists of a steel tube perforated with a ring of small holes at about 1 foot intervals. Each ring of perforations is enclosed by a rubber sleeve that fits tightly around the tube. The annular space between the steel tube and the material to be grouted is filled with a weak clay-cement grout. An injection pipe, fitted with packers, is lowered down into the steel tube until it is opposite one of the rubber sleeves. Pressure applied to the grout opens the valve formed by the rubber sleeve and allows the grout to escape through the holes and into the ground.

The principal advantages of this procedure are that the same tube can be re-entered as often as necessary to grout with different types of mixtures and to perform additional grouting, if necessary, even after a considerable lapse of time. As the procedure is expensive as compared with some other alternate methods of seepage control it is only adopted after considerable deliberation of project requirements.

Table VI shows some of the major projects wherein a grout cutoff was formed through deep deposits of alluvium, consisting of a wide variety of materials, from fine to medium sands on up to coarse sands and gravels with cobbles and boulders. The first use of the tube a' manchettes procedure on a large scale was at the Serre Poncon Dam on the Durance River in France. The cut-off, formed through alluvium some 100 meters deep, was preceeded by an extensive program of field tests. Since that time, the effectiveness of the procedure has

Table VI Effectiveness of Grouting In Alluvium At Some Major Dams

Project (Year)	Foundation Materials	Max. Depth m (ft)	Hydraulic Gradient Across Grout Curtain	Post-Grouting Permeability, cm/sec	Ref.
Sylvenstein Dam, Isar River, Germany (1958)	Sand and gravel $k=5\times10^{-1}$cm/sec	100 (330)	2 to 7	$k=1.3\times10^{-4}$	Lorenz, 1967
Serre Poncon Dam, Durance River, France (1959)	Sand, gravel and cobbles $k=3\times10^{-1}$cm/sec to 9×10^{-2}cm/sec	100 (360)	3.5 to 8	$k=2\times10^{-5}$	Haffen, 1963; Wolf, 1967
Terzaghi Dam (Mission Dam), Bridge River, British Columbia, Canada, (1960)	Sand, gravel and some boulders	153 (500)	3 to 4	Assumed: E = 90%*	Terzaghi, 1964, 1968
Notre-Dam de Commiers Dam, Drac River, France (1963)	Sand and gravel $k=10$ to 3×10^{-2} cm/sec	50	2.5 to 6.5	---	Bonazzi, 1965
Mattmark Dam, Visp River, Switzerland (1967)	Sand and gravel with cobbles $k=10^{-1}$ to 10^{-3} cm/sec	100 (300)	3 to 7	$k-2\times10^{-5}$	Gilg, 1961; Eng. News-Record, Nov. 26, 1964; Fruhauf, 1965
Mangla Dam (Closure Dam) Jhelum River, West Pakistan (1967)	Gravel and cobbles with sand $k=4\times10^{-1}$cm/sec	23 (75)	3 to 4	$k=5\times10^{-5}$cm/sec	Skempton, 1963; Binnie, 1967
Aswan Dam, Nile River, Egypt	Fine to coarse sands $k=1\times10^{-1}$ to 5×10^{-3}cm/sec	255 (835)	2 to	$k=3\times10^{-4}$cm/sec	Wafa, 1967

* $E = \frac{h'}{H}$ where h' = head loss across the curtain
H = difference in elevation between the reservoir at maximum pool and the tailwater level.

been demonstrated on other projects. Of particular interest, the field tests at Serre Poncon, Mangla and Aswan Dams have led to the conclusion that regardless of the initial permeability of the foundation materials, the post-grouting permeability is not likely to exceed 10^{-4} to 10^{-5} cm/sec. This is evident from a study of the permeability data in Table VI.

Of interest too, the table reveals that at about the contact of the impervious core with the grout curtain, hydraulic gradients are in the order of from 2 to 4. In some of the European dams, such as Sylvenstein, Serre Poncon and others, greater hydraulic gradients exist with depth, reaching a maximum of from 7 to 8. At the high Aswan and at Terzaghi Dam, the width of these deep grout curtains was maintained to provide a hydraulic gradient of not more than 4 at the base of the curtain.

4.4.4 Concrete Cutoff Wall

The concrete cutoff wall and the grouted cutoff each are not subject to visual inspection during construction, therefore they each require special knowledge, equipment and skilled workmen to achieve a satisfactory end project. Because of the specialization that is required only a very few firms are engaged in the construction of this type of wall for seepage control. Among others the firm of Impressa Costruizioni Opere Specializzate (ICOS) and Rodio-Soletanche are the most prominent in the field.

Fig. 22 shows the general steps in construction of the wall: the equipment used varies from project to project and between firms, but the overall sequence of construction is very much the same as that shown in the Figure.

The heavy drilling and excavating equipment works on rails laid down alongside a guide trench, bounded near the ground surface by opposing shallow guide walls of concrete. The distance between the guide walls normally varies from 1.5 to 2.5 feet, the range of thickness of most existing cutoff walls to date. At predetermined intervals along the trench percussion rigs with special chisels drill holes down

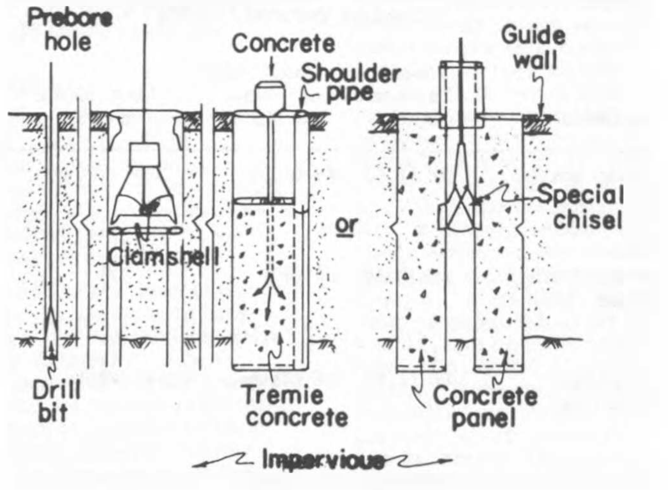


Fig. 22 Steps in Construction of Concrete Cutoff Wall

to an impervious stratum, in most instances bedrock, to serve as a guide for subsequent excavations of a wall section by a clam shell rig. After a clam shell removes the material between the holes and after clean-up of the trench bottom is done, tremie concrete is poured to form a wall panel. The concrete displaces the bentonite slurry that keeps the trench open during excavation. A critical feature in the formation of a tight wall is the joint between adjacent wall panels. A shoulder pipe, inserted at the end of the trench is one method forming a joint between walls. Diagram (C), Fig. 22, shows a shoulder pipe in place during the pouring of the concrete into the trench. After a certain set is achieved in the concrete, the pipe is withdrawn leaving a convex surface that permits keying of one end of the wall section with another. Alternatively, a special expanding chisel may be used to clean and form the ends of two opposing concrete panels so that a tight joint is formed when concrete is poured between them, see Diagram (D).

As the tremie concrete is poured, the concrete at the interface with the slurry tends to partially solidify and become contaminated with the slurry, leading to a weak, low-density concrete in the upper zone of the cut-off wall. At La Villita Dam in Mexico this zone varied from about 5 to 15 feet in depth and was subsequently removed and replaced with cast-in-place concrete. Such removal and replacement may not always be feasible; in such cases good techniques should be employed in pouring the tremie concrete and it appears advisable to insist on inspection of this upper zone by core borings or other procedures.

Table VII shows some of the major projects wherein the concrete cutoff wall has been employed for the control of underseepage. As shown, the usage has varied from providing a cutoff underneath cofferdams, such as at Manicougan 2 and 5, (Canada) to forming a permanent seepage barrier underneath main embankments, such as at La Villita (Mexico) and Kinzua dams (USA). At

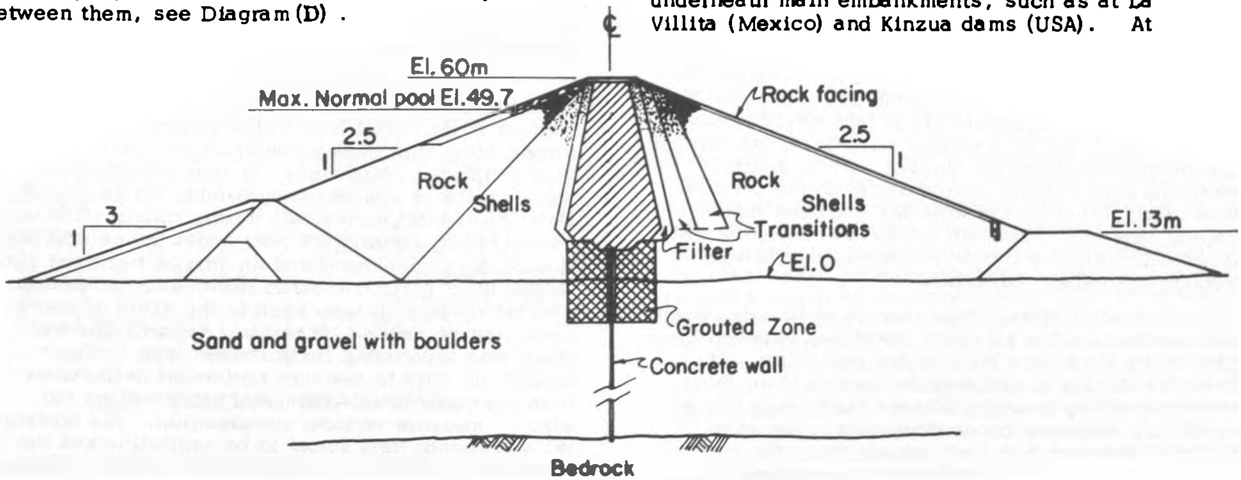


Fig. 23 Cross Section of La Villita Dam, Mexico

Table VII Current Practice: Thin Concrete Membranes Used as Vertical Cutoffs

Project	Foundation	Wall Thickness cm (ft)	Max. Depth of Wall m (ft)	Max. Head m (ft)	Remarks	References
Sesquile Dam, Bogota River, Columbia, South America (1963)	Sand and gravel	55 (1.8)	76 (255)	30+ (100+)	ICOS	Eng. News-Record, Mar. 14, 1963
Cofferdam-Manicouagan 2 Dams, Quebec, Canada (1963)	Alluvium, with boulders	76 (2.5)	27 (90)	(?)	ICOS	MacDonald, 1965; Conlon, 1967
Cofferdam-Manicouagan 5 Dam, Quebec, Canada (1964)	Sand, gravel and boulders	60 (2.0)	76 (250)	70 (230)	ICOS	Baribeau, 1967
Kinzua Dam (Allegheny Reservoir Dam), Pennsylvania, U.S.A. (1965)	Silts, sands and gravels	76 (2.5)	55 (180)	38 (125)	ICOS f'_c=3840 lb/sq. in. Cost: $20.00/sq.ft.	Fuquay, 1967; Monaghan, 1964
Peneos Dam, Peneos River, Greece (1965)	Sand and gravel	60 (2)	18 (60)	49 (160)	Rodio & Co. Reinforced	Gofas, 1965
Cofferdam-Arrow Dam, Columbia River, British Columbia, Canada (1967)	Sands, gravels, cobbles and boulders	76 (2.5)	50 (165)	35 (115)	ICOS[a] f'_c=4000 lb/sq. in. Cost $21.00/sq.ft.	Gadsby, 1968
La Villita Dam, Balsas River, Mexico (1968)	Sand and gravel w/cobbles	50 (1.7)	88 (290)	42 (140)	ICOS	Unpublished

a. Process of Impressa Costruzioni Opere Specializzate, Milan, Italy

La Villita Dam the wall was formed through a dense deposit of sand and gravel with boulders to a maximum depth of 290 feet. This wall (Fig. 23) was centrally located whereas the wall at Kinzua Dam was constructed outside of the limits of the embankment, being tied to the central earth core of the dam by an impervious upstream blanket. The reservoir in the back of Kinzua Dam was raised to its maximum level in early 1967 and according to Fuquay (1968) the head loss across the wall is that which would be expected if all the seepage were occurring through the upstream impervious blanket. At La Villita, maximum pool has not been reached at the present time (early 1969) so the wall has not been fully tested; however, observations to date indicate that it has successfully formed an impervious barrier through the valley alluvium.

At some sites, two concrete walls have been constructed and the alluvium contained between them grouted by the tube a'manchettes procedure. At Shek Pik Dam, a zoned earthfill dam in Hong Kong, two concrete walls about 20 feet apart were formed by 22 inch diameter concentric piles. The sand alluvium between and just outside the walls was grouted, reducing the coefficient of permeability, k, of the alluvium from 5×10^{-3} cm/sec to 4×10^{-5} cm/sec (Carlyle, 1965). A somewhat similar approach was used to form a cutoff underneath Obra Dam, an earth dam in India founded on alluvium composed of medium to coarse sands; maximum thickness in the order of 25 meters (Prakash and Assawal, 1967).

The possibility of rupture of the thin, vertical concrete cutoff wall cannot be ignored. Rupture could result from either upstream or downstream movement during construction, from earthquake-induced deflections, or from downdrag causing buckling or compressive failure. At La Villita Dam, horizontal movements during construction were minimized by placing the wall under the central portion of the embankment and an adjacent grouted zone in the upper portion assured reasonable protection against seepage losses even in the event of complete rupture caused by seismic action. The wall itself was thoroughly instrumented with inclinometers not only to measure horizontal deflections both upstream-downstream and cross-valley but also measure vertical compression. The horizontal movements were found to be negligible and the settlement of the top of the wall was about 6 inches or 0.2 per cent of its height. This compression was uniformly distributed from top to bottom with no discontinuities.

4.4.5 Slurry Trench Cutoff Walls

Slurry trench cutoffs were pioneered in the United States in the late 1940's with the first usage to control foundation seepage at Terminal Island in Long Beach, California (Leps, 1968). Since that time they have gained increasing acceptance as an economical means of controlling seepage underneath cofferdams and main embankments.

Fig. 24 shows the general steps in construction of the slurry trench cutoff. A backhoe or dragline excavates a trench through the pervious deposits down to suitability impervious materials. A bentonite slurry, retained in the trench above the existing groundwater level, prevents the trench walls from caving. After a sufficient length of trench has been excavated and the bottom suitably prepared, backfilling begins. The physical characteristics of the backfill are specially controlled; in general, the backfill should be well-graded, impermeable in-place, and sufficiently coarse to minimize post-construction settlements. A selected amount of bentonite slurry may be blended with the backfill to improve its properties. A general gradation band for the backfill by Jones (1967) is shown in Fig. 25.

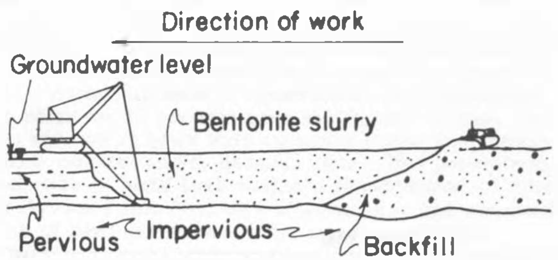


Fig. 24 Construction of Slurry Trench Cutoff

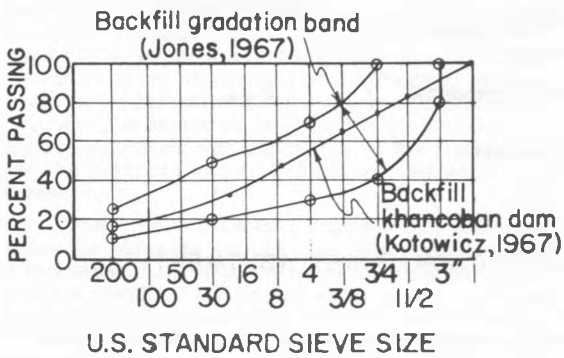


Fig. 25 Gradations of Backfill for Slurry Trench Cutoff

Jones (1967) tabulated various projects in which slurry trench cutoffs have been adopted. Their usage ranges from control of foundation seepage underneath dikes and levees to dams. The slurry trench cutoff trench at Wanapum Dam in 1958 was the first major installation in the United States designed to function on a long term basis for seepage control (Sherard et al, 1963) (Engstrom, 1963). The width of the cutoff at this project was in the order of 10 feet and the maximum depth was about 80 feet, although grouting through the bottom of the trench extended the cutoff to greater depths. The maximum hydraulic gradient across the cutoff is about 9. In general, the hydraulic gradients at other projects tabulated by Jones (1967) range from 7 to 12, but at Comanche Dam, Dike 2, Mokelumne River, California a hydraulic gradient of about 17 exists. However, the foundation materials are generally fine-grained and overlie well-graded gravels, which minimizes the hazards of piping. In a laboratory test of the Wanapum backfill materials piping occurred at a hydraulic gradient of about 35 (Jones, 1967). In this instance, piping occurred into some openwork gravels.

Kotowicz (1967) presents some interesting settlement data for a relatively shallow slurry trench cutoff, constructed at Khancoban Dam, just upstream of the axis of the main embankment. The dam is a homogeneous earth dam about 50 feet high, constructed on the Swampy Plain River in Australia. The trench is about 6 feet wide and it extends to a maximum depth of 15 feet. The gradation characteristics of the backfill used in the cutoff are shown in Fig. 25. The gradation curve shows that the material is well-graded, that the maximum size is about 6 inches, and that the percent passing the No. 200 sieve is in the order of 15 percent. Settlement devices were placed in the embankment during construction. They revealed that the surface of the trench settled about 3.5 percent of the height of the trench or in this instance, about 5 inches under about 60 feet of overburden. The settlement data indicated that there was no tendency for a separation to form between the cutoff and the overlying compacted materials, a conclusion also substantiated by piezometric data.

At Tortoles Dam on the Nazas River in Mexico, the Secretario de Recursos Hidraulicos used a partially-penetrating slurry trench about 23 m deep to prevent leakage through an extremely pervious layer of sand and gravels with boulders, Fig. 26. The bottom of the trench connected to a thick stratum of very dense and relatively impervious silty sands and gravels. The dam, about 34 m in height above the riverbed, was completed in September 1968 and the reservoir was immediately filled. Subsequent piezometric observations indicate that water levels downstream from the cutoff are reduced to acceptable levels, as well as the total leakage. Instruments were installed prior to construction of the embankment to measure the settlement of the clay core above the slurry trench. Fig. 27 shows both the transverse settlements under the core and downstream shell at maximum sections and the longitudinal settlements of the core near the right abutment. The settlement of the slurry trench backfill is seen to be of the order of 15 to 20 cm. This differential settlement is easily withstood without cracking by the highly plastic core which was compacted at a water content well above optimum.

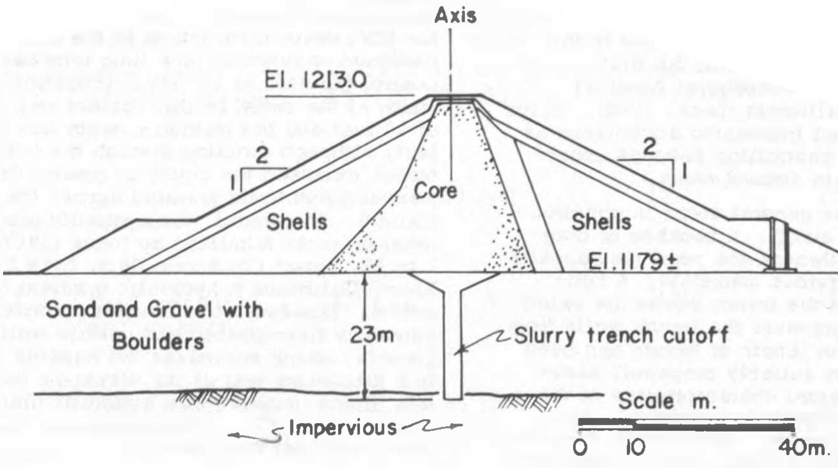


Fig. 26 Cross Section of Tortolos Dam, Mexico

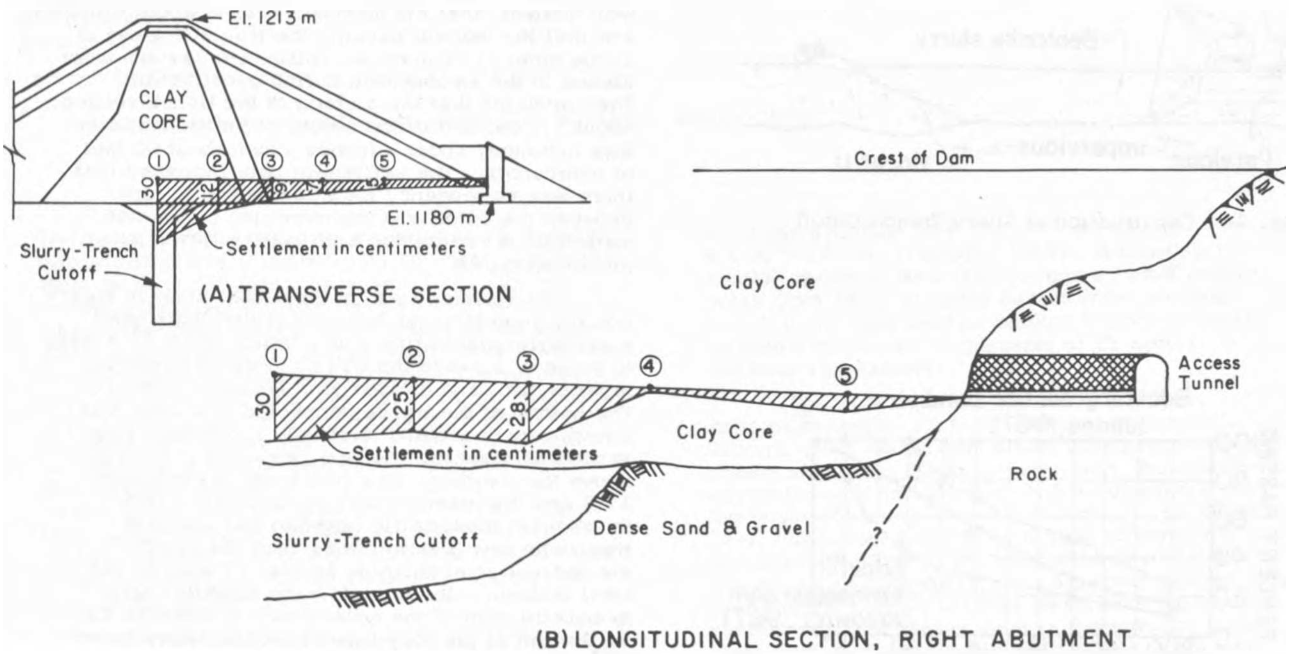


Fig. 27 Settlement of Slurry Trench Cutoff, Tortolos Dam

4.4.6 Upstream Impervious Blankets

If a positive cutoff is not required, or is too costly, an upstream impervious blanket combined with vertical drain wells in the downstream section is often used to advantage. An upstream blanket may result in major project economies, particularly if the only alternates consist of deep grout curtains or concrete cutoff walls. Since alluvial deposits in river valleys are often overlain by a surface layer of relatively impervious soils, it is advantageous if this natural impervious blanket can be incorporated into the overall scheme of seepage control (Bennett, 1946).

A comprehensive treatise on the design and effectiveness of upstream blanket-relief well systems to control seepage beneath dams and levees underlain by pervious sands was presented in two companion papers by Turnbull and Mansur (1961).

Upstream blankets, combined with downstream drainage, have been employed for some years, and observational data reflecting on their effectiveness, have been discussed in the literature. Lane and Wohlt (1961) discussed the performance of Ft. Randall and Gavins Point Dam, each of which was constructed in the 1950's on the Missouri River. The dams were provided with an impervious upstream blanket and a system of downstream relief wells. Ft. Randall Dam was underlain by alluvium, consisting primarily of fine to coarse sand with some gravel. The maximum depth of the alluvium was in the order of 170 feet. Periods of observation cover from 5 to 7 years. The observations revealed that the head loss under Gavins Point Dam, within five years after commencement of operations, had increased as much as 40 percent over the initial values: no net change in head loss occurred under the impervious blanket in Ft. Randall Dam. The persistent increase in head loss within a short period of time at Gavins Point Dam was attributed to the accumulation of sedimentation in the reservoir bottom.

Additional performance data on the effectiveness of upstream impervious blankets was reported by Brown (1961). He discussed the impervious blankets and drainage measures adopted to control seepage through the abutments of two dams on the Columbia River - Chief Joseph and McNary Dams. In both cases, the abutments consisted of coarse alluvium with design coefficients of permeability in the order of 0.5 cm/sec. The blankets extended upstream from the axis from 305 m to 610 m. In both instances, seepage quantities, measured in downstream collector drains and tunnels, diminished by as much as 50 to 60 percent of their initial values, all within 5 years after commencement of operations. As before, the increase in effectiveness of the blankets with time was attributed to the tightening up of the blankets and additional sedimentation in the reservoir.

These case histories suggest that feasibility studies for blankets should take into consideration the increase in the effectiveness of blankets with time as a result of additional sedimentation in the reservoir. Of course, this would be particularly important for dams across rivers which carry a relatively large amount of sediment.

More recently, Peterson (1968) commented on the performance of the South Saskatchewan River Dam in Canada, which was provided with an upstream blanket. This earth dam was founded on as much as 100 feet of fine to medium sand. It was filled in July 1967 to a water depth of 170 feet as compared to the design depth of 190 feet. Observations revealed that the performance of the dam was satisfactory with no excess seepage pressure in the downstream area (downstream filter and relief wells were provided) and a total measured seepage flow of 1500 gal. per min.

Some other dams in Canada, which contain upstream blankets to control underseepage through relatively deep deposits of alluvium, are Arrow Dam (Golder and Bennett, 1967) and Seymour Falls Dam (Ripley and Campbell, 1964). In the former dam, the upstream blanket is planned to be constructed by dumping glacial till through water.

4.4.7 Steel Sheet Piles

Steel sheet piles have proven to be rather ineffective as a positive means of controlling seepage through pervious deposits. Lane and Wohlt (1961) discussed the results of observations made on three earth dams along the Missouri River, each one constructed on extensive deposits of alluvial sands and gravels, reaching depths as much as 170 feet. In each instance, sheet piles were driven through the pervious strata to underlying bedrock to form a cutoff. The piezometer observations revealed that the effectiveness of the sheet piling was initially low: the head loss across the piling varying from 10 to 20 percent of the total head. Within 5 to 20 years time, however, the head loss across the piling increased to as much as 20 to 40 percent of the total head: this increase in effectiveness is attributed to the migration of fines and corrosion in the interlocks. According to these observations and other studies (Jaspar and Ringheim, 1953), it appears difficult to justify the use of sheet piling as a means of controlling seepage, particularly when there are probably other less expensive means at the present time that provide the same if not more positive results. Some methods may be perfected to improve the operating characteristics of sheet pile cutoffs, such as using vibrating pile driving hammers to reduce the probability of driving out of interlock and the use of bentonitic mud to seal the interlocks (Sherard, 1968); however, until such time, sheet pile walls should be considered no more effective than partial cutoffs.

An interesting modification of the basic sheet pile cutoff scheme was adopted at Swift Dam No. 1, a zoned earth dam constructed on the Lewis River, Washington in 1957 (De Luccia, 1958). Fig. 28 shows the basic elements of the sheet pile wall: an H-pile section with interlocks at each corner formed by welding 15-inch standard straight sheet pile sections onto each flange of an 18-inch wide flange beam. The design provided considerable stiffness and strength and the adjacent pile sections formed cells having clear openings of about 14 to 16 inches. After driving, the cells were cleaned out and filled with tremie concrete to further strengthen the wall and make it watertight. On top of this wall, a reinforced concrete cutoff wall was formed to form a waterstop in the impervious zone. The sheet pile wall was constructed to a maximum depth of 85 feet.

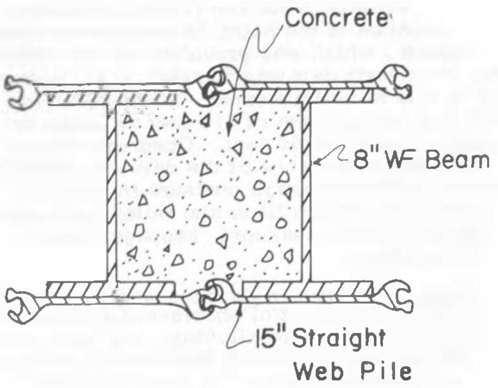


Fig. 28 Details of the Sheet-Pile Wall Installed at Swift Dam No. 1, Washington

4.4.8 Relief Wells

Relief wells are an important adjunct to most of the preceeding basic schemes for seepage control. They are used not only in nearly all cases with upstream impervious blankets, but also along with other schemes, to provide additional assurance that excess hydrostatic pressures do not develop in the downstream portion of the dam, which could lead to piping. They also reduce the quantity of uncontrolled seepage flowing downstream of the dam and, hence, they control to some extent the occurrence and/or discharge of springs. Casagrande (1961) pointed out the importance of extending the relief wells deep enough into the foundation so that the effects of minor geologic details on performance are minimized. Some design features of relief wells are presented by Sherard et al (1963) and Turnbull and Mansur (1961). The importance of continual observation and maintenance of relief wells, if they are essential to the overall system of seepage control, was brought out by Lacy and Van Schoick (1967).

Relief well systems are often tied into drainage galleries which extend underneath embankments and into abutments. Fig. 29 shows the system of drain wells and galleries constructed at El Infiernillo Dam, a rockfill dam constructed on the Balsas River in Mexico. Galleries, constructed for drainage as well as other purposes, have been located in the rock underneath embankments, in transition zones (Galloway, 1967) and in the cores of dams. Fig. 30 shows a cross section of Aswan Dam, with three galleries in the core and other seepage control measures, including an upstream blanket, a grout curtain and downstream relief wells.

The effects of uncontrolled seepage became dramatically evident at Fontenelle Dam, a 130-foot high earth dam constructed in 1961-64 on the Green River in Wyoming. Seepage through a system of relief joints in the shales and sandstones of the right abutment caused severe erosion and sloughing of embankment materials during reservoir-filling which, without prompt action, could have resulted in failure of the dam. Seepage control measures in the right abutment consisted primarily of a single-line grout curtain. As part of the remedial work, this curtain was extended and broadened by additional lines of grout holes. In addition, all of the damaged embankment was removed and replaced (Bellport, 1967).

4.5 Special Design Considerations

4.5.1 Cracking

Cracking in impervious zones and the associated danger of a failure by erosion or even breaching have been recognized for many years as hazards

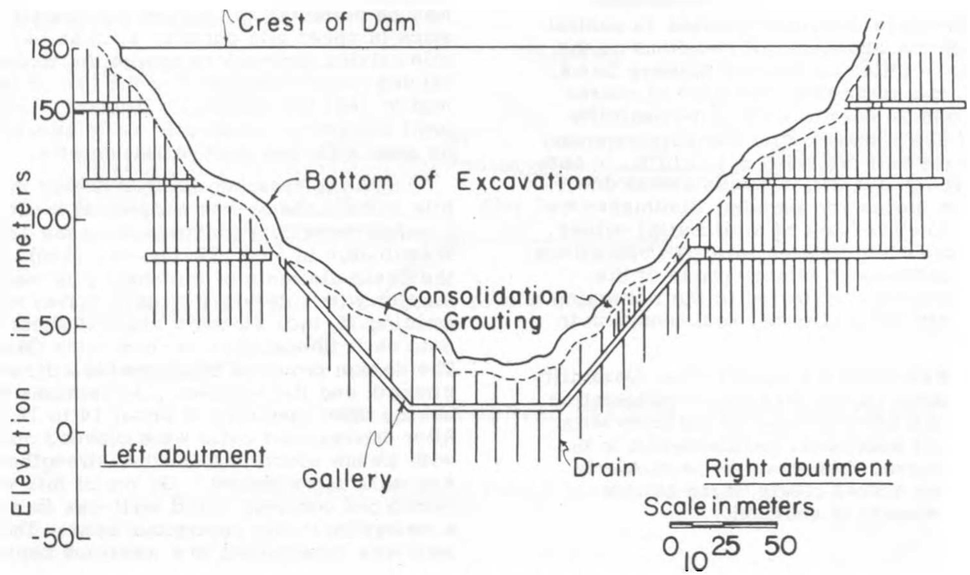


Fig. 29 System of Drainage Galleries at El Infiernillo Dam, Mexico

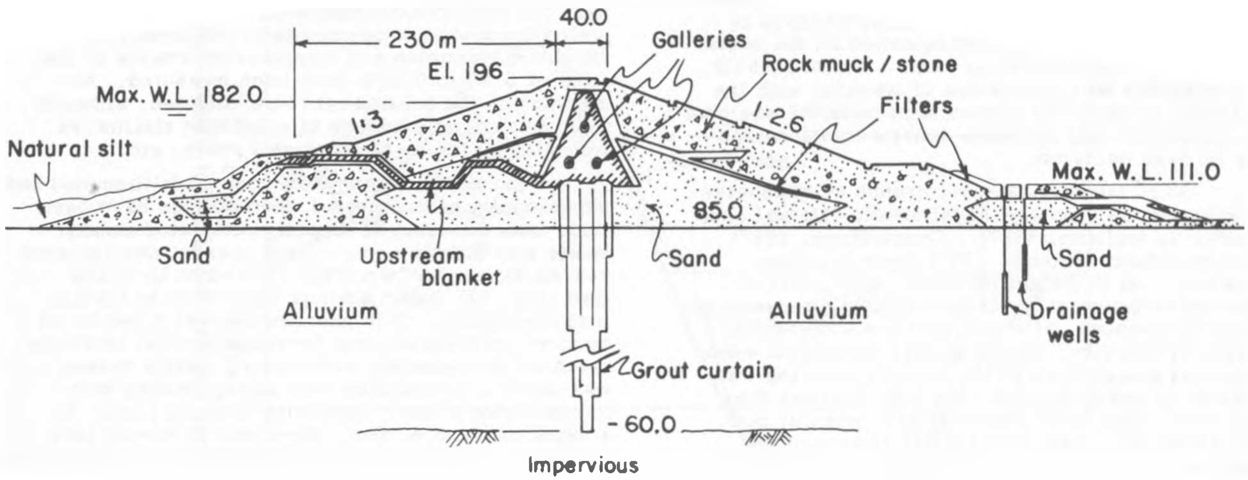


Fig. 30 Cross Section of High Aswan Dam, Egypt

that must be considered in design. Of particular concern with respect to cracking are the transverse cracks that extend from the upstream to the downstream side of an embankment and the horizontal cracks that may develop as a result of differential vertical movements within the impervious zone. The ASCE Committee on Earth and Rockfill Dams (1967) rated cracking within embankments a subject of high priority: one in which more information is greatly needed.

At the present time, very little quantitative data exist concerning the development of strains and cracking in embankments. Perhaps one of the best known case histories in which longitudinal movement data were available to correlate with transverse cracking was originally compiled and discussed by Sherard (1953). Sherard discussed the longitudinal movements observed between reference monuments on the crest of Rector Creek Dam, a zoned earth embankment, and the transverse cracking that was observed at the abutments. Cracking was observed shortly after reservoir-filling started and again two years later. Longitudinal strains, measured by use of crest monuments, varied between 0.1 to 0.35 percent at the time of cracking. According to Sherard, the longitudinal strains that led to crack development were a result of differential settlements between the central portion of the dam and the abutments. Leonards and Narain (1963) studied the settlement and performance characteristics of five dams, four of which developed transverse cracks at the abutments during or after reservoir storage. However, in only two dams were longitudinal strain data available. In all but one instance, the case histories discussed by Sherard, and Leonards and Narain were for earth embankments placed from 1 to 4 percent dry of optimum.

As part of the study, Leonards and Narain also conducted laboratory beam tests on material obtained from the dams as well as from a borrow source of residual limestone clay. Test results provided quantitative data whereby values of tensile strain to cause cracking could be related to the plasticity and the moisture-density characteristics of the materials. A summary of the laboratory test data for the five dams studied is presented below. All the values of tensile strain were obtained from specimens compacted at optimum moisture content and density.

Material Type[a.]	Tensile Strain, %[b.]
SM	0.19
SC	0.17
SM	0.24
SM	0.07[c.]
SM	0.24

a. Unified Soil Classification System
b. Strain at which cracking was first observed in the beams
c. Time of test was 2 days, other tests are of 4 weeks duration

In addition to tests on the materials obtained from the five dams, Leonards and Narain conducted beam tests using a highly plastic residual clay. The clay (CH) was compacted to a range of moisture contents from dry to 2 to 3 percent wet of optimum using both modified and standard Proctor procedures. They concluded from the beam tests that highly plastic clays are more flexible than clays of low plasticity, but the relative flexibility of clays of low plasticity cannot be distinguished on the basis of plasticity characteristics alone. Marsal (1959) also observed that correlation of cracking with the plasticity index of the embankment material was poor and suggested that moisture-density relationships may be more decisive.

More recently, the occurrence of transverse cracking on the crests of El Infiernillo (Marsal and Ramirez de Arellano, 1967) , Cougar(Pope, 1967) and Round Butte (Patrick, 1967) dams has been reported. At El Infiernillo Dam, minor surface cracking of the crest developed shortly after reservoir filling commenced, at which time the longitudinal strains as measured between crest monuments were somewhat greater than 0.1%, but the cracking was confined to the protective zone that overlies the clay core. This zone is essentially granular but with some clay. The longitudinal strains of the crest are shown in Fig. 31 (See Fig. 13 for the section and Fig. 29 for the profile of this dam). Note that extension strains have gradually increased with time, particularly on the left abutment, and have now approached 1.0% although there is no evidence of increased cracking.

Longitudinal strains of the crest of Netzahualcoytl Dam are shown in Fig. 32 for the first two post-construction years, during which period the reservoir has not been completely filled. Maximum extension and compression strains of the order of 0.10 to 0.20% have been measured. No cracking of the embankment was detected, although there is surface evidence of extension strains as indicated by tilting of guard-rail posts, etc.

Excessive or complete loss of drilling mud has occasionally been reported when borings have been made from the crest of recently completed embankments into the clay core. Such losses were reported at both Akosombo Dam (Fig. 17) and at La Villita Dam (Fig. 23), when borings were made to install instrumentation. This phenomonen was a matter of concern until subsequent investigations at La Villita revealed the probable mechanism. Here a boring was being accomplished with rotary drilling techniques using a heavy bentonitic drilling fluid. At a depth of about 40 feet, there was an abrupt loss of fluid. At approximately the same time, malfunctioning of instruments at about the same elevation but some 100 feet distant in a direction parallel to the axis indicated that fluid, pressumably drilling mud,

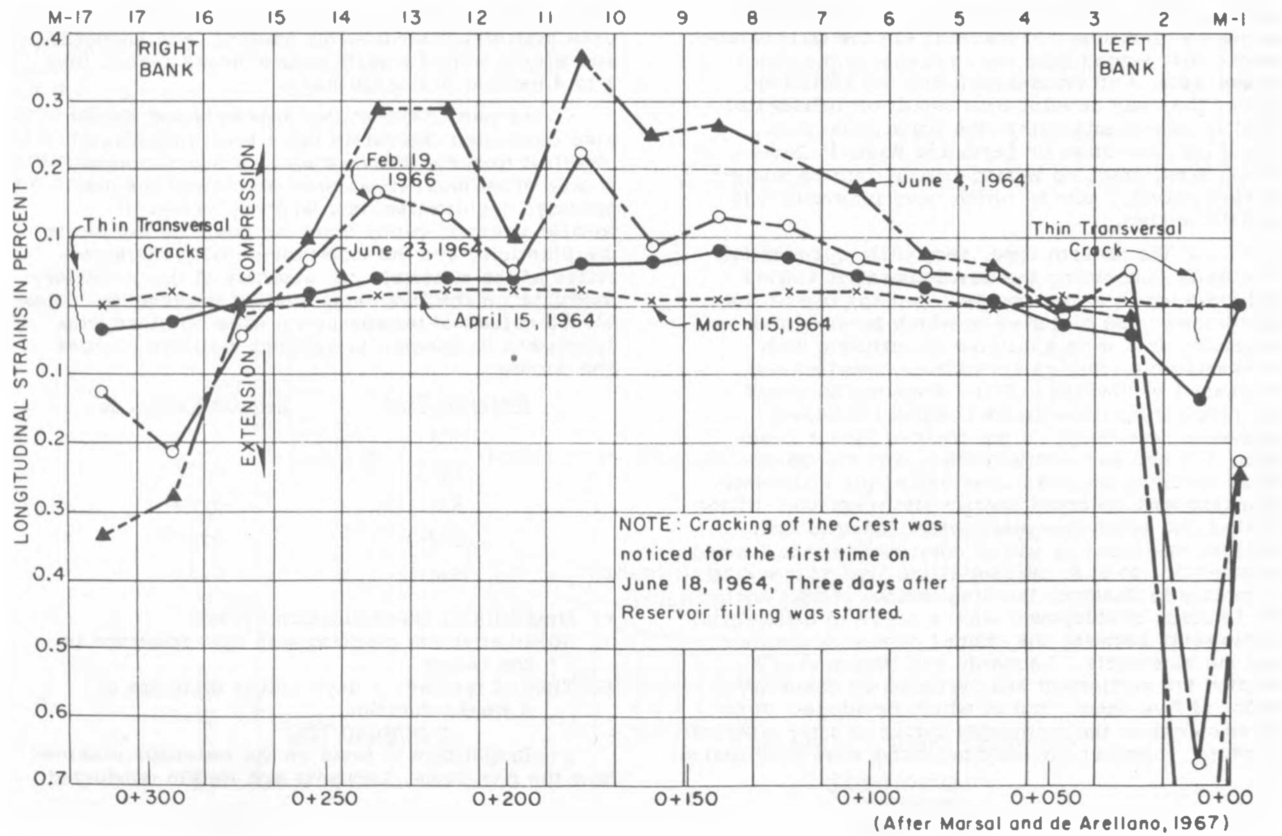


Fig. 31 Longitudinal Strains of the Crest of El Infiernillo Dam, Mexico

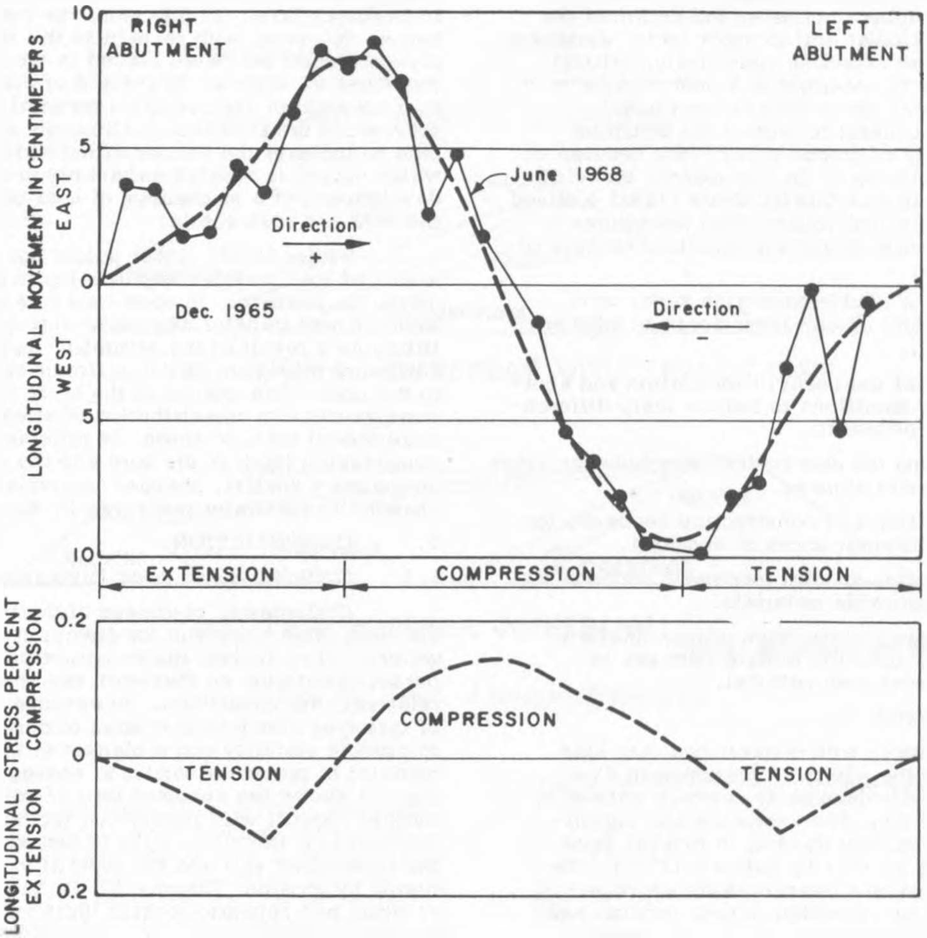


Fig. 32 Longitudinal Strains of the Crest of Netzahualcoyotl Dam, Mexico

entered the instrument system. The bore hole was flushed out with clean water and the rate of drop of the water surface observed with time. It was found that the water surface in the bore hole stabilized within minutes at an elevation such that the water pressure at the bottom of the borehole was nearly identical with that recorded by a nearby pressure cell in the core material, oriented such that its face was in a vertical plane parallel to the axis. It was evident that the coefficient of horizontal earth pressure in the vertical core against this plane was of the order 0.40, which is less than the fluid pressure of the drilling mud at the same depth. The low value of earth pressure resulted from lateral spreading of the upstream and downstream shells, which were granular. In any event, it is evident that the pressure of the drilling mud, which locally may have been increased by pumping pressures and the up-and-down motion of the drill bit, exceeded the earth pressure against a vertical plane parallel to the axis. Once a crack started it quickly spread longitudinally , but once the fluid pressure reduced to that of the soil itself the crack immediately closed. It is believed that the temporary opening of such a crack, although undesirable, would have no harmful effect on the imperviousness of the core, and as the reservoir is filled the earth pressure probably will increase. Fortunately, the earth pressure against transverse vertical planes is greater (except possibly near the abutments), therefore any such cracks that may be opened up by the drilling operations will spread in a longitudinal direction only.

At the present time design studies to evaluate the potential for embankment cracking are largely qualitative. On the basis of such studies as Sherard (1953) and Tamez and Springall (1959)general categories of soil types have been established on the basis of material properties, such as gradation and the plasticity of the fines. Each category is labeled as either highly susceptible, slightly susceptible, or not susceptible to cracking and, by use of such categories, a designer obtains an indication of the potential of a particular soil to crack in an embankment. He can then establish some design criteria for the placement of materials in a dam in order to reduce the danger of crack development and/or establish design criteria to protect the structure against the effects of cracks should they develop or a combination of the two. In this regard, the ASCE Committee on Earth and Rockfill Dams (1967) outlined the basic defensive and construction procedures to protect against cracking and the resultant hazards of piping as follows:

1) Use of a wide transition zone, or of properly graded filter zones of adequate width.
2) Special treatment of foundation and abutment conditions to reduce sharp differential settlement.
3) Arching the dam horizontally between steep abutment slopes.
4) Adjustment of construction sequence for the different zones or sections.
5) Requiring special placement methods for questionable materials.
6) Thorough compaction of rock shells to avoid inducing tensile stresses in adjacent core material.

4.5.2 Load Transfer

It is generally well-established that load transfer may develop within an embankment as a result of relative displacements between materials or between locations. The existence and significance of arching or load transfer in rockfill dams was first reported in 1951 by Lofquist (1951). He described pressure cell measurements which revealed a significant reduction in both vertical and horizontal pressures at depth in the thin cores of several rockfill dams. He attributed the reduction in pressure to the greater settlement of the core with respect to the shells and, hence, to a transfer of load from the core to the shells. Since that time, little mention of load transfer was made in the literature until the early 1960's. In 1961, Nonvieller and Anagnosti (1961) developed a theoretical approach to the state of stress in a core that settles with respect to the shells. Some of the effects of such movement on the characteristics of behavior of an earth embankment, the porewater pressures and settlements, were mentioned by Bishop and Vaughan (1961) with respect to Selset Dam. They attributed the lower than expected settlements and porewater pressures of the puddled clay core of the dam to the greater downward movement of the core with respect to the shells and a transfer of load from the core to the shells. Anagnosti (1961) pointed out that arching or load transfer can occur not only between the core and rock shells, but also cross-valley from abutment to abutment. Peck (1965) noted that, although the cores of some rockfill dams tend to move downward with respect to the shells, the opposite may also occur. Recently, Schober (1967) discussed the earth pressures that were measured at various depths in Gepatsch Dam. In this dam, the core apparently settled downward with respect to the shells. The pressure cells that were placed in the core recorded pressures as little as 45 percent of that corresponding to the weight of the overlying material. These cells provide for the first time sufficiently comprehensive data to indicate the manner of redistribution of stress which occurs in rockfill embankments due to the development of a mechanism of load transfer between the core and rock shells.

Squier (1967) (1968) points out that various modes of load transfer may develop in an embankment during its lifetime. In some cases, a change in the mode of load transfer may occur during reservoir filling as a result of the saturation and resultant downward movement of the upstream shell with respect to the core. The change in the mode of load transfer corresponds to a redistribution of stress in the embankment that, at times, is reflected by changes in compression (both in the core and the upstream-downstream shells), changes in displacements and changes in porewater pressures in this period.

5. CONSTRUCTION

5.1 Cofferdams and River Diversion

Cofferdams, composed of dumped rockfill, are most often employed for diversion of the river waters. They isolate the construction area and permit dewatering so that work can proceed under relatively dry conditions. In essence, a cofferdam of this type consists of a mass of dumped rockfill to provide stability and a blanket of impervious material to provide a barrier to seepage. Diagram A, Fig. 33 shows the simplest form of cofferdam, a dumped rockfill with impervious facing. In some instances, a transition zone is necessary between the impervious soil and the rockfill to control piping by erosion, Diagram B. On the other hand, at times two separate rockfill units may be required to effect diversion with the space between them filled with impervious soil, Diagram C. Such a configuration may be required to divide the head loss at the time of closure or to protect the impervious membrane from erosion. Of further benefit the construction confines the impervious soil and, by lengthening the seepage path, decreases the quantity of seepage flowing into the excavation.

In general, the design of both the cofferdam structures and the temporary waterways are inseparable. The higher the invert and the smaller the capacity of the diversion channels or tunnels the greater is the velocity of the flow through the cofferdam at the time of closure. Hence, the design of the waterways will dictate the minimum size of rock or concrete blocks that will be needed to achieve closure. In addition, the design of the temporary by-pass works will govern the quality of water to be stored during floods and, hence, control to a large extent the rate of placement of material in the cofferdam and the design height. On many major projects, model studies are used to arrive at a cofferdam layout and design that will insure diversion of the expected river flow at the prescribed time.

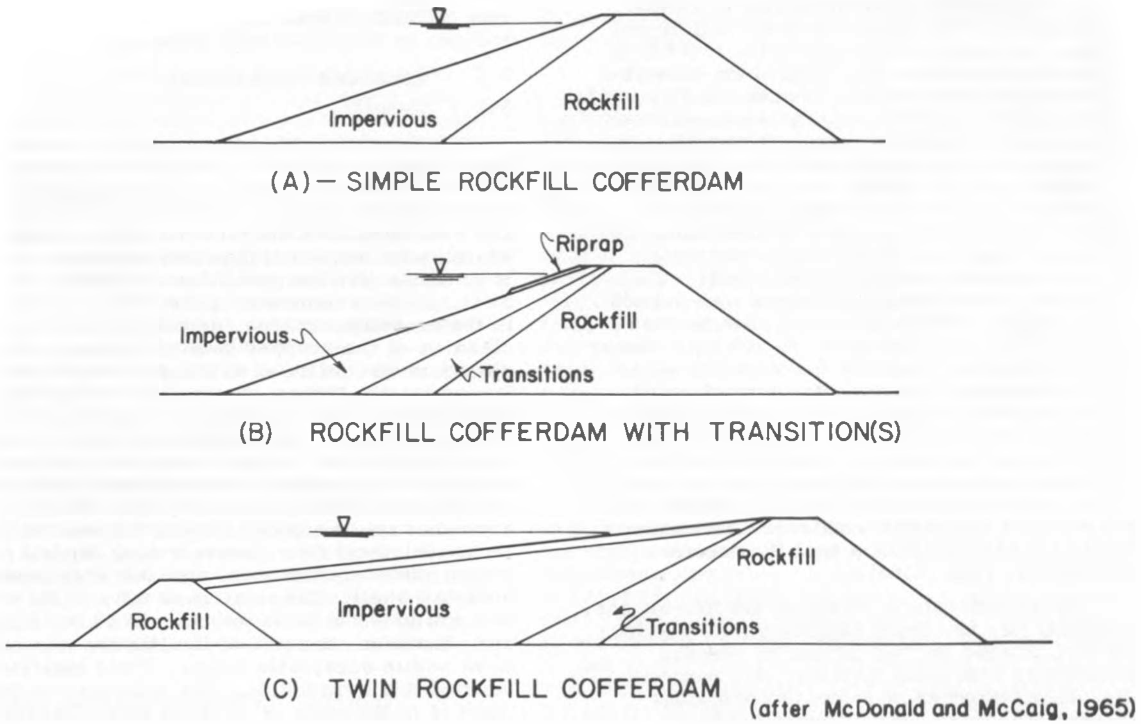


Fig. 33 Types of Rockfill Cofferdams

Where closure velocities are high and rock is expensive to obtain, other types of cofferdams may be used. At Squaw Rapids Dam in Saskatchewan, Canada, an unusual type of "cascade" cofferdam (Taylor, 1961) was used to slow a flow of 14,000 cfs down in stages so that dike material used to close the river would not be washed away as fast as it was dumped (Fig. 34). The cascade was actually a 200 ft. channel between the tip of the stem and the umbrella dike. The total drop in head was dissipated over four separate dikes constructed simultaneously.

In some cases, rockfill cofferdams are designed to be overtopped by floods. At both the Kariba and Roseires hydroelectric projects model studies were made and the cofferdams designed for such an occurrence. At Roseires, a maximum flow of about 8700 m^3/s was reached over the cofferdams with depths of water over the crests in the order of 5m (16 ft.). No major damage occurred to the cofferdams although some gabions on the downstream face of the upstream cofferdam were dislodged, most likely because of undermining, as the finer rockfill was washed through the gabion protection (Lane, 1967). The satisfactory experience with the Kariba and Roseires cofferdams led to further model studies designed to develop the requirements for rockfill dams overtopped and subjected to through flow (Oliver, 1967). Such a design would offer material benefits under certain conditions, such

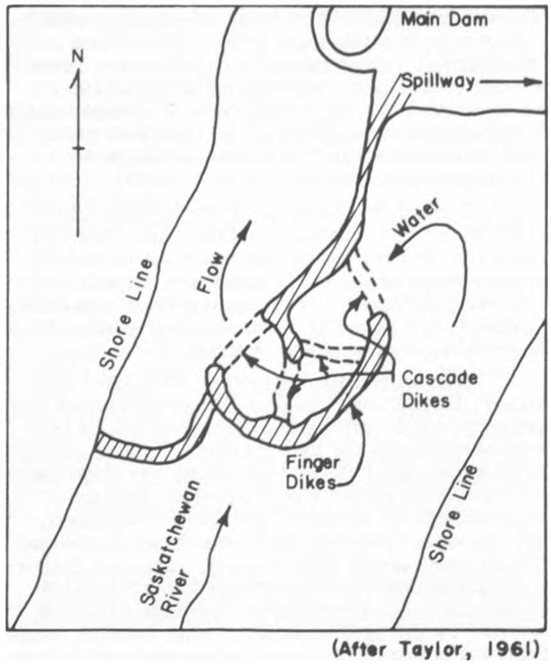


Fig. 34 Cascade Closure Dikes, Squaw Rapids, Canada

as savings in spillway and outlet costs, freeboard allowance and others.

Construction of the cofferdam is usually accomplished by the placement of rockfill by end dumping, although in some instances, rockfill is placed uniformly across the width of the diversion gap by dumping from trestles (Soucek and Gau, 1967). Prior to dumping, usually only the abutments are stripped, but on occasion, pervious deposits may also be stripped from the foundations. To construct the cofferdams for Akosombo Dam, Fig. 17, the entire bedrock surface was stripped of a thick deposit of sand alluvium. This required dredging to depths as much as 200 feet below the water surface (Engineering News-Record, 1963). The upstream and downstream cofferdams were formed by end dumping methods in depths of water that varied from 100 feet, upstream, to 200 feet, downstream. The sound blocks of quartzite placed in the cofferdams assumed an underwater slope of about 1.25H:1V (Bleifuss, 1964). The cofferdams were sealed with 3 to 4 zones of coarse to fine aggregate placed primarily by end-dumping and barge-dumping methods, although a dredge was used to discharge additional blanket material on the face of the downstream cofferdam during unwatering in order to effect a seal (Engineering News-Record, June, 1963).

As an alternate to stripping the foundations, the rockfill may be placed directly on the pervious soils and, if need be, the control of seepage accomplished with grout curtains, thin concrete walls, slurry trenches or upstream impervious blankets. At Mangla Dam, diversion was first effected with a small diversion dam and then a closure dam of rolled sandstone, some 200 feet high, was constructed across the river channel to insure safe discharge through the temporary waterways during periods of maximum flooding. The valley bottom is mantled with a layer of alluvium which in places is 26m (86 feet) thick. To control underseepage, three types of cutoffs were constructed: a 10-foot wide slurry trench cutoff was taken to bedrock on one bank; a grouted cutoff was constructed across the river bed; and a rolled-sandstone cutoff was completed in an excavated trench (Binnie, et al., 1967).

Cofferdams were incorporated into the shells of the dam such as in Akosombo, Mangla, Aswan and others. Some are merely breached to open drainage while still others are partially or completely removed. The operations of breaching or removal are normally accomplished with a dragline working on top of the cofferdam.

At El Infiernillo and other CFE dams in Mexico, initial river closure is accomplished with dumped rockfill cofferdams placed on top of in-place pervious sands and gravels in the river bed. The outer faces of these cofferdams are then sealed with transition zones and impervious facings, supplemented by upstream blankets if required. Next the water between the cofferdams is pumped down as much as is economically justified following which single row thin concrete cutoff walls or interlocking cast-in-place concrete piles are installed by the ICOS method on the interior faces of the cofferdams. Pumping is maintained to keep the water drawn down to the top of the walls during subsequent dewatering and construction of the clay core. This method has proven successful in several very difficult cases. It is expensive but positive, and can be scheduled with reasonable certainty.

5.2 Foundation Excavations

5.2.1 General

The extent of the excavations between cofferdams depends upon many factors, but essentially it is the minimum required to insure suitable support for the embankment. In most dams, all overburden and weathered rock are removed down to material, which, with respect to physical properties, is equal to or better than the rockfill or earthfill to be placed on it. Surface treatment of the bedrock, other than in the core-contact area, usually consists of machine clean-up of the stripped bedrock surface. Such a procedure was followed at Cougar, Round Butte, Summersville, Furnas, Akosombo, Portage Mountain, Oroville Dam, and, of course, many others. At Mont-Cenis Dam the downstream rockfill and filter zones were placed on sound rock, but upstream, the rockfill and protective rockwork were placed on either natural rock or soil, depending upon quality. A somewhat similar procedure was followed at Netzahualcoyotl Dam, where a deep deposit of clean poorly graded medium sand was left underneath the upstream shell. The relative density of the alluvium was evaluated to be in the order of 60 to 70 percent and, therefore, the risk of liquifaction was considered to be within acceptable limits. Field blasting tests verified this assumption. The thickness of the deposit is in the order of 30 m (99 feet) (Gamboa and Benassini, 1967).

It may be more economical in some instances to leave any unsuitable materials in place and to design the embankment accordingly. For example, at Selset Dam a relatively thick deposit of weak clay which existed underneath the embankment was left in place and sand drains installed to increase the rate of consolidation. By use of this procedure, the removal of some 200,000 cu. yds. of unsuitable material, under potentially difficult conditions, was avoided (J. Kennard and M. F. Kennard, 1962). In a similar fashion, sand drains were installed underneath sand dikes of the Kelsey and Kettle hydroelectric projects (Manitoba Hydroelectric Board) to facilitate drainage and accelerate consolidation of the permafrost foundations when they began to thaw (MacDonald, et al, 1960). At times, loose sand underneath embankments may be densified and left in place. The loose sands (N values 1 to 3 blows/ft) underlying Rio Casca III Dam were densified by controlled blasting. Dynamite was used at two depths, 5 m and 10 m. Hole spacing was in the order of 10 m. The maximum settlements of the ground after blasting were about 0.25 m which corresponds to about 2 percent of the maximum thickness of the deposit (Queiroz and Oliveira, 1967).

At some dams, a mantle of alluvium or moraine material exists across the site that is judged to be sufficiently competent to be left in place underneath the embankment. Such was the case at Gepatsch and Mammoth Pool Dams for example.

In the core and transition zones, additional excavation and stripping is normally required to reach rock that is sound and which can be shaped and otherwise treated by various means to provide suitable support.

At Karnafuli Dam in East Pakistan, a deep hole in the diversion channel under the downstream embankment was filled with fine sand by underwater dumping; this was subsequently compacted by underwater blasting (Hall 1962).

5.2.2 Core Contact Treatment

Stripping of the weak and decomposed rock from the core contact area usually exposes a highly irregular rock surface composed of vertical to near vertical surfaces, spanned by benches, overhangs and such structural features as joints, cracks, and bedding planes, and at times, fault zones. In some instances, such erosional features as pot-holes and buried river channels are uncovered. The irregular nature of the rock surface increases the potential for differential settlements and strains in the core that could lead to cracking. In addition, the irregularities and structural features could lead to the erosion of the core by flowing water because of open cracks and of the inability to form by compaction a tight core-rock bond. Because of these hazards, it is essential that more than routine attention be paid to the treatment of the core contact area which usually includes the area underlying the transition zones, particularly downstream of the core.

If the foundation consists of soft rock, treatment of the core contact area is minimized because surface irregularities are easily modified and the rock shaped to receive the first lift of core material. Bond between the core and the foundation can be readily attained by disc-harrowing or otherwise scarifying the foundation before the initial lift is placed or by fusing the initial lift of the core to the foundation with a heavy sheepsfoot roller. On the other hand, with hard rocks, those where rock-shaping or removal is difficult with ordinary earthmoving machinery, various irregularities and certain other structural features of the rock must be treated, using a sequence of operations, many of which are accomplished with hand labor.

As a significant part of this treatment, overhangs are trimmed back and some of the prominent vertical surfaces and rock pinnacles flattened to produce more moderate slopes. Such work is accomplished, preferably, without blasting in order to minimize the opening of old cracks and the development of new ones. As an alternate to rock excavation, a fillet of lean concrete or similar is sometimes placed to fill the gap below overhangs and against vertical surfaces to achieve more desirable slope configurations.

After stripping and special shaping where necessary, the rock surface is cleaned. This operation consists of removing all cracked and loose rock, pockets of earth and seams of decomposed rock, using shovel, pick and, at times, compressed air. Major structural features, such as fault zones and fissures are excavated to a suitable depth and prepared for backfilling. As a final step in the cleaning operations, the entire surface is thoroughly hosed down with powerful jets to wash the fines out of cracks and seams and off the rock surface.

Further treatment is undertaken to effectively smooth the local irregularities in the exposed clean surface and to seal any open cracks. Small grooves and ridges, formed by bedding planes or a particular joint system can most economically be treated at times by filling with lean concrete or grout. Potholes, if they are relatively wide and deep are also usually backfilled with concrete. Foundation treatment in the cutoff trench at Mammoth Pool Dam required the filling with concrete of potholes that reached some 2 to 19 feet across and to depths as much as 3 to 23 feet (Terzaghi, 1962). Pockets or depressions of more limited extent can be filled with hand-compacted earth. If the rock surface is pockmarked with numerous grooves and potholes it has been proven more economical at times to cover the entire area affected with a blanket of slurry or lean concrete. This was done, for example, at Kenny Dam (Huber, 1960). At times, buried river channels and fault channels have been uncovered in the excavations. At Cougar Dam an ancient river channel cut across the embankment in essentially an upstream-downstream direction. It reached a maximum depth of about 85 feet below the base level of the excavations, with a bottom width in the core contact area in the order of 20 feet. This channel was excavated to rock in the core and transition areas and filled with concrete under the core and with transition materials elsewhere (Basgen, 1964). At Furnas Dam a deep fault channel was disclosed in the excavations that required treatment, consisting essentially of grouting (Lyra and Queiroz, 1964).

Elsewhere, open cracks are sealed with slush-grout or slurry. If the cracks are fine and closely spaced the grout can be broomed across the surface to fill them. At Cougar Dam a cement-sand slurry was broomed on the surface to fill all surface cracks immediately prior to the placement of impervious material. Care was taken to insure that the slurry was covered with compacted material while still plastic (Fig. 35). In areas of highly fractured rock, a coat of gunite can be employed to seal the fractures and to form a rounded surface, which facilitates the compaction of the core or earth materials.

After the smoothing of local irregularities, the filling of depressions and the covering of all open cracks, blanket or area grouting is oftentimes accomplished, under relatively low pressures, to seal any open fissures and other similar openings in the near surface rock. Such grouting is usually undertaken across the full width of the core zone. The depth of grouting and the orientation and spacing of the grout holes are dependent upon known geological features of the bedrock and, hence, they are modified during grouting as more information becomes available.

5.2.3 Abutment Treatment

On the abutments, particular attention must be paid to the irregularities in the cleaned rock surface and to cracks and fissures in the rock. Since contact

(A) Slurry is spread to fill voids and to smooth irregular bedrock surface...

(B) ...and then initial lift of core material is placed

Fig. 35 Foundation Preparation in Core Contact Area, Cougar Dam, U.S.A.

pressures are low in the upper part of the core zone, it is important that a good seal be attained at the core-rock contact by adequate compaction. Hence, it is essential that the rock abutments be suitably shaped and prepared. Overhangs, rock pinnacles and vertical surfaces should be modified as noted earlier to produce moderate slopes and to eliminate sources of stress concentrations and possible differential settlements and strains in the core that could lead to cracking. At present the Corps of Engineers specifies that vertical offsets on abutment slopes in the order of 5 feet are tolerable provided that the benches between them result in an average slope somewhat comparable to the overall slope of adjacent areas (Johnson, 1964). At the present time, the maximum allowable slopes of any part of the abutment surface are dependent upon many factors, but foremost, on the judgment of the engineer. A slope of 1:1 is often specified but slopes as steep as 1/8 H:1 V (Netzahualcoyotl Dam) and .5 H:1V (Blue Mesa Dam) have also been adopted in certain instances. Conservative abutment slopes in the order of 1.5 H:1V were specified at Kenny Dam, due in part to the unprecendented height of this sloping core dam at the time of construction in 1952 (Lawton and Lester, 1964).

Field measurements of the internal deformations of earth and rockfill dams (See Section 7 of this paper) indicate rather conclusively that slippage does not develop at the contact of either the core or the shell with the abutment. Furthermore, abutment irregularities such as protrusions, depressions, changes in slopes, etc., appear to have little overall effect on the internal deformations that develop during construction. There are, however, local effects resulting in the development of increased shear strains within the embankment, and certainly in localized zones of high shear stresses within the embankment.

In the case of protruding rock knobs, the increased shear stresses may shear off the knob if the rock is insufficiently strong to resist these increased loads, or has a plane of weakness parallel to the abutment slope.

In the case of buried rigid structures, such as cut-off walls and conduits, the structure must be designed to resist these increased loads. If the structure does not yield, the overburden pressure is great enough to cause the soil to flow plastically around the object; there will be slippage at the contact but not the development of open cracks or fissures. If, however, the buried structure is in the upper portion of the embankment or core, then the cracks may not be self-healing and open voids may result. For these reasons it is concluded that the requirements for abutments shaping and trimming may be relaxed in the lower sections of the abutment, but that special attention must be paid to the upper portion.

Prior to the placement and compaction of the core the rock is usually cleaned of all loose rock fragments and any open joints, cracks or bedding planes filled with grout or treated with gunite. Design requirements also necessitate at times special placement procedures of the core immediately adjacent to the abutment slopes to effect a suitable bond or to obtain certain desirable effects. In some instances, material with specific properties is placed and compacted by hand against the abutments. At Mission Dam, because of the large anticipated settlements of the embankment and other considerations, a 1-foot thick layer of wet clay was compacted against the rock abutments with hand-operated tampers (Terzaghi and Lacroix, 1964). A layer of hand-placed and compacted silt or clay material was used at Mont-Cenis Dam to insure an adequate seal at the core-rock contact, both in the valley and on the abutments (Marchand, et al., 1967). The basic core material in Monte-Cenis Dam was relatively coarse-grained containing a maximum size in the order of 100mm. At Miboro Dam a finer core material (max. size 5 cm) was placed on the abutments and foundation where the sheepsfoot roller was not accessible and compacted with hand-tampers. The core material for the dam consisted of disintegrated granite mixed with clay: maximum size in the order of 15 cm (6 in.) (Asao, 1964).

In some cases, the core and the transitions are flared at the abutments to provide an increased length of seepage and added protection against erosion as for example at Round Butte, El Infiernillo and Furnas Dams.

5.3 Placement and Compaction of Materials

5.3.1 Core Materials

The materials that comprise the cores of dams are widely diversified, from silts and clays to fine granular soils with some silt, to coarse-grained soils, such as glacial till or moraines, that contain an appreciable percentage of gravel and cobble-size material. Because of such diversification, and for other reasons, such as climate, length of construction season and desired physical properties, the methods used to place and compact the core are quite variable from one dam to another and from one region to another. In general, however, the maximum allowable particle size is usually specified to range from 1/2 to 2/3 the maximum lift thickness, although more importantly, lift thicknesses are adjusted so that placement procedures do not result in segregation and compaction results in fairly uniform densities with depth. In actual dimensions, the thickness of the lifts for the core materials in some major dams varies from about 6 in. (15 cm) to 20 in. (50 cm): the latter thickness was used in the core of Holjes Dam when field tests revealed that the maximum particle size could be increased to 12 in. (30 cm) without undue segregation (Reinius, 1964). Compaction of these larger lifts was accomplished with vibratory rollers. At times even thicker lifts can be tolerated. Kjaernsli and Torblaa (1961) describe some field tests at Slottmoberget Dam, which proved that it was practical to place without segregation a moraine material (maximum particle size of 60 cm) in 3 ft. lifts, and to obtain adequate compaction, using an 8 ton vibratory roller.

The moisture content of most core materials is adjusted, preferably in the borrow, either by drying, using admixtures, or by adding water so that certain physical properties, such as strength, permeability, and flexibility are achieved in the compaction process. In arid regions, the clay may be stockpiled, at which time the required water is added. Generally, limiting moisture contents exist for any particular core material: the lower limit is that which will not result in additional settlement of the core upon saturation; the upper limit is that which makes the material difficult to work with, place and compact. Between these limiting values, a moisture content can be selected that takes into account both economic considerations and design requirements. Economic considerations encompass, but are not limited to, such factors as climate and the length of the construction season.

In practice the cores of most major zoned-earth and rockfill dams are placed and compacted at or wet of optimum moisture. For example, at Netzahualcoyotl Dam in Mexico, the core was placed at an average moisture content well above optimum, that is a moisture content of about plus 7 percent. The core material is a silt (ML to MH), derived from the weathering of a weakly-cemented conglomerate (Gamboa and Benassini, 1967).

The cores of Trangslet (Persson, 1964), Summersville (Barnes, 1964), Peruca (Nonvieller, 1964) Mont-Cenis (Marchand et al., 1967) and Cougar (Basgen, 1964) Dams were compacted at or slightly wet of optimum: at Mangla (Binnie et al., 1967), Gepatsch (Lauffer and Schober, 1964) Holjes (Reinus, 1964) and Akosombo (Ware and Hooper, 1964), the cores were compacted at moisture contents slightly wet to 2 percent wet of optimum. At Messaure Dam the wet fill method was used and the core placed and compacted about 4 percent wet of optimum (Bernell, 1964). The wet fill method, originally devised in Sweden, allows for the placement and compaction of sandy or silty moraines in a short and wet construction season (Nilsson and Lofquist, 1955).

Exceptions to this range are the earth dams of the Bureau of Reclamation that are constructed mainly throughout the Western United States. The range of compaction moisture contents in these dams usually varies from 0.7 percent wet to 2.5 percent dry of optimum (Esmiol, 1953) and (Collins and Davis, 1958). As a recent example, the earth zone of Blue Mesa Dam, a 300-ft. high zoned earth dam constructed (1966) in Colorado by the Bureau, was placed and compacted with an average moisture content 1.2 percent dry of optimum (Collins, 1964).

In contrast, most earth dams of the U. S. Corps of Engineers are compacted with a plastic core section, placed at an average moisture content slightly wet of optimum (Turnbull and Shockley, 1958).

Ideally it appears advantageous to place the lower portions of the core at water contents below optimum to reduce pore pressures and compressibility, and to place the upper portions and those zones adjacent to the abutments above optimum to attain increased plasticity and flexibility, thus decreasing the possibility of cracking.

As with material types, the equipment used to compact the core zones of dams is variable, although static sheepsfoot rollers and rubber-tired rollers are most often employed. Up to the present time, 40 to 50 ton rubber-tired rollers were often selected, but more recently, compaction has been accomplished at two major dams, Oroville (Gordon and Wulff, 1964) and Portage Mountain* (Low and Lyell, 1967), using 90 to 100 ton rubber-tired rollers. In the former instance, the core was composed of 3-inch minus material with 10 to 40 percent passing the No. 200 sieve: it was placed in 10 inch layers (compacted thickness). In Portage Mountain Dam, a similar lift thickness was specified, although the core was comprised of a silty sand (SM), with a maximum size in the order of 3/4 inch. To compact the clay core of Mangla, a 45 ton rubber-tired roller was employed with each of the five wheels independently suspended (Binnie et al, 1967).

*Recently named W.A.C. Bennett Dam (Engineering News-Record, 1968).

5.3.2 Transitions

To compact transition zones, most often vibratory rollers are used, particularly when the zones consist essentially of granular materials with a minimum amount of fines. In some cases, however, a certain number of passes of a crawler tractor is specified to effect compaction, such as in Tooma Dam (Pinkerton and McConnell, 1964) and Furnas Dam (Queiroz, 1964). At Gepatsch Dam, the transitions were compacted only by the hauling and spreading equipment. At Tooma and El Infiernillo Dams a minimum relative density of 70 percent was specified to be achieved in the transitions. For all coarse-grained materials in Mangla Dam -- drains, filters, washed gravel fills and similar -- a relative density of 70 percent was also considered satisfactory.

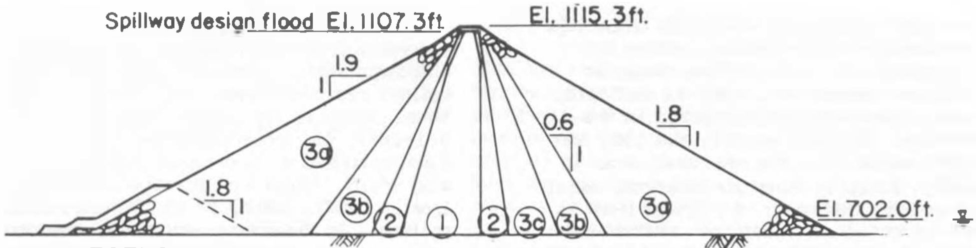
5.3.3 Rockfill

In the years 1940 to 1960, most of the major rockfill dams were constructed of sound pieces of rock, dumped in lifts varying in height from 15 to 100 feet and sluiced with water to rock ratios in the order of 2:1 to 4:1. Nantahala (Growden, 1960) Kenny (Lawton and Lester, 1964), Miboro (Asao, 1964) and some of the central core T.V.A. dams (Leonard and Raine, 1960) are representative of dumped rockfill construction in this period. During the latter part of this period, however, the superior performance characteristics of rockfill, compacted in thin lifts with vibratory rollers was publicized (Hellstrom, 1955) and (Roberts, 1958). It soon became apparent that it would be completely practical to utilize in certain zones of a dam, rock which was not suitable for dumped rockfill, making a compacted rockfill dam superior not only on the basis of performance, but also on the basis of economics. In most recent dams, the bulk of the rockfill is predominantly compacted rockfill.

Carters Dam, a 400-foot high zoned rockfill dam, constructed on the Coosawatte River in Georgia, provides an excellent example of the flexibility and economic advantages inherent in the use of compacted rockfill. Fig. 36 shows the design cross-section of the dam, which is composed of 5 basic zones: Zone 1 is composed of disintegrated rock and residual overburden; Zone 2 of weathered rock; Zone 3C of random to moderately weathered rock; Zone 3B of sound to slightly weathered rock; and Zone 3A of sound fresh rock. The legend outlines the variable thicknesses of layers and types of compactors used to place the embankment materials.

In general, compacted rockfill is placed in most dams in lifts less than 2m, although greater thicknesses have been used occasionally: for example, a lift thickness of up to 3m was used at Goschenenalp Dam (Cooke, 1960). In this dam, the rockfill was dumped wet in layers and compacted with hauling and spreading equipment. The shell material consisted of talus material with about 45 percent smaller than the 100mm size. Above lift thicknesses of about 3m to 4m the rockfill falls in the dumped rockfill category. As noted in the discussion of the placement and compaction procedures for the core, a wide variety of equipment is used to compact rockfill, from crawler tractors (Furnas, Messaure and Netzahualcoyotl Dams), to rubber-tired rollers (Brownlee) to vibratory rollers. At present, the static weight of vibratory rollers varies between about 3.5 tons to 15 tons, although in the near future even heavier rollers will probably become more prevalent. Recently, rollers, varying in weight (static) from 8 to 10 tons were used to compact the shells of such dams as Mangla, Cougar, Round Butte, Gepatsch and Mont-Cenis. At Portage Mountain Dam, compaction of the granular random shells was effected by a "mule train" consisting of a crawler-tractor pulling 6 - 6 ton rollers, hitched in 3 tandem pairs and compacting a path three times the width of one roller. It was judged that one pass of the mule-train was equivalent to four passes of a single roller (Low and Lyell, 1967).

Whether compacted rockfill is sluiced or not is also a variable practice, dependent in most respects on the physical characteristics of the rock being placed. The compacted rockfill in Mont-Cenis, Trangslet, Messaure and Akosombo Dams was sluiced, the latter with water to rock ratios in the order of 1/2:1. However, in many of the more recent



LEGEND

ZONE	MAX. SIZE	LAYER THICKNESS (after compaction)	COMPACTION
1	4in.	6in.	Sheepsfoot
2	8in.	12in.	50 ton rubber-tired
3c	(not to intefere	2ft.	—
3b	with	2ft.	—
3a	compaction)	3ft.	Vibratory roller

Fig. 36 Design Cross Section of Carters Dam, Georgia, U.S.A.

dams, the rockfill was not sluiced, such as in Cougar, Round Butte, El Infiernillo, Gepatsch, Carters, Summersville and Mangla. In addition, compaction of the granular shells in Oroville and Portage Mountain Dams was done without sluicing. At Netzahualcoyotl Dam heavy rains undoubtedly produced the same effect as sluicing.

In some cases, the pollution or contamination of the river with sluicing runoff, which may require the collection and special treatment of the runoff in sedimentation ponds, may override all other considerations.

In some of the Scandanavian countries and others, rockfill is placed in specified portions of the shells under winter conditions. In these areas the rockfill is dumped and compacted in lifts with vibratory rollers. In order to eliminate the risk from freezing, the height of the winter fill is restricted to 10m each year. Winter fills permit economical construction of rockfill dams in areas where otherwise under normal practices, low temperatures during long periods would cause scheduling difficulties. Continuing studies of settlements and compressibility of summer and winter-placed rockfill indicate that very little difference in compressibility exists between summer and winter-placed rockfill (Bernell, 1967). Winter rockfill was also placed in the shells of Gepatsch Dam, toward tne outer slopes. Placement of such fill contributed to a reduction in construction time by as much as one year (Neuhauser and Wessiak, 1967).

The requirements for sluicing, which add appreciably to the cost of a rockfill embankment, need further review and study. It appears that for minimum post-construction settlement the shells should be constructed of well-graded unweathered hard rock or of sand and gravel, placed in layers, and watered or sluiced and well compacted. The effects of sluicing are to weaken the points of contact and thus permit them to break down. This breakage, however, may not develop until the full weight of the embankment is attained. Therefore it is possible that flooding of the surface at perhaps 10 or 20 meter intervals may be fully as effective as continuous sluicing during placement. It also may not be as essential to wet the downstream shell in arid climates since it will never subsequently become saturated.

5.4 Underwater Fills

Conventional methods of constructing a dam in the dry may at times be both impractical and uneconomical because of the requirements and uncertainties in connection with the construction of the necessary cofferdams and the dewatering. Extensive deposits of pervious alluvium, in some cases extending to unknown depths, all covered by deep and fast flowing river waters, usually are the major conditions resulting in the development of a design using underwater methods of construction.

The Dalles Closure Dam ranks high on a list of major embankments of a more permanent nature which were constructed of underwater rockfill. The construction of this embankment required the end-dumping of sound rockfill into depths of water as much as 180 feet with flows up to 200,000 cubic feet per second. Fig. 37 shows a cross-section of the dam which included a dumped rock diversion fill to divert the river flow, the completion of the closure dam by end-dumping, and the placement of an impervious blanket on the upstream face. The materials for the various zones grade in coarseness

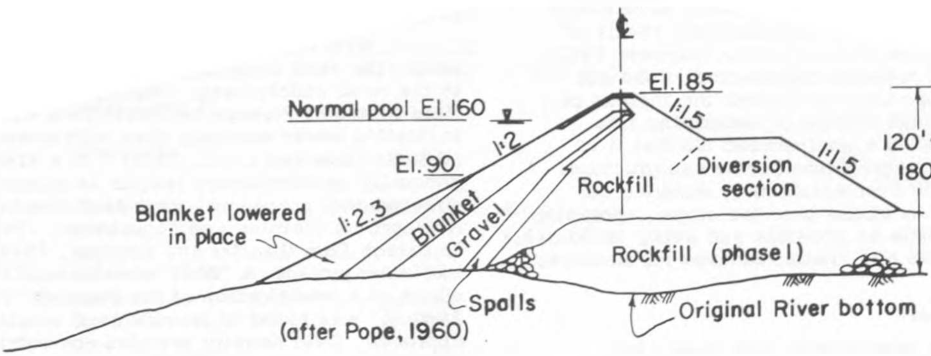


Fig. 37 Typical Cross Section of The Dalles Closure Dam, U.S.A.

from quarry-run rock to sandy gravel, each zone serving as a transition zone. All the materials were placed below water level by end-dumping except that the outer wedge of blanket material, below about elevation 90, was lowered into place to prevent segregation. Laboratory tests on the granular blanket material indicated a permeability in the order of 10^{-2} ft. per minute (Pope, 1960). Another dam employing somewhat the same construction as The Dalles Closure Dam is Arrow Dam, one of the three dams to be built in Canada under the Columbia River Treaty. On this project extensive deposits of pervious alluvium and design heights of cofferdams in the order of 100 feet led to the undertaking of underwater construction. The dam will be composed of a shell of sand and gravel material faced with an impervious sloping core which in turn is connected to an impervious upstream blanket. The impervious materials will consist of glacial till. All the materials, both the sand and gravel and the glacial till, will be dumped by barges in-place. Above the water level the embankment will be completed by normal procedures (Golder and Bazett, 1967).

As an alternate to the basic approach described above two cofferdams may be constructed, one upstream and the other downstream. In between these two cofferdams, dumped pervious fill is placed to some level above the river. The remainder of the embankment is then constructed in the dry after some densification of the pervious materials, possibly by vibro-flotation, and the construction of a positive cut-off. Recent examples of this basic approach are the High Aswan Dam, Fig. 30, and somewhat earlier the embankment closure section for the Wanapum Hydroelectric Project on the Columbia River. Jarkvissle Dam was constructed in a similar manner as that described above, i.e., gravel fill was placed to above river level between two cofferdams. The depth of water was in the order of 5m. The earth dam, which contains a central core and an upstream blanket was then constructed on the gravel fill, using normal construction procedures. However, after construction, settlements and some cracking and piping were observed in the downstream portion of the dam that required placement of additional gravel fill and filter material (Pira and Bernell, 1967).

5.5 Rip-Rap

Studies by the U.S. Corps of Engineers have shown that well graded rock from spalls to maximum size can effectively protect the upstream shells of dams from the effects of wave action (Bertram, 1962). Such a blanket, if properly constructed, does not contain open voids which can permit the ingress of water and subsequent erosion of underlying fine materials. With such a well-graded blanket it is important to avoid segregation during construction. This can usually be accomplished by dumping the rip-rap directly from trucks onto the slope, spreading with tractors as little as possible and using backhoes, orange peel buckets and similar devices for finishing as necessary.

5.5.1 Soil Cement

Within the past decade soil cement has partially replaced rip-rap as an effective and economical means of providing slope protection, particularly if sound durable rock is not economically available at or close to the site. The use of soil cement in this regard was pioneered in this country by the Bureau of Reclamation with a test section in the Bonney Dam Reservoir, Colorado, in 1951 (Holtz and Walker, 1962). Since that time and with continued favorable performance, soil cement slope protection has been used on dams varying in height from 20 to more than 200 feet (Wilder, 1967). Fig. 16 shows the proposed upstream protection of Castaic Dam with a soil cement facing. Studies have shown that it costs about $7.50 per cubic yard to provide a soil cement facing, complete in place and with a thickness of 2 feet measured normal to the slope (Wilder, 1967). At Cherry Dam, an 86-foot high rolled fill embankment in Kansas, the cost of the soil-cement facing was $700,000 less than that of the rip-rap facing (Engineering News-Record, 1964).

5.6 Field Control

The need for field testing and control of materials placed in earth and rockfill dams has been well established. The specific reasons for control of the placement of embankment materials are numerous and varied, but basically field testing and other control measures are required to insure that the properties of the soil or rockfill in-situ are compatible with design assumptions.

In recent years, the volumes of embankments and the rates of placement have increased. It is not uncommon for placement rates to be within the range of 50,000 to 150,000 cu. yds. per day; the upper range was approached at Portage Mountain and Oroville Dams. Such rates have become increasingly feasible by use of large wheel excavators, such as used at Abiquiu (Dunaway, 1960) and San Luis and Oroville Dams (McMinn, 1964) (Construction Methods, 1964) and conveyor systems, such as used at Portage Mountain Dam (Low, 1967). In addition to these has been the increased capacity of earth moving equipment, such as 100-ton bottom-dump trucks. The ever increasing rates of placement of materials in an embankment have forced a critical review of control techniques and field test data analysis. For many projects, this review has involved the trial of different field testing methods during the construction of full scale test fills. Resulting changes have included the development of more rapid, yet accurate, testing techniques and larger size apparatus to accommodate a wide range of material types, together with the statistical approach to specification requirements.

With regard to field control of fine-grained soils, the sand cone method of field density testing is the most widely used. Nuclear equipment was field tested at Portage Mountain Dam with the results indicating lower accuracy than with conventional methods (Low and Lyell, 1967). In a similar manner generally unsatisfactory results in comparison with conventional procedures were also obtained at Holjes Dam (Reinius and Fagerstrom, 1966) and Gepatsch Dam (Lauffer and Schober, 1964). At the latter project, a "body measurement method" which is a modification of the Swedish "Pyconometer Method" was found to provide good results. In this approach, field density samples are obtained by placing basket-like plastic nets in the fill. After compaction of enough material to fill the basket the net is carefully excavated and the density and water content of the enclosed soil sample determined.

Calibration of this test method showed a range of variation in density in the order of ± 1.5 percent.

Other standard tests performed in the control of impervious fills include gradation, compaction and Atterberg limit determinations. Occasional permeability tests are also included, usually in conjunction with compaction tests. The frequency at which control tests are taken varies considerably with the type of materials used and the agency responsible. Table VIII summarizes the frequency of testing for a number of major dams, constructed by different agencies.

For the density control of coarse-grained embankment materials the trend has been to large diameter (2-1/2 to 6 feet) in-place density tests. Table IX shows the size of template used, weight of material excavated and the resulting volume of the hole at several embankment dams. Volume measurements are generally determined by a water replacement method, wherein the amount of water used is accurately measured as it is fed from a field test vehicle. Another method used on occasion to measure volume utilizes the backfilling of the hole with washed pea gravel. A unique method of volume determination of large diameter tests is reported by Ringheim (1967); at the South Saskatchewan River project the cavities were backfilled with calibrated wheat.

Where the water replacement method is used, the cavity is usually made watertight by a thin polyethylene sheet. At Cougar Dam a coating of fire clay, of known volume was employed as a base for the polyethylene sheet (Basgen, 1964). In most test procedures, the polyethylene sheet is draped over the test surface and ring and filled with water to calibrate the roughness across the surface of the test area.

The high placement rates previously mentioned have resulted in the necessity for rapid determination of material grading. The rapid method of gradation control developed at the Oroville and the Portage Mountain projects has proven successful. With this method, wash sieving and wet weighing is used to determine the percentage passing one or two critical screen sizes. Then, with an estimate or a rapid determination of moisture content, the suitability of the material is judged. The rapid gradation test is estimated to take approximately 20 minutes. The procedure followed at Oroville included a conventional gradation analysis on a carefully split portion of the sample as a check on the rapid test results.

The suitability of most cohesionless coarse-grained soils is evaluated by comparing the field density with the maximum and minimum or just the maximum density as determined in a laboratory test. The data are expressed either in terms of a relative

Table VIII Frequency of Field Testing In A Selected Number of Dams

		Frequency of Test X 1000 cu. yds. (Except as Noted)				
Dam	Test	Core	Transition	Filter	Drain	Shell
Portage Mountain	Field Density	4.0	5.0	2.5	50.0	10.0
	Gradation	1.5	1.8	0.5	3.0	3.5
Mangla	Field Density	1.5 to 4.0				8.0 to 10.0
Oroville	Field Density	3.0 to 4.0	50.0	50.0		
	Gradation	3.0 to 4.0	Ea. 24 hrs.		150.0	150.0
	Compaction	3.0 to 4.0			50.0	Ea. 24 hrs.
	Atterberg Limits	3.0 to 4.0				
	Moisture Content	3.0 to 4.0	Ea. 8-12 hrs.		As needed	
Gepatsch	Field Density	0.65	6.5			Ea. 6 to 12 months
	Gradation	0.65	6.5			Ea. 6 to 12 months
	Compaction	20.0	40.0			
	Permeability	20.0	40.0			
Akosombo	Field Density	0.80				
Cougar	Field Density	Ea. day				
	Gradation	Ea. day				
* *	Field Density	0.5 to 4.0				10.0 to 100.0

* * Summary, Joint ASCE-USCOLD Committee on Current United States Practice in the Design and Construction of Arch Dams, Embankment Dams and Concrete Gravity Dams (1967)

Table IX Some Details of the Field Density Test Procedures In Coarse Embankment Materials

Dam	Template Or Ring Diameter in feet	Amount of Material Taken in Lbs.	Vol. of Hole, Ft^3
Oroville	6.0	1500 to 1800	10 - 12
Portage Mountain	2.5	150	1 - 2
Gepatsch	3.9	340 (approx)	2.8
Cougar	6.0	2400 (approx)	20.0

density or in terms of a percentage of the maximum density. As noted earlier in another section, in many instances a relative density of 70 percent is considered satisfactory. Where the percentage of maximum density is used, from 95 to 98 percent is, in many cases, the minimum specified.

For most projects the maximum density of cohesionless coarse-fill materials is determined by vibratory methods. At Portage Mountain Dam, ASTM method D 2049-64T was used. Similar test procedures were employed at Cougar and Oroville Dams with larger size sample molds.

The analysis of construction control data and an evaluation of the suitability of the earthwork have, in recent years, tended toward a statistical approach. As pointed out by Turnbull, et al., (1966) "there is a growing awareness of the random variation of the various soil properties pertinent to earthwork compaction." Among others, the statistical approach (assumed normal random variation of test data) has been used at the Mangla Dam (Atkinson and Kerr, 1967), Portage Mountain Dam and of course numerous others. At the latter project, the specifications allowed for one in every ten tests to be below the minimum acceptable percentage compaction.

6. INSTRUMENTATION

6.1 Introduction

Prior to about 1960, instrumentation placed in embankments was, with but few exceptions, inadequate to provide insight into the behavior of the materials within the various zones comprising the dam or in the underlying foundations. The principal types of instrumentation for measuring settlements or displacements in this period were plates, surface reference monuments or crossarm devices or similar. The crossarm devices were primarily placed in earth embankments and except for such dams as Quoich Dam (Roberts, 1958) and certain other dams in Sweden (Westerberg et al, 1951) very little instrumentation was installed within rockfill dams. The plates and reference monuments were generally placed on the outer slopes after construction was completed. Hence, as a result of the limited availability and use of instrumentation few data were obtained concerning the magnitude and distribution of displacements, strains, and stresses occurring within an embankment during the critical construction and post-construction periods. The observational approach to design is discussed by Peck (1969).

Beginning about 1960 instrumentation has been developed and installed in an ever increasing number of major dams to measure horizontal displacements, strains, settlements and stresses. These various types of instruments are providing, among other things, significant quantitative data concerning the relationship between the properties of embankment materials and displacement strains, and stresses that occur in embankments during various critical periods, including earthquake events. Some of the principal types of instrumentation that are presently available for installation in the embankments and foundations of embankment dams are discussed in the following sections. The discussion covers piezometers and various devices which permit the measurement of displacement, strain and stress within an embankment. Other devices for observing surface movements are briefly mentioned. The needs for instrumentation and the problems associated with their design and performance are discussed by Wilson (1967).

6.2 Piezometers

Piezometers measure the static pressure or head (elevation to which water will rise in an open standpipe) of the air or fluid in the pore space between the solid mineral grains of the soil. These air or fluid pressures are referred to as neutral stresses and knowledge of the magnitude of these stresses is essential to an evaluation of the performance of an embankment.

During construction, piezometers may reflect a build-up of pore pressure as a result of an increase in embankment height or, alternatively, they may reflect dissipation of pressure with time. In any event, any changes in conjunction with appropriate displacement measurements provide insight into changing effective stress patterns and areas of minimum or maximum compressibility. Following construction and after reservoir filling, the piezometers reveal the pattern of seepage through the dam. To accomplish the foregoing functions in a meaningful manner, piezometers must be selected that (1) are reliable and durable, (2) exhibit satisfactory time lag and sensitivity characteristics and (3) contain elements with certain prescribed characteristics which insure that the desired pressures are being measured. For example, to exclude the effects of pore air pressure and to insure that pore water pressure is being measured, fine ceramic tips, which have large entry air values, should be specified (Bishop et al, 1961).

Although there does not today (1969) exist a single all-purpose piezometer that will give reliable results over an extended period of time in all types of soils, there are four general types in current use which have proven generally satisfactory.

6.2.1 Open Standpipe Piezometer

This type of piezometer varies mainly in the diameter of the standpipe and in the type and volume of the collecting chamber. The simplest type is merely a cased or open observation well in which the water level is measured directly by means of a small probe. In this case the static head is the average head which exists over the depth of the inflow part of the wall below the water table. This measured head may be higher or lower than the free water table and, in the case of moderately impervious soils, may be subject to a large time lag.

The Casagrande-type piezometer consists of a porous stone tip embedded in sand in a sealed-off portion of a boring, and connected with a 3/8-inch diameter plastic riser tube (Shannon, et al, 1962). When properly installed, the Casagrande-type has proven successful for many materials and its non-metallic construction is corrosion resistant. In compressible embankments downdrag forces may cause severe buckling of the riser pipe and under such conditions it should be placed inside a telescoping outer casing.

Although the open standpipe type is not satisfactory in very impervious soils because of time lag, and in partially saturated soils because of the difficulties of evaluating pore-air or pore-water effects the simplicity, ruggedness and over-all reliability of this type dictates its use in many installations. As a matter of interest, the reliability of unproven piezometers is usually evaluated on the basis of how well the results agree with those of the Casagrande-type.

6.2.2 Hydraulic Type

The hydraulic type piezometer consists of a collection chamber connected directly to a pressure gauge near the downstream face of the dam. The pressure gauge and its housing should be at an elevation slightly greater than the piezometer tip. The USBR type of piezometer (Earth Manual, 1960) has been used successfully in hundreds of major projects. It requires long tubing lines, expensive and complex gauge houses, and careful techniques during installation and operation. De-airing and water circulating procedures may influence and change the pore water pressure readings. Leaks at fittings must be especially avoided.

Single pressure lines from the tip to the gauge are unsatisfactory because air may collect in the line, therefore twin lines are used so that water may be circulated to purge the lines. This requires that the lines be of small diameter. Many variations of the hydraulic type are available, differing mainly in the size and type of tubing, and design of pressure gauges and circulating systems.

The locations of the hydraulic piezometers installed in the core and transitions of Oroville Dam are shown in Fig. 38. The terminal "S" in the core block serves the hydraulic piezometers in the embankment below elevation 540. The remaining piezometers above elevation 540 are monitored in the shelter, designated Terminal T (Perry and Kruse, 1968). In addition to these piezometers, six hydrodynamic piezometers were embedded in the upstream pervious and transition zones. These piezometers are modified electrical pore pressure devices, containing a strain gage sensor. During a seismic event, the piezometers will be triggered and any transient pressures measured and recorded.

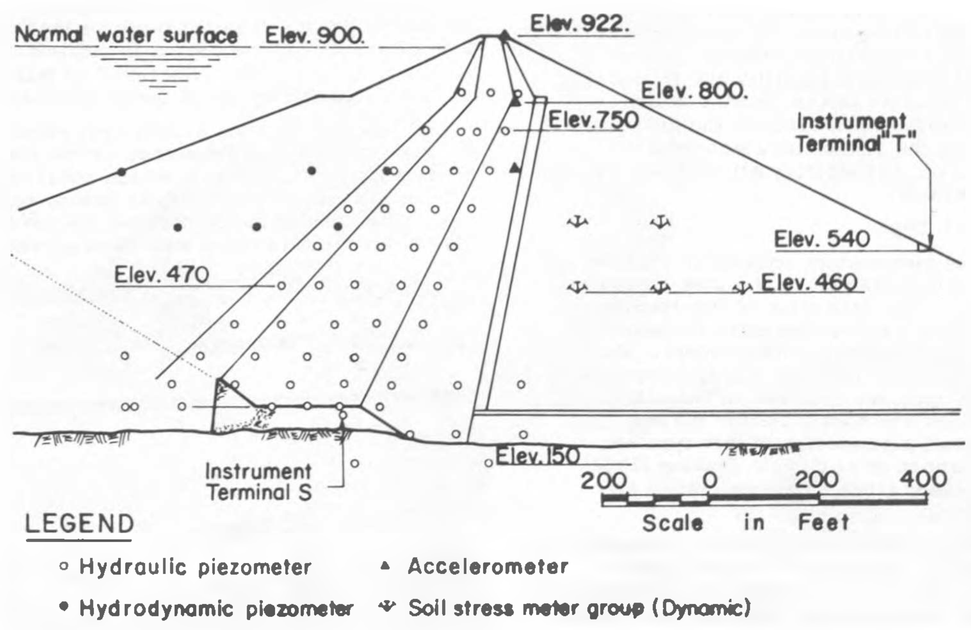


Fig. 38 Layout of Piezometers, Accelerometers and Pressure Cells in Oroville Dam, California

6.2.3 Pneumatic Type

The pneumatic type piezometer consists of a sealed tip containing a pressure-sensitive diaphragm or valve. Gas or a fluid introduced into one of two lines leading to the piezometer causes the diaphragm or valve to open when the pressure applied equals the balancing hydrostatic pressure on the other side of the diaphragm. The opening of the diaphragm or valve bypasses the gas or fluid into a return line. The applied pressure required to open the diaphragm is taken as the pore pressure at the piezometer. In the Warlam piezometer (Warlam and Thomas, 1965), air is used whereas in the Gloetzl piezometer (Lauffer and Schober, 1964) a hydraulic fluid is used. Fig. 39 shows the field installation of a pneumatic piezometer of a design in which the valve closes and locks a pressure into the outlet line equal to that of the pore pressure.

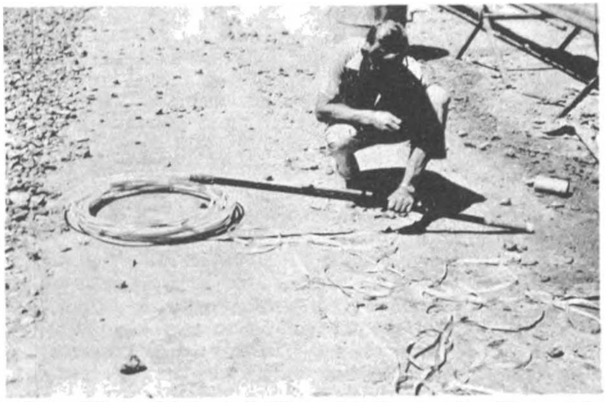


Fig. 39 Field Installation of a Pneumatic Piezometer

There are many advantages to pneumatic piezometers: (1) the small volume change required to operate the valve and resulting negligible time lag; (2) simplicity of operation; (3) capability of purging the lines; (4) minimum interference with construction; (5) long-term stability and (6) the instrumentation terminal can be located at any reasonable elevation with respect to the piezometer. It is probable that the pneumatic piezometer will prove to be the most satisfactory all-purpose type for most installations.

6.2.4 Electrical Type

Electrical piezometers consist of a tip having a diaphragm that is deflected by the pore pressure against one face. The deflection of the diaphragm, which is proportional to the pressure, is measured by means of various electrical transducers. Such devices have negligible time lag and are extremely sensitive. Thus they are suitable for installation in quite impervious and highly plastic clayey materials. Typical piezometers of this type are described by Shannon et al (1962); Cooling (1962); Brooker and Lindberg (1965); Penman (1961); and Bishop et al (1964).

Because of severe environmental problems and long-term stability, the use of electrical piezometers is not generally recommended for installations in embankments where reliable readings are required over an extended period of time.

6.3 Internal Movement Devices

Internal-movement devices are used to measure the components of movements and strains within an embankment, both during and subsequent to construction. Therefore they must be installed as the embankment is placed, an initial reading must be obtained at the same time, and frequent readings should be obtained thereafter.

6.3.1 Vertical Movement Devices

These devices may be installed so as to either measure the settlement of numerous points, all essentially within the same horizontal plane, or to measure the settlement at different depths in a vertical plane. In the latter instance, the settlement readings allow compressions of embankment materials to be calculated.

The USBR crossarms are noteworthy as being the first of their type and today are still considered to be the most reliable. The crossarm gauge consists of a series of telescoping 1-1/2 inch and 2-inch pipe sections with the 1-1/2 inch sections anchored to the embankment by horizontal crossarms at 5-feet or 10-feet intervals. A torpedo attached to a steel engineer's tape is lowered down the pipes. Pawls successively engage the lower ends of the 1-1/2 inch pipe. To avoid interference with the contractors equipment, the pipe and crossarm may be kept below the surface elevation at all times, new sections being added by digging pits to expose the lower sections.

Three-inch, telescoping, grooved-aluminum casing sections, used with certain types of inclinometers (described subsequently herein), have also been successfully used to measure settlements and compressions in both earth and rockfill dams (Wilson, 1962). To measure settlements, a settlement torpedo with steel tape is used to measure successively the distance from the top of the casing to the bottom of each 5 or 10-foot section of casing. Crossarms are not required because the friction between the soil and the casing is sufficient to force the joints to telescope. Section 7 of this paper presents data from some of these installations.

Fig. 40 shows a simple, yet effective, vertical-movement device developed by the Walla Walla District Corps of Engineers which consists of wires attached to couplings of plastic casing set in a bore hole. The annular space between the plastic casing and the borehole is filled with sand or weak grout,

Fig. 40 Vertical Movement Device of Simple Design

and the wires maintained taut with tension springs. Settlement or vertical movement is detected by a change in distance between a mark on the wire and the frame which is anchored at the ground surface. To measure accurately much smaller settlements or vertical strain, long-gauge, multiposition borehole extensometers may be used as shown in Fig. 41. Several types of anchors are available, including types that may be grouted or wedged into rock. The anchors shown in Fig. 41 require an uncased hole from 3 inches to 6 inches in diameter and they consist of three stainless-steel bars which expand outwards into the soil or rock. A double rod is used to expand the anchors. The hole may be filled with drilling mud, grease, or weak grout. The high-tensile-strength wires are encased in a plastic tube. In caving soils, a thin plastic casing may be used to stabilize the borehole as the anchors can be punched through the plastic casing and into the soil. As many as six anchors can be placed in a single borehole. After passing over the pulley of a precision potentiometer, each wire is maintained at constant tension with preformed coil springs in an instrument box, similar to that shown in Fig. 42. These devices are easily read with a simple Wheatstone bridge circuit, have great sensitivity (approximately 0.002 in.), and considerable range (2 inches or more).

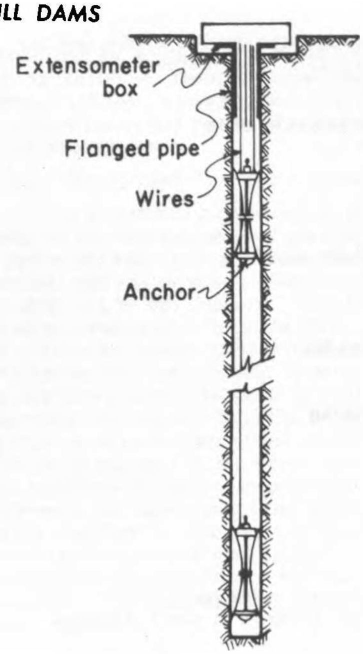


Fig. 41 Long-Gauge Multiposition Bore Hole Extensometer

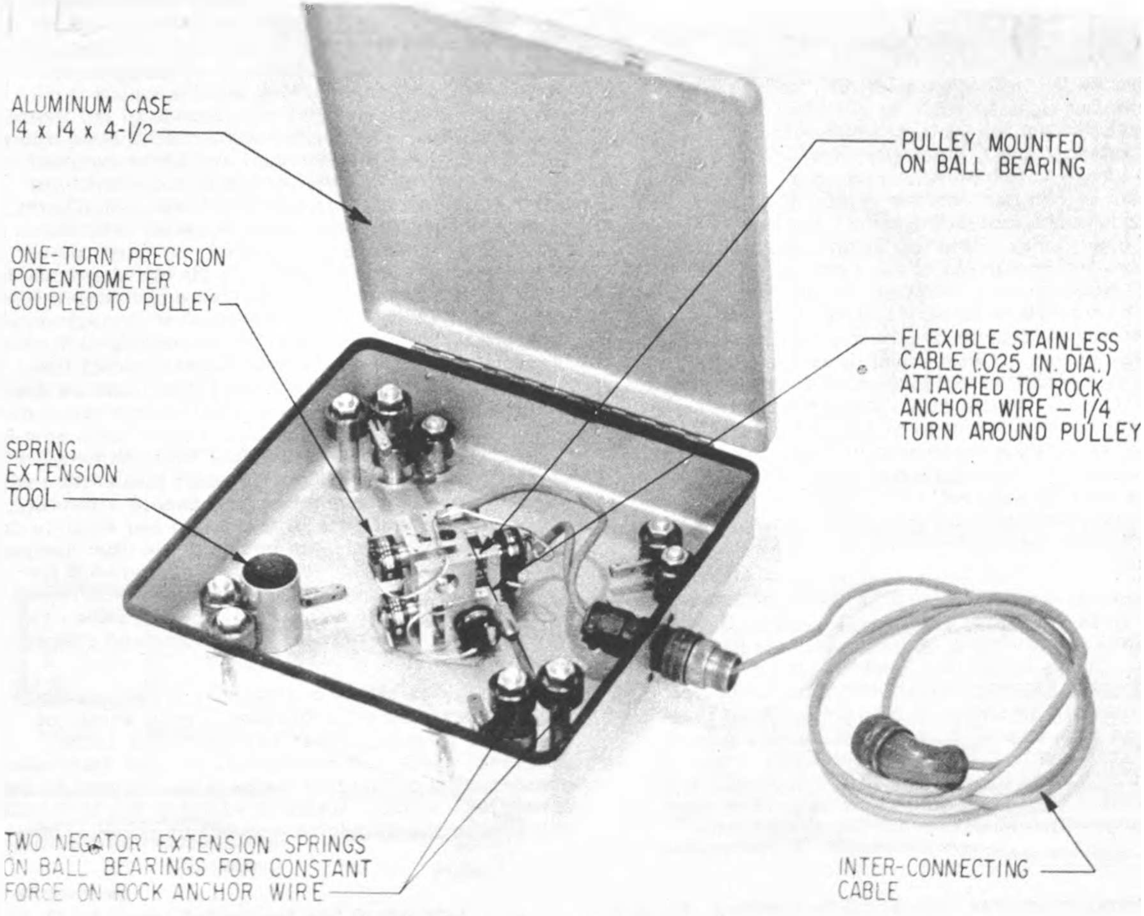


Fig. 42 Extensometer Instrument Box

A relatively inexpensive device is the Geonor settlement probe, Fig. 43. This device, as

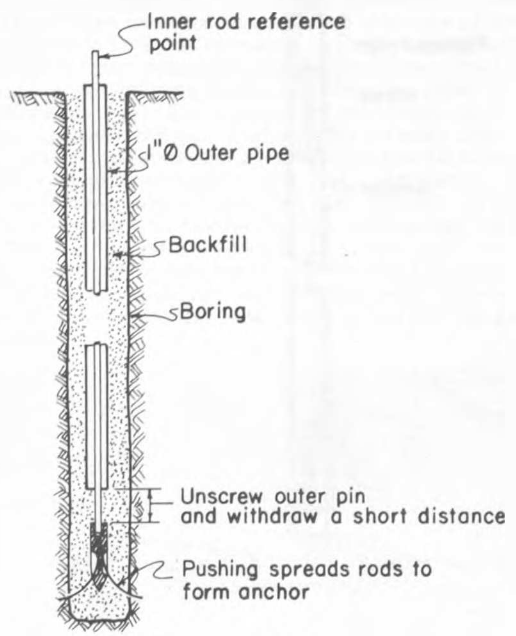


Fig. 43 Geonor Settlement Probe

described by L. Bjerrum et al (1965), consists of a three-pronged tip connected to a double rod which is lowered down a borehole or driven into soft ground to the desired depth. When the outer rod is held and the inner rod driven with a hammer, the three prongs are forced out into the surrounding soil. The outer rod is then uncovered from the tip and withdrawn a few inches. The top of the inner rod, which remains connected to the anchor, is used as a reference point to measure the settlement of the tip. This device is particularly well-suited to measuring settlement of soft foundations under low embankments. Other types of single-anchorage devices are described by Bjerrum et al (1965), and Cooling (1962). Rouse et al (1965) describe a rock-foundation gauge used at Davis Dam (USBR) in which the relative movement between the anchor at the bottom of a bore hole and the surface is detected with a sensitive dial gauge. In such cases, the gauge must be permanently mounted in a protective gauge house which is accessible to personnel.

Vertical movements in an embankment can also be measured using fluid level devices that are based upon the principle of intercommunicating vessels. These settlement devices are of two general types. An individual-unit type, installed at Oroville Dam is shown in Fig. 44 (Golze, 1966). It is made from two, 2-inch diameter brass pipe-nipples soldered to a common diaphragm. Pipe caps are secured at both ends of the assembly and it is mounted vertically on a steel base plate for anchorage in the embankment. The diaphragm

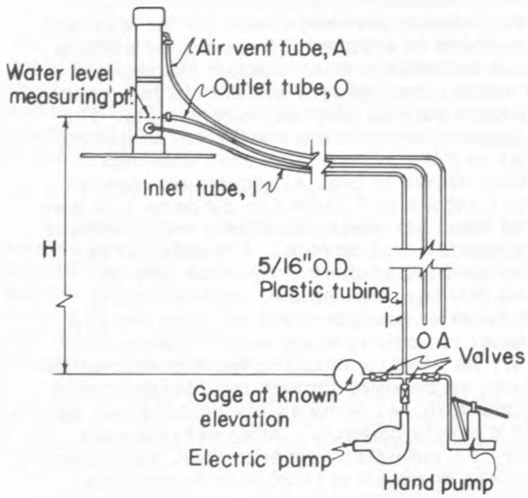


Fig. 44 Schematic Diagram Showing Operation of Fluid-Level Settlement Device, Oroville Dam

separates the upper (air) chamber from the lower (overflow) chamber and encloses a plastic float valve which prevents water from entering the air chamber during flushing of the lower chamber. Three, 5/16-inch O.D. Saran-plastic tubes are embedded in trenches which are excavated to maintain continuous downward slopes to the instrument terminal. The instrument terminal is equipped with a pump, air compressor, and high-precision pressure gauges. A settlement reading involves three steps taken in order: (1) Flushing the lower chamber of the fluid device with water until the return flow is free of air bubbles; (2) using the compressor to force air into the air chamber from which it enters the lower chamber and forces the water level to descend to the level of the outlet tube; and (3) after all water is removed from the outlet tube, evidenced by visual observance of the return flow, pressure measurement is made on the inlet line.

Precise elevation of the device is established after corrections are made for water temperature of the column, and for barometric pressure difference between the elevations of the gauge and fluid device. A high-precision pressure gauge is required for the fluid-level measuring systems. Readings are frequently checked by reflushing the lines and repeating the pressure measurement. Readings are said to be repeatable within a variance of 0.03 feet of water.

Fig. 45 shows a somewhat similar device which was installed in the downstream shells of Messaure (Bernell, 1964) and Djatiluhur Dams (Ambrose, 1965). It consists of an open standpipe connected by tubing to a water tank, located on the downstream slope. Water is added to the tank until overflow of the standpipe occurs, as revealed by

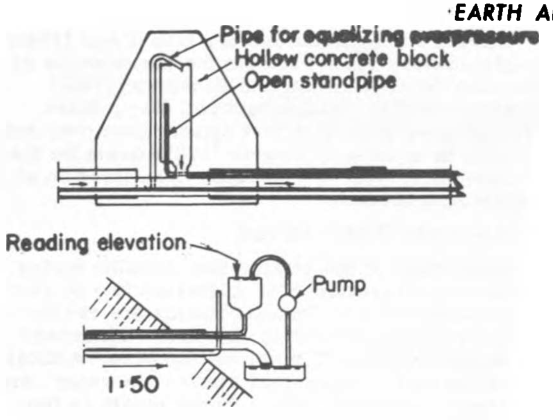


Fig. 45 Vertical Settlement Device in Rockfill, Messaure Dam

observation of the discharge tubing. The elevation of the standpipe is then determined, using a graduated scale on the water tank.

The same principle of inter-communicating vessels can be adopted to measure settlement at different locations, using horizontal tubing, placed in the embankment in an upstream-downstream direction or similar. In this device, a vessel filled with water, called a torpedo, is pulled through a horizontal tube that is inclined slightly upward. The elevation of the torpedo at any location is obtained by use of a manometer connected to the device. Details of the device used at Gepatsch Dam are shown in Fig. 46. Horizontal displacements along the tubing can also be measured by anchoring metal plates around the tubing at select intervals. To locate the plates in the fill a Sonde (Idel) is drawn through the tube. When it reaches midway through the metal plates, an audible "beep" is produced at the power source. A metered cable, used to draw the Sonde through the tubing, provides the distance to the metal plate. Hence, with the horizontal tube installation both vertical and horizontal embankment displacements can be determined. The Idel Sonde system was used to measure horizontal displacements in Gepatsch Dam (Lauffer and Schober, 1964), (Schober, 1967) and Carters Dam (Engineering-News Record, 1966). Of course the same principle of the

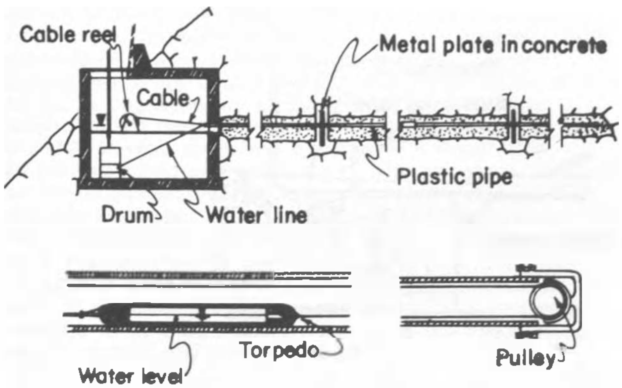


Fig. 46 Fluid-Level Settlement and Horizontal Movement Device, Gepatsch Dam

Sonde can be employed to measure movements along tubing which is oriented either vertically or inclined. Bernell (1964) discussed the substitution of cobalt isotopes for metal plates to measure displacements in Messaure Dam.

6.3.2 Horizontal Movement Devices

All internal movement devices require placement in the embankment of plastic or metal tubing. A probe (Sonde) or similar can be inserted as described above, or wires attached to anchors can be brought out to the surface. The horizontal movement device installed at Oroville Dam, Fig. 47, consists of 1/8-inch aircraft cables connected to horizontal crossarms and brought out to an instrument shelter on the downstream slope (Fig. 48). A winch (not shown) facilitates lifting the weight and transferring it to each individual cable in turn for deflection measurement at constant tension. A mark on the instrument shelter is surveyed periodically to reference its movement (and that of the embedded cable anchors) to the project bench marks. A somewhat similar device was installed in Geehi Dam, a rockfill dam in the Snowy Mountains Scheme, Australia (Hosking and Hilton, 1963) and at Tortolos Dam, Mexico.

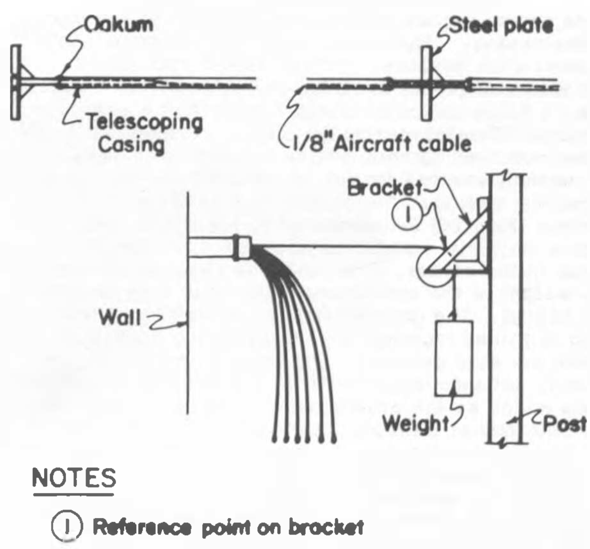


Fig. 47 Horizontal Movement Device, Oroville Dam

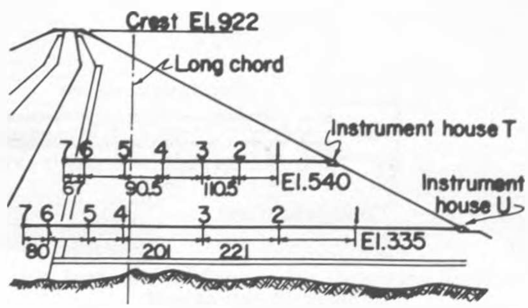


Fig. 48 Location of Horizontal Movement Devices, Oroville Dam

Long-gauge, multiposition extensometers, similar to those used for vertical movements, are used in the horizontal position to measure small movements precisely. They are preferably installed at time of construction, but can be installed in a horizontal drill hole at a later date if necessary.

At Fall Creek Dam, several horizontal crossarm gauges were installed to permit measurement of horizontal displacements. The crossarm locations were determined using a torpedo with spring-loaded prawls, which were retractable by the application of tension to the return cable (Pope, 1967). Fig. 49 shows the essential details of this gauge. The distance from the crossarms to the benchmark was measured with a steel tape. Vertical movement at the crossarm locations was measured by means of the principle of inter-communicating vessels, discussed earlier.

Inclinometers are used extensively to investigate the displacements in embankments. These devices basically consist of a pendulum-actuated transducer enclosed in a watertight torpedo, which is lowered down a near-vertical casing, measuring the inclination from the vertical of the casing at frequent intervals. Horizontal movements deflect the casing, thus changing its inclination.

Most of these devices incorporate sensing units whose output can be remotely read at the top of the casing. Shannon et al (1962) describe several such devices. Wilson (1962) and Wilson and Hancock (1965) describe an inclinometer referred to as a Slope Indicator which consists of a pendulum-actuated Wheatstone-bridge circuit enclosed in a wheel-mounted torpedo whose azimuth is controlled by vertical grooves formed in the walls of extruded aluminum casing. The control box at the ground surface (Fig. 50) is connected to the instrument with a multiwire conductor having a stranded steel cable in the center. This cable is used to support the weight of the instrument while it is lowered down the casing. The grooved casing, 5 feet to 10 feet long is joined together with telescoping couplings which are also grooved. The bottom section is usually set into the foundation and additional sections added as the embankment is placed. Readings are obtained at frequent intervals of depth in each of the four grooves; thus, the two components of horizontal movement can be easily computed. In addition, the vertical movement can be obtained by measuring the depth to each of the telescoping joints using a special settlement torpedo.

Marsal and Ramirez (1965), (1965) and (1967) used inclinometers to investigate the performance of El Infiernillo Dam. Gamboa and Benassini (1967) describe their use in Netzahualcoyotl Dam, Sainz Ortiz (1967) used them at a test embankment over soft ground, and Kaufman and Weaver (1967) describe their use in connection with the stability investigation at the Atchafalaya Levees.

6.3.3 Horizontal Strain Meters

Knowledge of the strains that develop during construction is essential to an understanding of embankment performance. Vertical strains are easily computed from settlement data obtained with crossarms, telescoping casing or any of the other vertical-movement devices. Horizontal strains, however, are more difficult to obtain. If the gauge length is too small local variations may result in non-representative data. Too long a gauge length will integrate true variations into an "average" value. Marsal and Ramirez de Arellano (1965) describe a horizontal strain meter developed for use in El Infiernillo Dam. It consists of three linear potentiometers enclosed in a watertight case which are actuated by solid rods connected to anchor plates embedded in the embankment. A gauge length of from 10 to 15 feet was used.

6.4 Surface Measurements

Surface measurements, if carried out with the necessary degree of precision, provide information as to the direction and rate of movement of the ground surface after completion of the embankment. The most difficult problem is often the establishment of a reference base line on firm ground located outside the area of movement. Since this distance may be considerable, normal survey errors often exceed tolerable limits.

Whenever possible, movement stakes should be set on a direct line of sight between stable bench marks. As movement develops, the offset of these stakes from the line of sight must be measured with precision. At Infiernillo Dam specially designed vernier gauges with targets were used to measure transverse movements of the crest. Where the crest is curved, as at Netzahualcoyotl Dam, precise measurements are made more difficult and triangulation may be the best approach. A good discussion of surface-movement techniques is given by G. Oberti (1964).

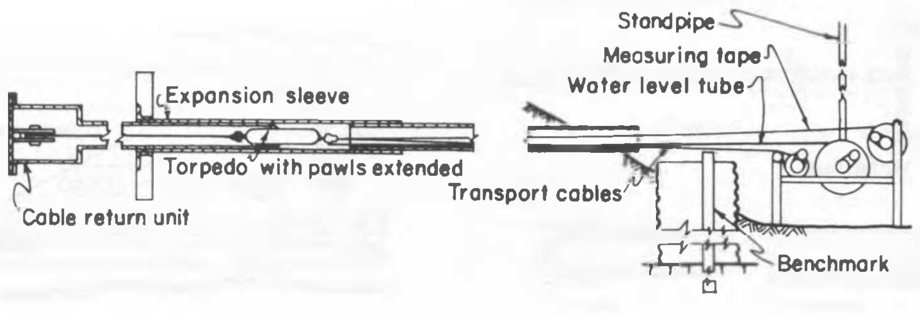


Fig. 49 Horizontal Strain Gauge, Fall Creek Dam

Fig. 50 Inclinometer for Detecting Horizontal Ground Movements

It is often necessary to locate the zones in which tension or compression is developing by means of surface surveys. This is especially true in cases of incipient slope instability. Where movements are moderate, spring-loaded, above-ground wires are used by the Walla Walla District Corps of Engineers as shown on Fig. 51. When more precise measurements are desired, such as in hillside creep, the wires are placed underground in plastic tubes and connected under constant tension to single potentiometer units similar to those previously described. To compensate for temperature variations, thermocouple wires may also be placed in or adjacent to the tubes. Several such units are connected back to back to monitor the performance of long slopes. These devices can also incorporate circuits to give automatic warning in the event movements exceed any preset amount.

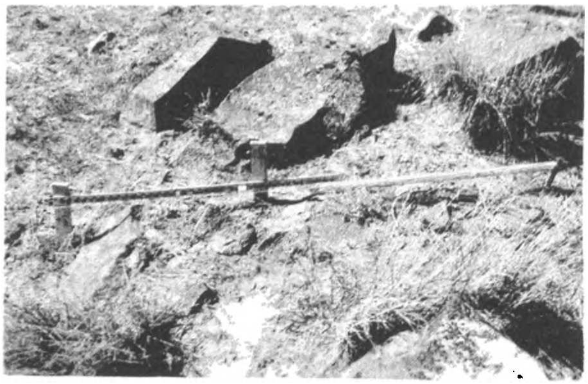


Fig. 51 Inexpensive Device for Detecting Surface Horizontal Movements

6.5 Stress Measurements

Earth-pressure measurements involve either pressures against structures or the free-field stress within an embankment. Reliable instruments and devices for measuring pressures against structures exist, Bishop et al (1961), Kruse (1965) and Kaufman and Sherman (1964). The most serious problem associated with the measurement of free-field stresses in soils is the stiffness of the gauge itself. If it is too stiff, the gauge will be subjected to stresses greater than those in the surrounding soil; the converse is true if the gauge is more compressible than the soil. In addition, it is difficult, if not impossible, to place the packing material around the gauge to have the same properties as the rest of the embankment. The ideal gauge, therefore, is of larger diameter to minimize local stress concentrations and of such thinness as to minimize stiffness effects. Attempts to develop such a gauge have not been too successful because large diameter, thin gauges are especially susceptible to shear stresses (which may register as normal stresses because of membrane distortion) temperature effects, and line leakage and expansion. Recent developments at the U. S. Army Waterways Experiment Station at Vicksburg, Miss., are described by J. K. Ingram (1965).

The thin rectangular pressure cells installed at Gepatsch Dam (Lauffer and Schober, 1964) and (Schober, 1967) and, Carters Dam (Engineering News-Record, 1966) appear to provide reliable data. Fig. 52 shows the cell, which consists of a 20-cm-by-30-cm pressure pad with a stiff rim. To measure the pressure exerted on the pad, the hydraulic pressure in the pressure line is slowly increased. When the pressure in the line equals the pressure on the cell, the diaphragm opens, bypassing hydraulic fluid to the cell return line. The pressure at which bypass occurs is indicated by appropriate measuring devices. This pressure, after corrections for temperature, elevation differences and others are made, is the pressure acting on the cell.

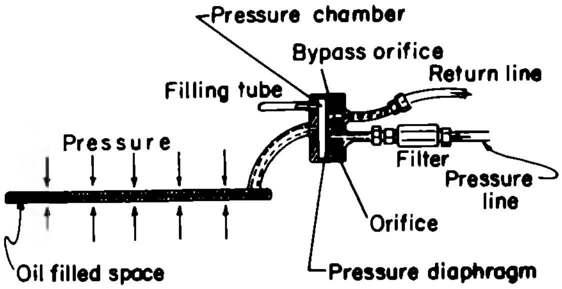


Fig. 52 Sketch of Gloetzl Pressure Cell

A new approach to the design of pressure cells has been tried at Oroville Dam. As described by Thayer (1966) this cell is designed to record transient stresses resulting from seismic shocks. Each cell is 5 inches thick, 30 inches in diameter, and weighs approximately 1,000 lbs. Fig. 38 shows the network of dynamic stress cells in the downstream shell of the dam. The cells were installed in arrays of three cells each at depths as great as 400 feet. A unique feature of the cell is that its modulus of deformation (elasticity) can be adjusted by pressurization of the cell fluid. To match the modulus of the cell with that of the soil, stress-strain data from triaxial compression tests were evaluated. On this basis the initial fluid pressure in the cell was made equal to 125 psi. Although not shown in Fig. 38, 27 other smaller stress cells were placed in the core, transitions and downstream shell to measure static stresses.

6.6 Earthquake Recorders

The effects of earthquakes on earth and rockfill embankments are not fully understood. To gain better insight into such effects, it is essential that records of acceleration be obtained at various locations on the embankment as well as at locations nearby, but removed from the embankment. This requires three-component, self-contained units which are actuated when a certain level of acceleration is obtained, and which then record for at least several minutes. At Infiernillo Dam, five such devices were installed; three on the dam itself, one on the right bank, and one in the underground powerhouse on the left bank. Typical recordings from a moderately strong earthquake are available (Marsal and Ramirez de Arellano, 1967). These five recorders were supplemented by 21 seismoscopes developed at the Institute of Engineering, National University of Mexico. Seismoscopes are low-cost auxiliary devices that record maximum displacement in all three components of an actual physical system of known dynamic properties. They supplement, but do not take the place, of strong motion recorders.

An advanced type of accelerometer is being installed for the first time at Oroville: three near the crest of the dam and one more in the foundation as shown in Fig. 38. Among other functions, the network of accelerometers is also designed to trigger both the dynamic stress and the hydrodynamic pore pressure monitoring systems during an earthquake. Fig. 53 shows the recorder and the accelerometer device: the latter was developed by the missile industry for inertial guidance systems. This particular installation has a frequency response from 0.1 cps to 20 cps and can be set to trigger and record automatically over a wide range of energy levels from 0.001 g to 1.0 g. The model shown in Fig. 53 is designed to be buried directly in the fill; others are available for placement in NX boreholes.

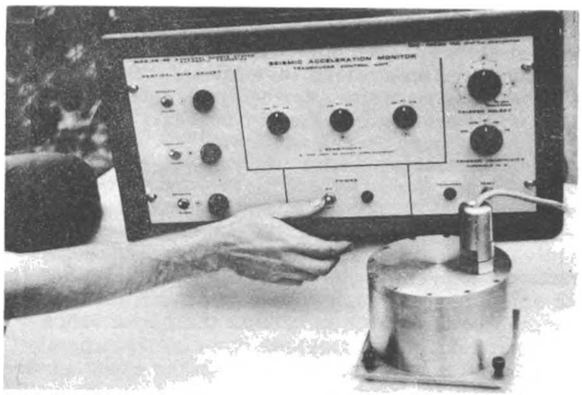


Fig. 53 Strong Motion Earthquake Sensing Unit And Recorder for Use at Oroville Dam

6.7 Additional Comments

Many reliable instruments and techniques are available to investigate and monitor the performance of embankments. Rugged instrumentation, which will give reliable data over an extended period of time, is preferable to sophisticated systems that may have a brief life. Experience has shown that the installation of instrumentation does not unduly hamper contractor operations and, with a proper approach, damage to instrumentation can be held to a minimum. Considering the vast advantages to a well-planned and thorough scheme of instrumentation, the costs are small, varying in general, from 0.1 percent to as much as 1.5 percent of the total cost of the embankment alone, depending upon the problems present.

At Angostura Dam in Mexico, the proposed instrumentation is shown in Fig. 54. This instrumentation, more comprehensive than that usually installed, is planned to provide insight into the distribution of stresses and strains within a modern rockfill dam.

7. DEFORMATION OF EARTH AND ROCKFILL DAMS

7.1 General

Observation of internal settlements and of pore pressures during construction have been made by the U.S. Bureau of Reclamation for at least 30 years. Post-construction settlement observations of the crest and occasionally horizontal measurements have been made over an even longer span of time by many agencies and engineers. However, it is only during the past decade that detailed and systematic observation of internal movements during construction has been undertaken on earth and rockfill dams. A few examples of typical data from a selected few of these dams is given in this section.

7.2 Mammoth Pool Dam

Mammoth Pool Dam (Fig. 14) on the San Joaquin River (Engineering News Record, 1960) in California is a zoned, rolled earthfill dam of disintegrated granite, about 360 feet in height, with steeply rising, smooth convex-shaped abutments of granite. Because of concern over slippage and possible seepage along the contact between the rolled embankment and the smooth rock abutments, instrumentation was installed to measure possible movement at this interface. Fig. 55 shows the overall distribution of vertical and horizontal movements above the left abutment, and Fig. 56 shows the detailed movements near the interface. It was found that there was zero displacement at the interface, with maximum shear strains developing at least 5 feet away from the abutment.

7.3 El Infiernillo Dam

El Infiernillo Dam (Fig. 8) is a major rockfill dam with a narrow central core, located on the Balsas River about 200 miles southwest of Mexico City. The dam is located in a narrow gorge and a comprehensive system of instrumentation was installed to measure strains and the three components of displacements in the core and rockfill during and after construction. The rockfill is primarily a sound diorite from quarry operations.

The performance of El Infiernillo Dam and a detailed description of the instrumentation has been given by Marsal and de Arellano (1963, 1964, 1965, 1967). Squier (1967) has reviewed in detail the deformations and probable mechanism of load transfer. The cross-valley horizontal movements and the vertical movements of the core are shown in Fig. 57, and comparative compression curves of the core and rockfill are shown in Fig. 58. It is significant that the rockfill (compacted in layers but not watered) was as compressible as the highly plastic clay core.

The first saturation of the upstream rockfill was abrupt when the reservoir overtopped the upstream cofferdam, causing substantial settlements upstream from the core. The crest deflected initially in an upstream direction, followed by a reversal and subsequent downstream movements. These crest movements are shown in Fig. 59 and the distribution with depth in Fig. 60. Minor surface cracking of the crest occurred shortly after reservoir filling started. Subsequent longitudinal strains of the crest although larger (up to 0.3%) did not cause further cracking (Fig. 31). Squier (1967) concludes that there was a transferral of load from the upstream shell to the core when the shell settled during the filling of the reservoir, but unfortunately this cannot be verified by instrumentation. The post construction settlements and movements of a point on the crest are shown in Fig. 61.

7.4 Netzahualcoyotl Dam

This rockfill dam is situated in a narrow gorge of the Grijalva River in the State of Chiapas, Mexico. Its general features and performance have been described by Gamboa and Benassini (1967). The wide central core of this 450-foot high dam is of reddish lateritic soil and the rockfill shells were of weakly cemented conglomerate which broke down during blasting and placing to give the appearance of a poorly graded sandy gravel.

The vertical compressibility of the core and of the rockfill, as measured by the telescoping casing of the inclinometers are shown in Fig. 62. Note that the conglomerate is slightly more compressible than the quarry rock at El Infiernillo, but that the lateritic core of low plasticity is less compressible than the highly plastic clay used at Infiernillo.

Longitudinal strains of the crest of Netzahualcoyotl Dam are shown in Fig. 32 for the first two post-construction years.

7.5 Muddy Run Embankment

The Muddy Run Pumped Storage Hydroelectric Project, located on the east bank of the Susquehanna River in Pennsylvania, includes a 250-foot high embankment of micaceous schist with vertical central core of low-plasticity residual soils. Wilson and Marano (1968) describe extensive field tests prior to construction and the performance of the completed embankment.

Of primary concern was the anticipated high compressibility of the rockfill and its effect on the performance of the structure. The in-place compressibility of the rock as contrasted with that of the core is shown in Fig. 63. Comparative vertical movements

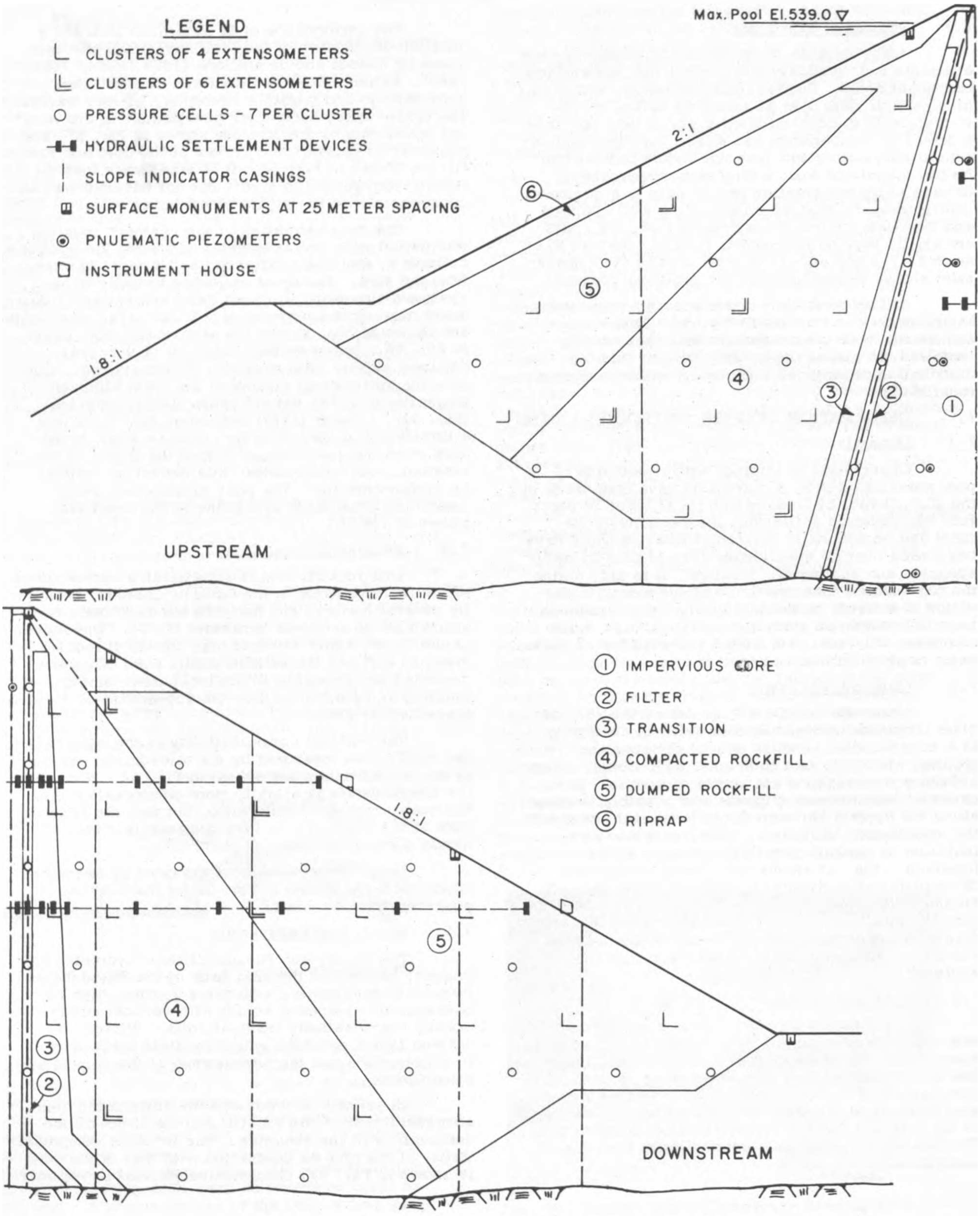
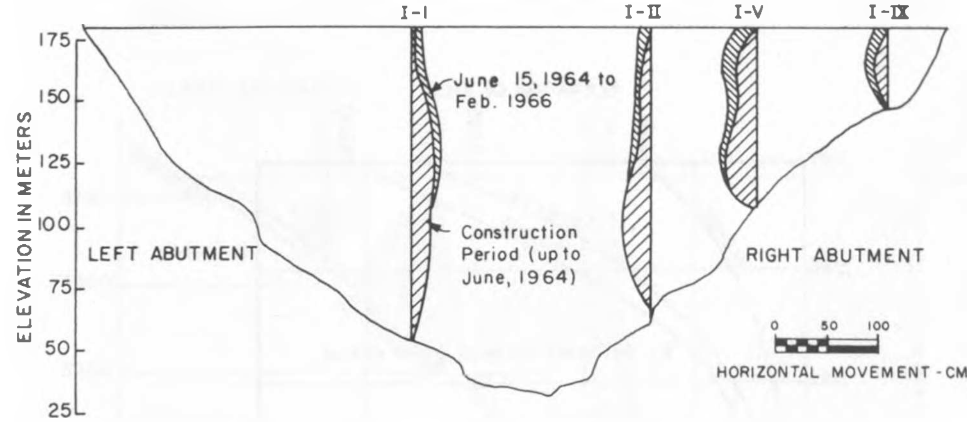


Fig. 54 Instrumentation Proposed for Angostura Dam, Mexico



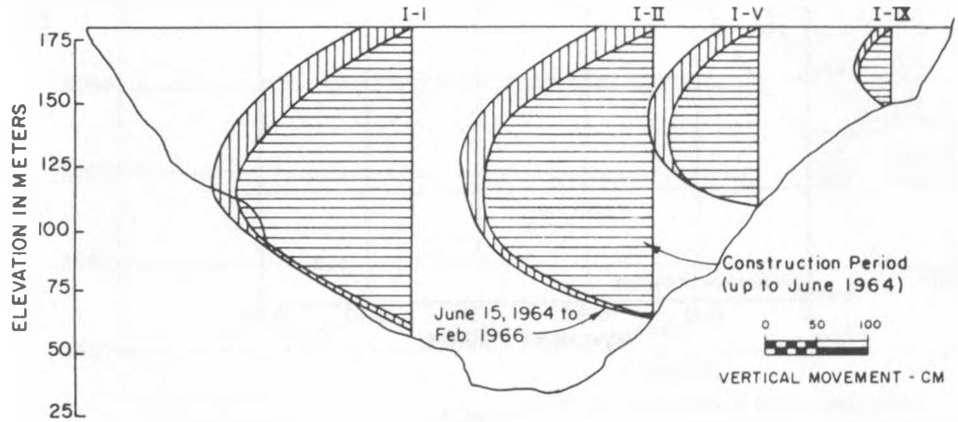


Fig. 57 Cross-Valley and Vertical Core Movements, El Infiernillo Dam

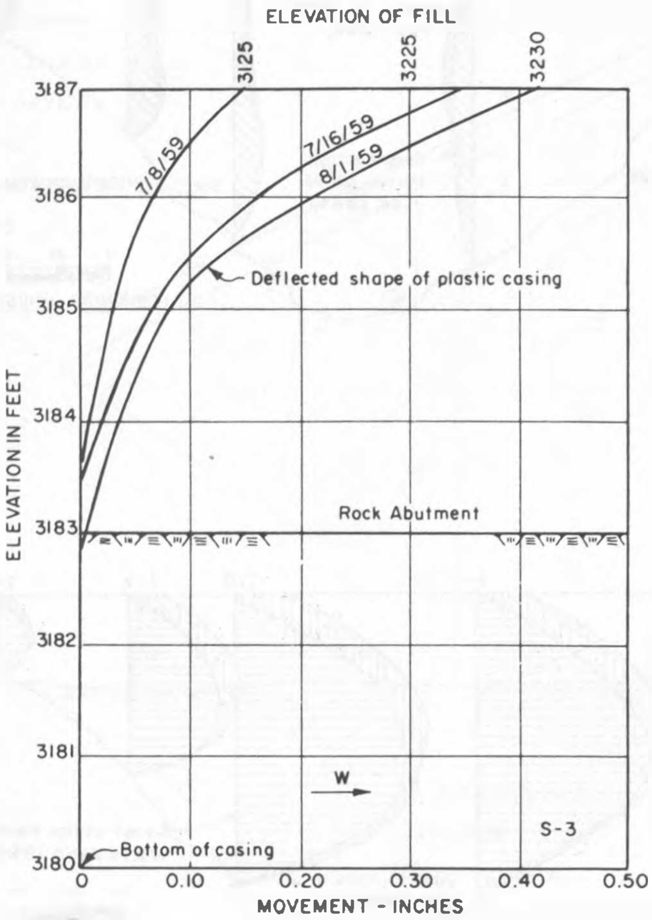


Fig. 56 Horizontal Movement of Embankments At Contact With Rock Abutments, Mammoth Pool Dam

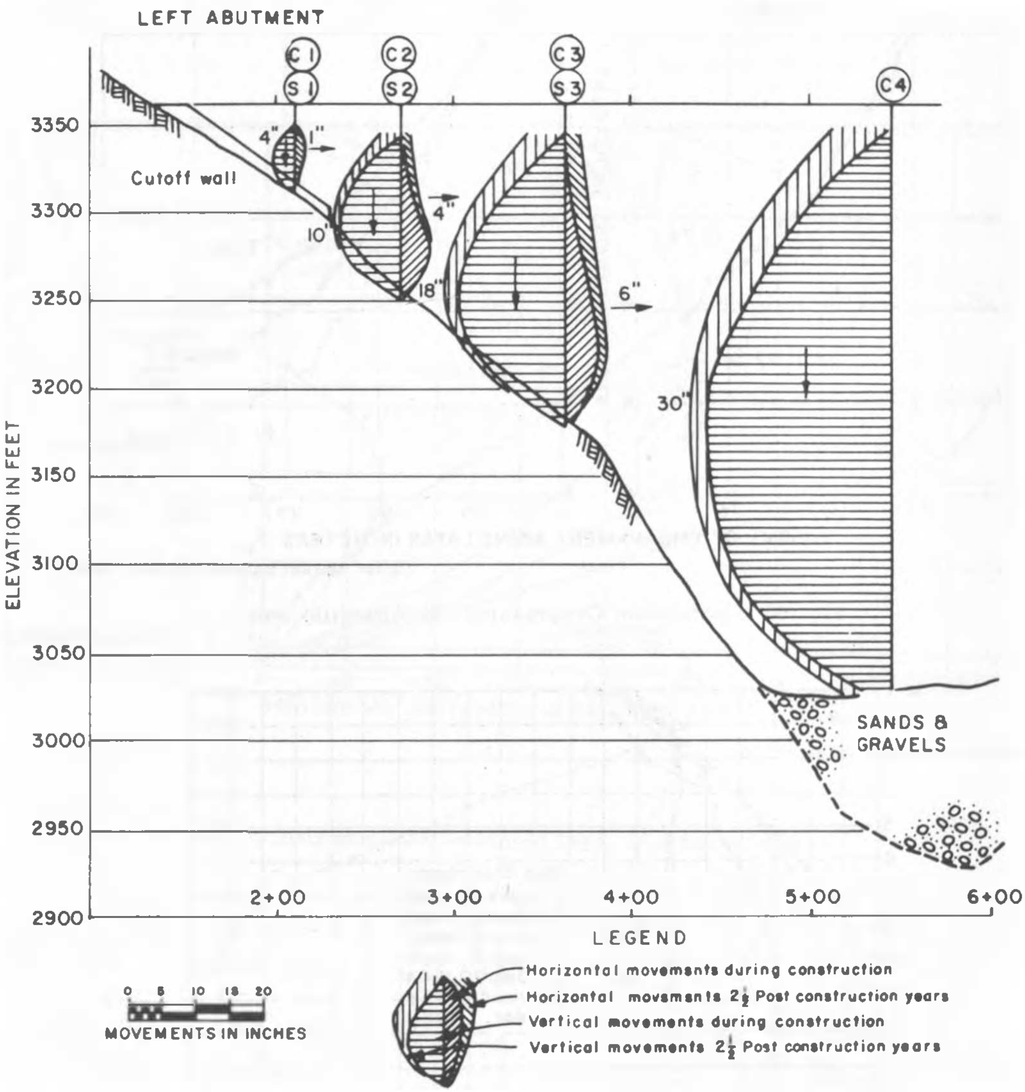


Fig. 55 Horizontal and Vertical Movements, Mammoth Pool Dam

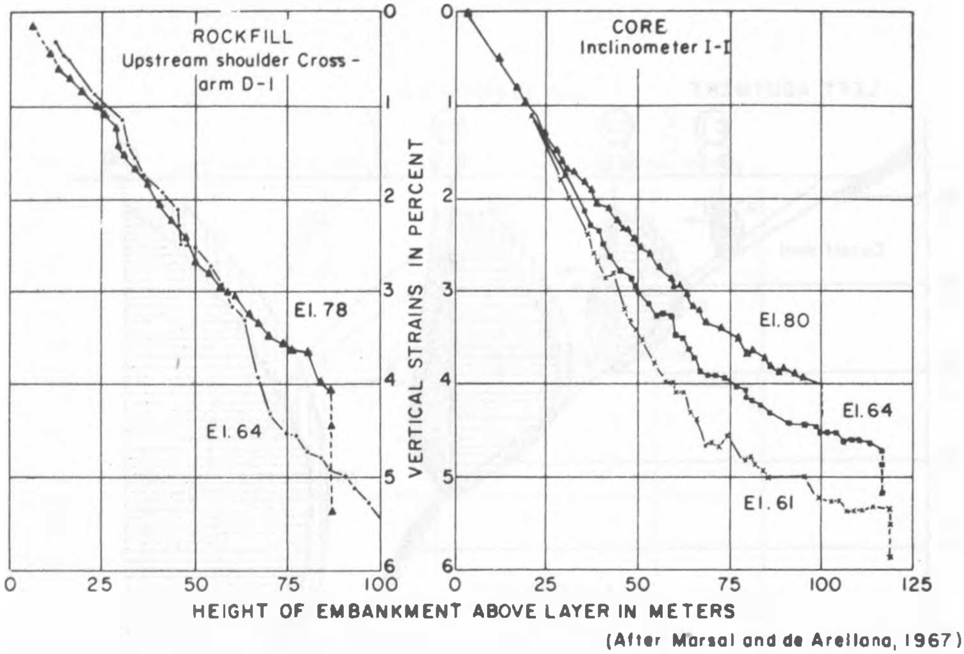


Fig. 58 Embankment Compression, El Infiernillo Dam

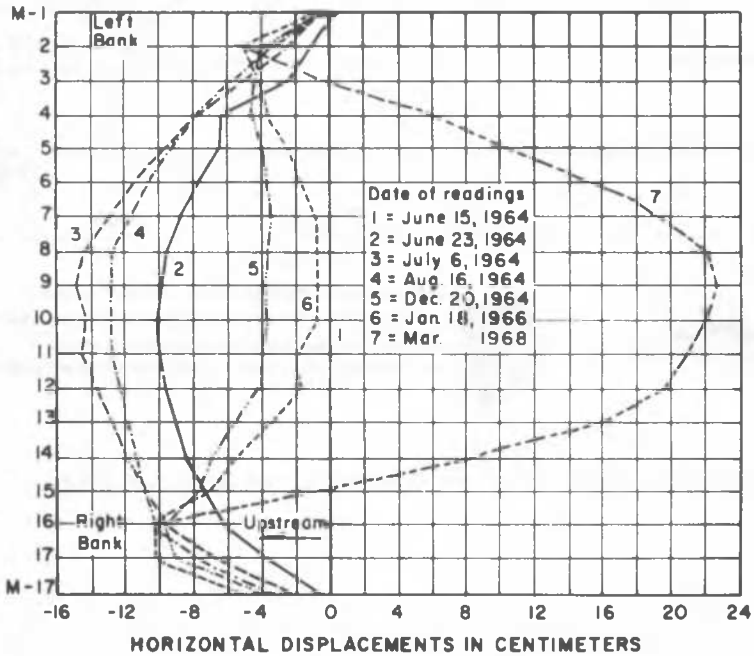
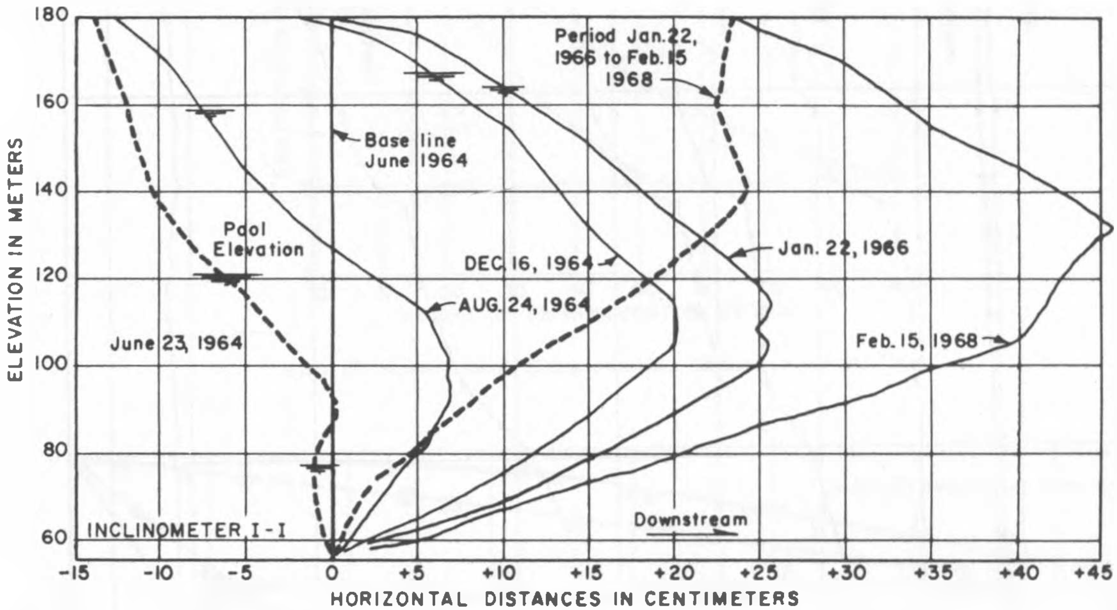


Fig. 59 Post Construction Deflection of Crest, El Infiernillo Dam



(After Marsal and de Arellana 1967)

Fig. 60 Post Construction Deflection of Core, El Infiernillo Dam

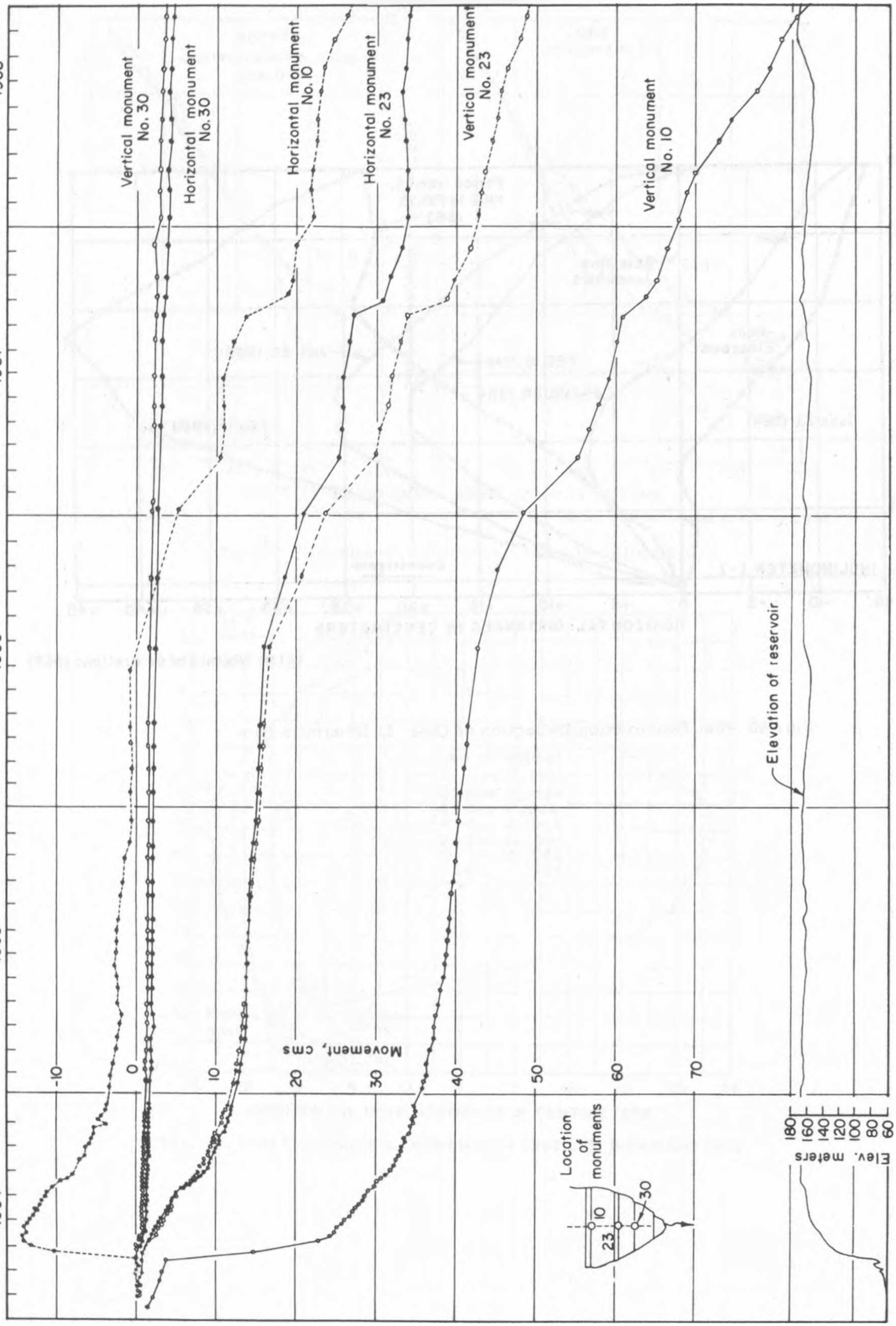


Fig. 61 Post-Construction Movements, El Infiernillo Dam

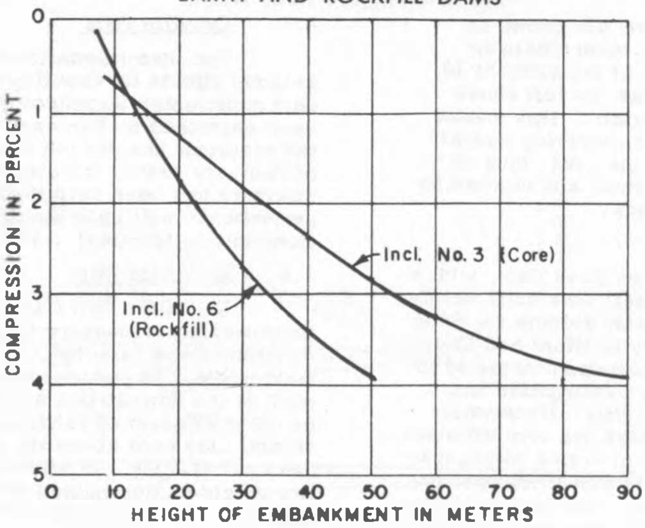


Fig. 62 Embankment Compression, Netzahualcoyotl Dam

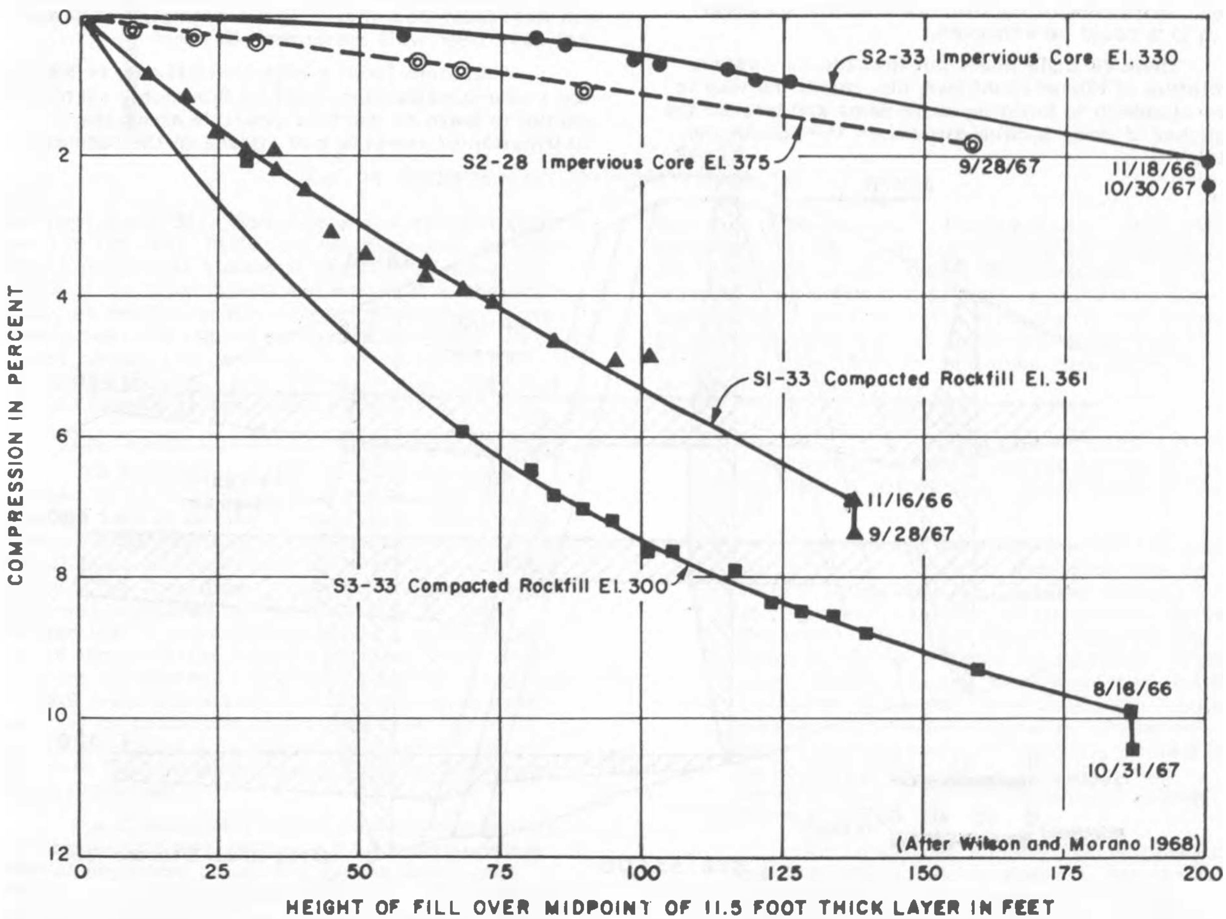


Fig. 63 Embankment Compression, Muddy Run Project

(settlements) of a typical section are shown in Fig. 64. It is obvious that the shear strain in the filter and transition zones, of the order of 10 percent, is sufficient to mobilize the full shear strength of these granular materials. This means that a substantial portion of the overlying weight of the shell was transferred to the core, thus decreasing the settlement of the shell and increasing that of the core near the interface.

7.6 Plover Cove Dam

The 6800-foot long Plover Cove Dam, with a maximum height of about 130 feet, was built mainly under water to provide fresh water storage for Hong Kong and has been described by Guilford and Chan (1969). The extensive instrumentation installed to monitor the performance during construction has been described by Dunnicliff (1968). Underwater earth dam construction techniques are very different from conventional methods and give rise to unusual problems. Extensive and costly site investigations, including large-scale field tests, are fully justified. At Plover Cover the performance was satisfactory even though segregation during placing of underwater fill increased the permeability of the core. This core, of decomposed rock, was raised in 3-foot thick layers placed by lowering 22 cu. yd. buckets through the water. With careful control, side slopes as steep at 1:4 or 5 could be achieved.

There is little doubt but that the successful completion of Plover Cove Dam has paved the way to a new approach to building earth dams and may be the forerunner of many similar structures throughout the world.

7.7 Oroville Dam

The instrumentation for pore pressures and seismic effects for Oroville Dam, a 770-foot high dam constructed largely of dredger tailings, has been described by Perry and Kruse (1968). Although the reservoir has not yet been completely filled (as of February 1969), the overall performance of the structure has been satisfactory. An analysis of the performance will be presented at the 10th ICOLD Congress in Montreal, 1970.

7.8 La Villita Dam

La Villita Dam (Fig. 23) was thoroughly instrumented to measure the performance of the deep concrete cutoff (see Par. 4.4.4) and of the embankment itself. Of particular interest was the settlement of the downstream shell which was supported by up to 300 feet of sands and gravels and of the central clay core above the grouted zone and the deep cutoff. Fig. 65 shows the settlements at two levels as determined by hydraulic settlement devices. The beneficial effect of grouting the upper stratum of sands and gravels adjacent to the cutoff wall is clearly evident.

7.9 Miscellaneous

Gepatsch Dam (Fig. 19), Carters Dam (Fig. 36) and Akosombo Dam (Fig. 17) are examples of dams that have been well instrumented.

Angostura Dam, a high rockfill dam in Mexico now under construction, will be thoroughly instrumented to learn as much as possible about the distribution of stresses and strains in the core and

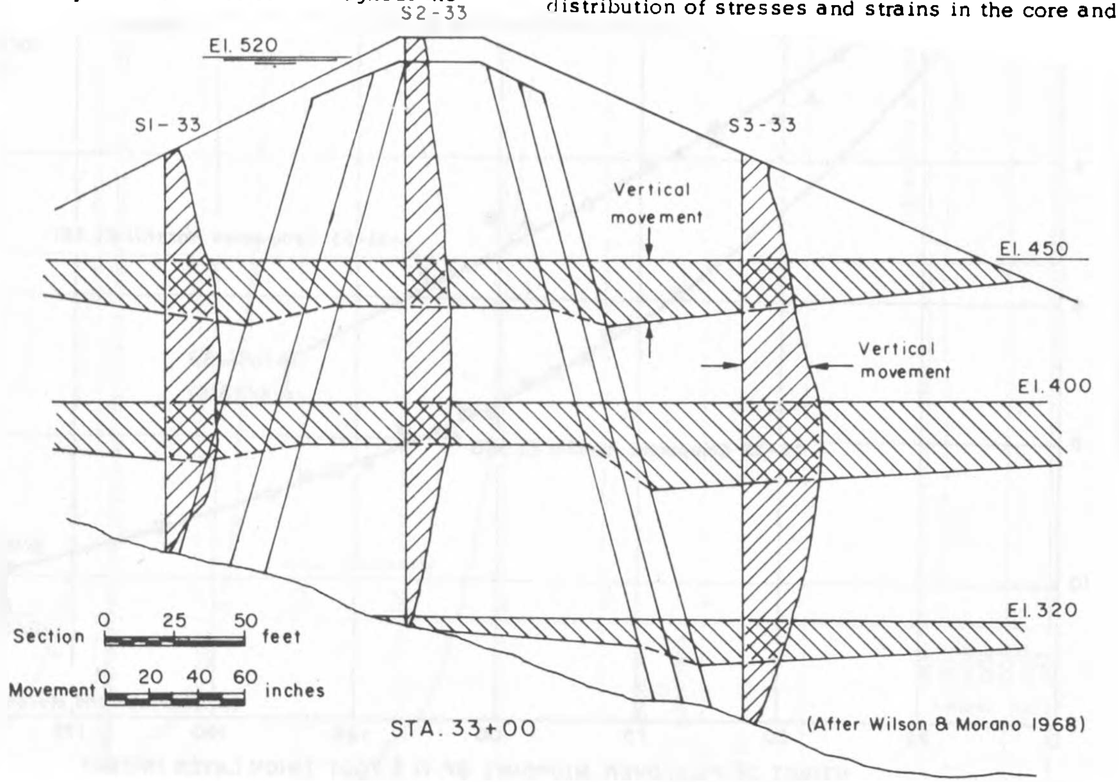


Fig. 64 Vertical Movements During Construction and First Filling Of the Reservoir, Muddy Run Project

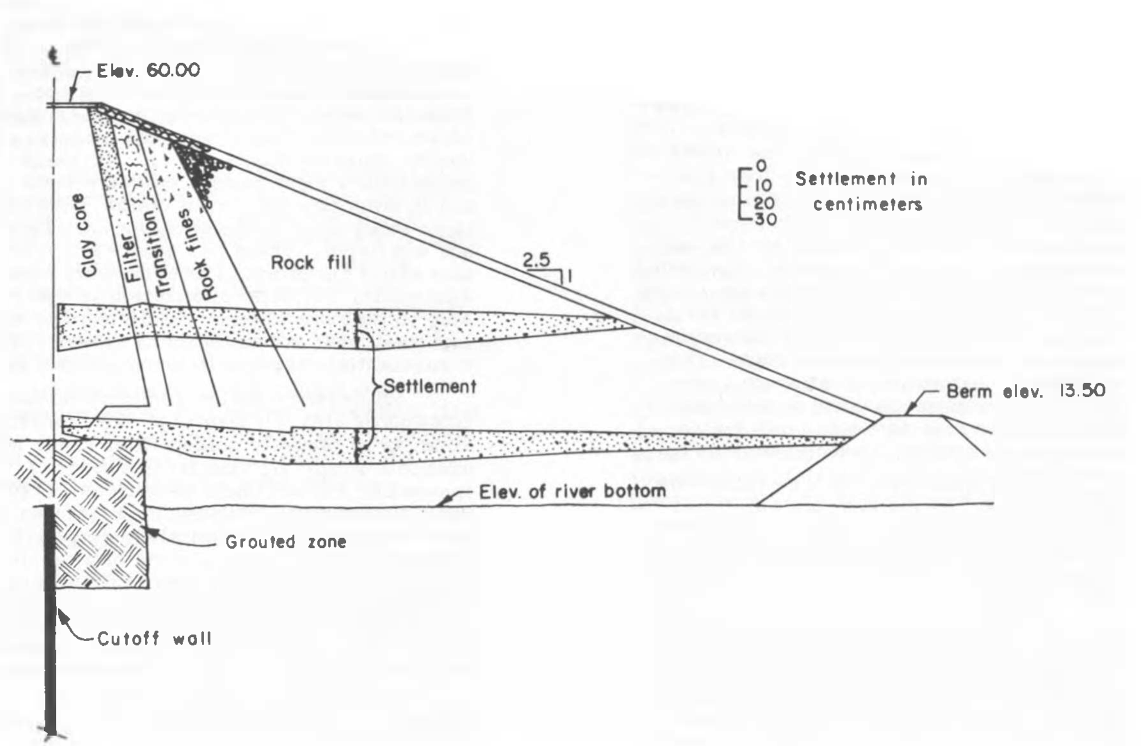


Fig. 65 Settlement of Downstream Shell, La Villita Dam

downstream rockfill. The proposed instrumentation is shown in Fig. 54. Particular attention will be paid to the longitudinal strains in the core in the upper portion of the embankment and adjacent to the abutments, as well as to the differential vertical movements across the filter and transition zones. Clusters of total pressure cells and of extensometers for measuring strain will be installed at systematically spaced intervals in the downstream shell.

8 PROBLEMS IN DESIGN AND CONSTRUCTION

The American Society of Civil Engineers assigned to its Committee on Earth and Rockfill Dams the task of making a compilation of problems concerning the design and construction of earth and rockfill dams on which more information is urgently needed. The Committee prepared a Progress Report which was published by the ASCE (1967). They selected the 10 most vital problems and tabulated them in their relative order of importance as determined by its members. For each problem, methods of attack were appraised under the following headings: (a) by collection of existing data; (b) by theoretical analysis or laboratory research; (c) by full scale field observations; and (d) by defensive design procedure.

The Committee's views were assembled in a first draft which was reviewed by a representative group of engineers practising in this field, both within and outside the United States. The general agreement reached by most of the individuals as to the vital problems and to the order of priority of these problems is in accord with the views of the General Reporter, who was one of the reviewers. With the permission of the Chairman of the ASCE Committee, the ten problems selected by the Committee, in the order of their suggested priority, are listed below together with the Committee's comments. It should be noted that this list includes only embankment problems. The following is quoted directly from the ASCE Journal (1967).

"1. - STRENGTH AND VOLUME CHANGE CHARACTERISTICS OF GRAVEL AND ROCKFILL MATERIALS UNDER HIGH CONFINING PRESSURES

(a) Triaxial test data for soil and rockfill of various types are available from the United States Bureau of Reclamation and Corps of Engineers, the State of California, Harvard University, Tippetts-Abbett-McCarthy and Stratton, Harza Engineering Company, Woodward, Clyde, Sherard and Associates, British Columbia Hydro and Power Authority, Tiroler Wasserkraftwerke AG (Austria), Comision Federal de Electricidad (Mexico), Secretaria de Recursos Hidraulicos (Mexico), and Versuchsanstalt fur Wasserbau and Erdbau (Switzerland).

(b) Laboratory tests appear to provide the best means of investigation, but it would be worth while to supplement them with fundamental theoretical research into the behavior of granular materials. The possibility of predicting stress-deformation and strength properties of coarse gravel and rockfill materials on the basis of tests on small particle sizes with extrapolation to full scale values merits investigation. The new triaxial machine of the Comision Federal de Electricidad in Mexico, having a sample diameter of more than 1 m, should make possible tests of great value on the effect of maximum particle size in high load triaxial devices. The term "high confining pressures" is related to the material; these pressures will be lower for rock that is soft or weathered, but that can not be disregarded for use. Before- and after-testing grain size analysis of materials used in triaxial tests is needed, to indicate how much disintegration occurs for various types of rock. The time effect of saturation on strength, consolidation, and pore pressure development in the stressed rock particles, and the sequence of application, need further study.

(c) Analyses of rock talus avalanches (termed "Muren" in the Austrian and Swiss Alps), taking into account their relative density, grain size distribution, and thickness of deposit, should prove to be of significant value.

(d) Defensive design procedures include the use of conservative values for the angle of internal friction, and a careful consideration of possible decrease in both shearing strength and permeability resulting from breakdown in grain size because of the high loads imposed. In general, flatter slopes are indicated for high dams until investigations justify slopes comparable to those currently used on lower dams. This is corroborated by the results of triaxial tests performed at El Infiernillo and confined compression tests performed at the National University of Mexico. It was found that the principal stress ratio at failure decreases substantially as confining pressures increase from 0.5 to 25 kg per sq. cm.

2. - CRACKING WITHIN EMBANKMENTS

(a) Possible sources of information are: Bureau of Reclamation, Corps of Engineers, Comision Federal de Electricidad, Secretaria de Recursos Hidraulicos, and others.

(b) Laboratory model research on this problem is considered to be of questionable value at present; however, any laboratory data on the stress-deformation and strength properties of various types of material in the dams in which cracks were observed should be useful for establishing design criteria. Laboratory research is needed to study relative susceptibility to cracking, and self-healing properties of embankment materials. It would be useful to devise standard tests for assessing the elongation deformation at failure and the self-healing characteristics of the material after cracking. Information is also needed on the change in plasticity of embankment materials with time.

(c) Field observations of the progress of visible cracking in dams should be made. However, the most dangerous cracks in the interior of an embankment may not be discovered until piping failure takes place. Potential means of discovery are piezometers, slope indicator tubes, and Idel tubes in zones where cracking might occur. In-place permeability tests and air pressure tests may aid in detecting cracks, if control observations made prior to the development of cracks are available. There has been much speculation about distortion of embankments near the abutments, but little information is available to indicate whether the distortions are localized near the abutment or spread out for a substantial distance from the contact fact.

(d) Defense design and construction procedures are: (1) Use of a wide transition zone, or of properly graded filter zones of adequate width; (2) special treatment of foundation and abutment conditions to reduce sharp differential settlement; (3) arching the dam horizontally between steep abutment slopes; (4) adjustment of construction sequence for the different zones or sections; (5) requiring special placement methods for questionable materials; and (6) thorough compaction of rock shells to avoid inducing tensile stresses in adjacent core material.

3. - COMPACTION METHODS IN COARSE GRAVELS AND ROCKFILLS

(a) Data are available from the Bureau of Reclamation, Corps of Engineers, State of California, Union Electric Company, Electricite de France, Tyrolean Hydro-electric Company, and from corresponding organizations in Brazil, Sweden, and Germany.

(b) Theoretical analysis and laboratory research are not applicable to the problem.

(c) Evaluation of compaction methods can be made by comparison of field densities with laboratory vibrated materials during construction, and by measuring decrease in thickness of layers in field compaction tests. Settlement measurements of shell materials can be made during and after construction. The remote settlement device working on the principle of communicating vessels is useful in rock embankments.

(d) Defensive design is not applicable.

4. - SLIDING FACTORS FOR ROCKFILL ON FOUNDATION CONTACT SURFACES

(a) Some information on friction tests is available from Djatiluhur Dam (Coyne et Bellier, Paris), and Goscheneralp Dam (Versuchsanstalt fur Wasserbau and Erdbau, ETH, Zurich). Data on movements of rockfill in the vicinity of the foundation rock are available from these organizations, and from Comision Federal de Electricidad and Secretaria de Recursos Hidraulicos.

(b) Theoretical analysis is not a promising approach. Laboratory experiments would have to be made on a model scale to be practical, but care must be taken to assure that particle sizes used provide a degree of roughness and interlocking equivalent to that anticipated in the prototype.

(c) Instruments to measure vertical and horizontal movement of rockfill near the contact surface should be installed wherever feasible. Slope indicator tubes and Idel tubes may be used effectively for this purpose. Large scale field tests to determine the resistance to sliding of rockfill on foundation contact surfaces are feasible, and they are recommended.

(d) Roughness of the contact rock surface equivalent to that within succeeding lifts of the embankment may be assumed to provide equal frictional resistance, provided the foundation rock is not a weak material.

5. - PREDICTION OF PORE PRESSURES IN COMPACTED COHESIVE SOILS

(a) Pore pressure data are available from the Bureau of Reclamation, Corps of Engineers, and private agencies in the United States. In addition, data can be obtained from several European countries, Australia, and Brazil.

(b) Theoretical studies and laboratory investigations would be valuable if coordinated with actual projects on which extensive and reliable pore pressure measurements are made. Consideration should be given to the problem of identifying the maximum hydraulic gradient that is acceptable through a core.

(c) Observations of pore pressure in dams should be extended, and the relative merits of various methods for measuring pore pressure investigated by parallel installations. A careful study is needed of the difference between pore-water pressure and pore-air pressure, and equipment should developed to measure both pressures. More attention should be given to the measurement of pore pressures in the upstream portions of cohesive zones or the core which are influenced by fluctuations in reservoir level. Settlement gages should always be installed in conjunction with pore-pressure measuring devices.

(d) Defensive design procedures include the use of relatively narrow impervious cores in zones dams, careful selection of materials for embankments, control of placement water content, and use of drainage layers and zones. A compromise must be found between compaction moist enough to secure a core that will not be liable to cracking, and compaction which is not so wet that pore pressures will be excessive.

6. - DYNAMIC BEHAVIOR OF EMBANKMENTS IN EARTHQUAKE REGIONS

(a) Available data on performance of dams subjected to earthquake shock can be collected from California, Chile, Peru, Japan, Mexico, Turkey, and other seismic regions.

(b) The conventional methods for theoretical analysis of embankments under seismic loading are inadequate. Laboratory experiments are not likely to provide results that can be directly relied upon until observations of prototype phenomena permit data comparison. However, much can be learned about the basic behavior of different types of soil under dynamic conditions by laboratory tests. A systematic study of the effect of repetitions of strain (not necessarily dynamic) under undrained conditions on the pore pressures in sands of varying density and grain size would provide considerable insight into the factors governing loss of strength during earthquakes, Investigations should be made to identify the characteristics of materials that are liable to liquefy under the effect of pulsating stresses, and to define a "dynamic critical density."

(c) Accelerometers should be installed at both the base and crest of major structures. Some useful information may be obtained from an investigation of the stability of talus slopes subjected to shocks. A study should be made of reservoir slides, and the method of estimating resulting waves and their propagation through reservoirs.

(d) Defensive design procedures include special zoning, greater freeboard, and flatter slopes than those adopted for design in non-seismic regions. In addition, consideration should be given to a broader crest or higher freeboard, as a protection against overtopping of brief duration caused by seismically induced surges in the reservoir. Cores should be made thicker and of self-healing soils.

7. - STRESS AND DEFORMATION MEASUREMENTS IN EMBANKMENTS

(a) Data on reliable stress measurements are virtually nonexistent, because of the previous lack of an effective stress meter for use in earth or rockfill dams. Existing measurements are limited to those of differential vertical deformation, and total horizontal displacement on the outer slopes.

(b) In connection with the measurement of movements within rockfill, an instrument called the R. F. Cavity Extensiometer has been developed by the British Aircraft Corporation. It is based on the measurement of resonance at radio frequencies, and may have useful applications. A cell for the measurement of normal stresses has been built at the University of Mexico.

By arranging groups of six cells each in various parts of the dam, the distribution of stresses could be measured. Electro-acoustic stress meters manufactured by Oficina Galileo di Milano have been installed in several earth dams in Italy.

(c) Observations of deformations and stresses in dams should be encouraged. Use can be made of currently available devices for measuring vertical and horizontal deformations, and stress meters such as those mentioned above.

(d) Not applicable to this problem.

8. - CONTROL OF COMPACTION OF COARSE GRAVELS, COBBLES, AND ROCKFILLS

(a) Reports are available of field control tests from density measurements of Oroville Dam, California, Mica Dam, British Columbia, and Cougar Dam, Oregon. At El Infiernillo Dam, 20 field density tests were performed, each involving about 6 cu m of material. At Mattmark Dam, Switzerland, a few test holes of about 100 cu m were excavated. The large excavations were necessary because of the presence of individual blocks of 1/2 cu m and 3/4 cu m.

(b) Laboratory vibratory compaction methods should be standardized to the extend practical for evaluating effect of vibration by field compaction equipment. Laboratory procedures used at Oroville Dam (by the State of California) and Cougar Dam (by the Corps of Engineers) provide a basis for establishing standard procedures. The size of the mold, the method of placement, and the method of vibration should be related to the maximum size of particle in the material to be tested. For materials contaimining large elements, the accuracy of relative density is questionable, because of the difficulty involved in obtaining consistent measurements of the minimum density. It would probably be more logical to identify only a maximum density from a standardized procedure.

(c) Field density measurements using sand or water replacement are well known. Procedures should be developed, however, for holes several cu yd in volume. The minimum allowable size of hole should be related to the maximum particle size. A problem that deserves additional study is the question of the allowable percentage of fines that can be permitted in a gravel or compacted rockfill without adversely affecting the compaction characteristics of the material.

(d) Not applicable.

9. - SLOPE PROTECTION FOR EARTH DAMS

(a) The report on slope protection by the Corps of Engineers, published in 1949, should be brought up to date by incorporating the data accumulated by a large number of agencies having information on the subject.

(b) Criteria for upstream slope protection are being investigated by the Beach Erosion Board of the Corps of Engineers, using large scale models. The Hydraulics Research Station in England has issued a useful report on riprap for embankments, based on model studies. Laboratory studies of riprap substitutes have been made, such as those carried out by the Bureau of Reclamation on asphaltic concrete and soil cement. Properties of rock for riprap can be determined by petrographic procedures and concrete aggregate laboratory tests.

(c) Field inspection reports of slope protection should be carefully documented in order to be of maximum value. Where the need for maintenance is continuous, or where failures occur, the construction records should be analyzed along with the properties of the materials. A standard method for measuring riprap sizes in connection with both design and construction is needed. Criteria should be developed for evaluating beaching slopes that give adequate wave protection. Climatic and other data should be obtained for protecting downstream slopes by vegetation.

(d) Relatively flat slopes require small maximum size for riprap. Economics and the difficulty of placing very thin riprap layers govern the extent to which flattening can be substituted for rock size. Measures should be taken to collect and channel runoff on downstream slopes in such a manner as to minimize erosion. In desert areas free-draining pervious cover can prevent erosion from both wind and rain, and minimize the development of shrinkage cracks.

10. - LOW-COST ADMIXTURES FOR IMRPOVING SOIL CHARACTERISTICS

(a) Data are available on the use of bentonite, cement, and lime as admixtures for soils. Admixtures causing brittleness are undersirable. Generally, admixtures increase cohesion but not the angle of internal friction. An increase in cohesion is not of value in high dams. Bentonite and clays have been added to core materials solely to increase imperviousness. Soils from different sources have also been mixed to increase imperviousness.

(b) Laboratory research on materials with desirable properties may be profitable.

(c) Information is needed on the permanency of some admixtures, and on the extent to which the water at the site may affect their durability.

(d) Not applicable. "

The ASCE Report quoted above is limited to 10 high-priority problems relating to the embankment itself. Of these, Problem 7 relating to stress and deformation measurements in embankments appears to the authors in particular to be worthy of higher priority. In fact, many of the other problems may be at least partially solved by observing in great detail the behavior of embankments now under construction, or planned for future construction. A good example is the instrumentation planned for Angostura Dam (Fig. 54). The same instrumentation used for observing the performance of the embankment would also provide at least partial answers to other problems, including abutment and foundation treatment.

In the introduction to this paper it was stated that changes evolve slowly with respect to the design and construction of earth and rockfill dams. As in other fields, however, progress accelerates as knowledge increases. The historical development of earth and rockfill dams indicates three eras of progress, in each of which the gains probably exceeded the previous total sum of knowledge.

Prior to 1850	Empirical knowledge and rule-of-thumb procedures. Extensive use of rock masonry.
1850 - 1940	Development of new construction procedures and use of different materials such as hydraulic fill dams and dumped rockfill dams. Frequent failures.
1940 - 1968	Application of theory to design. Development of efficient earth-moving and compaction equipment. Effective construction control and supervision of dams.

The next decade will undoubtedly see higher and higher dams built, and safe dams designed and built as sites having marginal foundation conditions. Two questions, however, will always remain at least partially unanswered: 1) What is the maximum flood that will occur during the life of the project; and 2) What is the maximum earthquake that will occur during this same period? If the designing engineer knew the answers to the above questions he could design the dam to survive that occurrence. Since the answers at present depend on circumstances beyond the control and understanding of the engineer, the designer must still rely heavily on judgment and experience.

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RÉSUMÉ

HISTOIRE

A travers les années, les coupes des barrages à digue ont évolué progressivement des coupes homogènes de terre aux massifs à zones, ces derniers souvent composés d'assez grandes quantités de matériaux grossiers et granulaires, comme, par exemple, le gravier ou le remblai rocheux. Aujourd'hui, on ne construit que rarement les barrages de remblai hydraulique à cause, d'une part, de la difficulté des contrôles pendant la construction, et d'autre part, des nouveaux développements dans l'équipement de portage et de compactage de la terre, développements qui permettent qu'on économise dans le déplacement et le compactage de la terre et des matériaux de remblai rocheux. La popularité du barrage ordinaire à remblai rocheux basculé a diminué aussi, à cause des économies qui proviennent de l'emploi d'un ample assortiment de matériaux à propriétés variables, et à cause du développement d'un matériel de compactage qui améliore ces propriétés, les rendant acceptables aux besoins de la construction. Au présent on peut dire que la plupart des barrages à digue se composent de zones de terre ou de remblai rocheux, ou tous les deux, qu'on place en niveaux, les compactant avec un équipement convenable au compactage. Aux États-Unis, le Bureau de Réclamation montra le chemin, en applicant les principes de la mécanique du sol au dessin des barrages en terre vers 1941 et pendant les années suivantes. Chaque année la hauteur des nouveaux barrages augmente, et le nombre des barrages en terre et en remblai rocheux augmente en raison inverse du nombre de ceux qu'on bâtit en maçonnerie et en béton.
(Fig. 2.)

ÉTUDES SUR LE TERRAIN

Pour le dessin des barrages en terre, on se charge d'une variété d'explorations du site et d'études au laboratoire et au bureau. Ces explorations et études sont tout d'abord d'un caractère général, mais elles deviennent plus précises à mesure que des questions spécifiques se posent pendant les investigations.

Les études sur le terrain comprennent des levés géologiques, des investigations de l'eau souterraine et des contrôles divers du site. Depuis plusieurs années on utilise de plus en plus la photographie aérienne comme supplément des reconnaissances et des autres moyens d'exploration sous-sol. Afin de faciliter l'analyse sous-sol des détails de la structure du fond rocheux, l'emploi des appareils photographiques de sondage, des explorations par télévision, et des levés géophysiques devient toujours plus général aussi. Ces techniques servent de supplément de l'analyse visuelle des conditions géologiques, qu'on fait par le moyen de puits, de tunnels, et de fossés, sans vraiment remplacer cette analyse.

Depuis quelque temps, on étend de plus en plus les recherches jusqu'à embrasser non seulement le site du barrage, mais les réservoirs aussi, surtout depuis la catastrophe à Vaiont et à cause de la fréquence des comptes rendus des déplacements des masses en d'autres réservoirs, comme celui du barrage de Gepatsch, parmi d'autres exemples. On fait plus d'attention aussi au rapport apparent entre la fréquence accrue des tremblements de terre dans les environs d'un projet hydro-électrique et la première mise en eau d'un réservoir.

Les explorations du site aujourd'hui entraînent de plus en plus l'emploi des programmes d'essais sur le terrain. De tels programmes peuvent comprendre des épreuves dans les zones d'emprunt, par exemple, le sautage dans les carrières, ou en d'autres cas, le calibrage étendu des grains dans les dépôts granulés, comme on fit au barrage d'Oroville. Les programmes d'essais sur le terrain comprennent aussi des éxperiences tels que l'injection et des essais de cisaillement étendus. Ceux-ci comprennent parfois des essais de cisaillement directs, qui utilisent des échantillons type du matériau de la partie extérieure de la digue, soit-il le gravier ou le remblai rocheux; ou ils peuvent comprendre la torsion, ou des essais de cisaillement directs sur les matériaux de fondation enrobés, tels que les argiles schisteuses ou les matériaux de fondation qui ont des zones faibles de mylonite, de lignite ou d'autres minéraux.

On utilise de plus en plus les maquettes des remblayages comme une phase importante des études du dessin. Celles-ci, mieux que toute autre méthode, peuvent révéler les meilleurs procédés pour le placement et le compactage des remblais aussi bien que le résultat de ces procédés: le caractère distinctif du remblai achevé. On peut ainsi essayer les procédés qu'on emploie, tels que l'épaisseur du niveau, le genre du matériel de compactage, le nombre de passages, et ainsi de suite. (Fig. 6.) En plus des maquettes des remblais, on peut construire aussi des maquettes des digues, afin d'avoir des renseignements sur les réactions des digues et de la fondation aux surcharges qu'on leur impose.

En sus du procurement des renseignements utiles sur la practicabilité des procédés et des paramètres du dessin (renseignements qui ne sont pas accessibles, d'un point de vue pratique, aux essais au laboratoire), tous les genres d'essais sur le terrain fournissent des renseignements pré-contractuels très utiles à l'entrepreneur.

LES ESSAIS AU LABORATOIRE

On peut obtenir, des essais sur le terrain, des échantillons pour les épreuves au laboratoire. On fait ces épreuves afin d'avoir des renseignements spécifiques sur les propriétés des matériaux pour qu'on puisse évaluer leur tenue sous les conditions de chargement et d'infiltration qui se produisent pendant le durée du projet. Il est généralement reconnu que la constatation de la résistance des matériaux et de leur variation avec le temps est un aspect difficile mais d'une importance vitale dans le dessin des barrages en terre. Au présent, on fixe d'ordinaire la résistance au cisaillement par des essais triaxiaux faits sous trois modes différentes de chargement et de drainage. Les trois modes, ou trois essais, sont (1) l'essai (R) de compression triaxial sans drainage, ou rapide sur échantillon consolidé; (2) l'essai (Q) de compression triaxial sans drainage, sur échantillon pas consolidé; et (3) l'essai (S) triaxial drainé. Pour des sols cohésifs, les paramètres de résistance des essais (Q) s'utilisent souvent pour évaluer la stabilité de la digue pour les conditions à la fin de la construction; on emploie l'essai (S) pour constater la stabilité de la digue sous la condition d'une infiltration continuelle pendant quelque temps; et les essais (R) ou ($\bar{R}$) servent à évaluer la stabilité sous les conditions d'affaissement rapide. Bien sûr, en pratique, il y a diverses voies d'accès à une évaluation de la stabilité de la digue, et il est bien possible qu'il soit convenable d'apporter des modifications à l'énoncé général écrit ci-dessus sur les procédés courants.

A l'égard des matériaux granulés, on fait de plus en plus attention aux propriétés de résistance et de changement de volume du gravier et du remblai rocheux sous la condition d'une grande pression qui les resserre. Depuis 1955 environ, jusqu'au présent, les essais démontrent que pour de certains matériaux granulés, la raison de contrainte principale à la rupture décroît à mesure que la pression resserrante augmente. Comme Marsal et d'autres personnes remarquèrent, le phénomène de la réduction de la résistance au cisaillement accompagnée par un accroissement de pression resserrante se rapporte à la fragmentation ou l'écroulement des grains dont se compose l'échantillon (Fig. 10). Ce rapport a des résultats évidents sur la stabilité des digues, surtout si on se rend compte de la tendance de construire les barrages toujours plus hauts. La Comité ASCE sur les Barrages en Terre et en Remblai Rocheux (1967) esquissa des mesures préventives dans le dessin, qu'on pouvait prendre à l'égard du phénomène d'une réduction de résistance au cisaillement avec une augmentation des pressions reserrantes, ainsi qu'il suit:

(1) Utiliser des valeurs conservatives pour fixer l'angle de friction intérieure, et considérer sérieusement la possibilité d'une diminution de la résistance au cisaillement et de la perméabilité, résultat d'un écroulement dans la dimension des grains à cause des grandes charges y imposées.
(2) En général, on recommande les pentes relativement aplaties pour les hauts barrages, jusqu'à ce que les investigations justifient des déclivités comparables à celles qu'on emploie couramment sur les barrages moins hauts.

LE DESSIN DE LA DIGUE

Au présent, diverses coupes existent pour les barrages à digues. Dans le cas d'une digue en terre, la digue peut se composer d'une seule zone qu'on doit fournir d'un système de drainage intérieur en forme de "cheminée", ou bien elle peut se composer de deux zones. De l'autre côté, une coupe de digue peut comprendre plusieurs zones, et peut se composer d'un extérieur granulé, d'une zone de transition, et d'un noyau de matériau impénétrable. Depuis peu d'années, un changement important survient à l'égard de l'emplacement des noyaux de terre dans les coupes de digues, surtout celles qui se composent de remblai rocheux. Autrefois on choisissait d'ordinaire ou un noyau vertical ou un noyau incliné, le premier étant situé dans la partie centrale du barrage, et le dernier étendu sur une déclivité de remblais rocheux basculés, en l'ordre de 1.35H:1V. Plus récemment, pourtant, bien des noyaux se situent dans une position intermédiaire de manière qu'on puisse les désigner "noyaux en pente modérée". En général, on peut appeler la position du noyau "en pente modérée" quand le pan du noyau qui se trouve à l'aval est d'une déclivité de 0.5H:1V environ. (On voit plusieurs exemples des situations des noyaux dans la Fig. 13.) A l'égard de la coupe finale de la digue, la division en zones dépend de beaucoup d'éléments, dont le plus important est la sûreté, en ce qui concerne la stabilité et l'infiltration; un deuxième élément important est l'économie. Les considérations financières comprennent la disponabilité et les propriétés physiques des matériaux régionaux, les prévisions de la construction, les problèmes de la fondation et du mur parafouille, aussi bien que les considérations de la dérivation. En conformité avec l'usage de l'U.S. Corps des Ingénieurs, les exigeances fondamentales du dessin, qu'il faut satisfaire afin d'assurer une structure convenable, sont les suivantes:

(1) Il faut que les pentes de la digue soient solides sous toutes les conditions de construction de d'opération, ce qui comprend l'affaissement rapide de la mare.

(2) Il ne faut pas que la digue impose des contraintes excessives à la fondation.

(3) Il faut qu'on règle le courant de filtration à travers la digue, la fondation et les culées, afin qu'il n'y arrive aucune érosion interne ni déplacement du matériau en solution. En outre, l'intention du projet

peut imposer des bornes à la quantité de filtration.

(4) Il faut que la partie supérieure du barrage soit d'une hauteur qui empêche que les vagues la surmontent, et qui tient compte de l'entassement de la fondation et des remblais.

(5) Il faut que la capacité du déversoir et de l'orifice de décharge suffise pour empêcher le dépassement de la digue par les vagues.

Quant aux énoncés (1) et (2) ci-dessus, on fait souvent des analyses de la stabilité. Ces analyses fournissent un moyen d'évaluer la marge de sécurité des coupes de digues différentes sous des conditions variées de chargement et d'infiltration. On fait ces analyses pour évaluer le hasard d'une rupture de cisaillement, où une partie d'une digue, ou d'une digue et sa fondation, se déplace sur une surface bien définie et qui se rapporte au reste de la masse. En plus des considérations d'une rupture de cisaillement, il faut évaluer aussi la possibilité des ruptures qui proviennent d'une déformation excessive ou de la liquéfaction.

Au présent, il y a plusieurs procédés disponibles pour faire les analyses de la stabilité. En général, les méthodes fondamentales comprennent ou l'arc du cercle ou la méthode du coin glissant, ou une combinaison des deux. Avec toutes ces méthodes, ou peut faire des computations en utilisant ou le principe de tension totale ou celui de tension efficace. Dans la plupart des cas, on fait les analyses en se rendant compte seulement d'une surface de rupture à deux dimensions.

Il faut évaluer les conditions variées de chargement ou des états d'infiltration, ou tous les deux, qui existent pendant la durée d'un barrage, à l'égard de leur effet sur la stabilité de la digue. On classifie d'ordinaire les conditions critiques, ou les stades, ainsi qu'il suit: (1) le stade de construction, (2) le stade d'infiltration continuelle ou de réservoir rempli, et (3) le stade d'affaissement, soit avec une mare pleine ou partielle. En plus de ceux-ci, il faut évaluer les effets des tremblements de terre sur le fonctionnement de la digue.

Au présent, on analyse la stabilité générale d'une digue soumise aux tremblements de terre de plusieurs façons différentes qu'on peut désigner ou pseudo-statique, en conformité avec Monsieur H. Seed, ou la voie de la réponse dynamique. Monsieur N. Newmark fut le premier à proposer les concepts fondamentaux de cette méthode, où on évalue les effets des tremblements de terre sur le fonctionnement de la digue par les déformations qu'ils produisent, et pas tellement par une réduction numérique d'un élément de sûreté. En général, la plupart des compagnies de dessin emploient au présent la méthode dite pseudo-statique comme la méthode fondamentale de l'analyse.

LE RÈGLEMENT DE L'INFILTRATION

Il est nécessaire non seulement que les digues et les fondations répondent aux critères pour la stabilité, mais qu'elles remplissent de certaines exigeances à l'égard du règlement de l'infiltration aussi. Dans tous les cas, quelques mesures de précaution contre l'infiltration sont nécessaires afin de protéger le barrage des effets peu désirables ou dangereux de l'infiltration, qui passe à travers le barrage lui-même, ou sous le barrage ou à travers les fondations ou les culées. A l'intérieur, les digues en terre ont des zones progressives, des tapis horizontaux de drainage, des drains en "cheminée" et d'autres drains au pied du barrage, pour gouverner l'infiltration dans la partie à l'aval du barrage. Plusieurs personnes trouvent qu'une combinaison de filtres inclinés et horizontaux est le meilleure solution et la plus positive pour le règlement de l'infiltration à l'intérieur (Fig. 14). On peut y trouver encore un avantage en ce qu'une telle configuration de drains permet l'inclusion dans la partie extérieure à l'aval du barrage d'une grande variété de matériaux qui, autrement, auraient été perdus (Fig. 16). De l'autre côté, si la coupe de la digue est surtout une coupe de remblai rocheux, composée d'un noyau impénétrable borné par un extérieur grossier, des transitions sont nécessaires pour diminuer l'érosion interne des matériaux du noyau, érosion qui pourrait arriver ou à travers les fissures qui peuvent développer à la longue, ou par la migration des particules fines sous les inclinaisons hydrauliques existantes. Les transitions doivent se composer de niveaux multiples (Fig. 17) ou de zones relativement larges et remplies de matériau soigneusement classé (et dont les grains sont de plusieurs dimensions) (Fig. 19).

A l'égard des fondations sous-jacentes et des culées, il y a une variété de méthodes pour gouverner l'infiltration. La question de quelles mesures sont préférables vis-à-vis un projet particulier dépend de plusieurs éléments, mais en général, il faut assurer la sûreté de la digue, et en outre, il faut justifier le genre du traitement par des considérations financières. En bien des cas, la considération de toutes les exigeances du projet mène à l'adoption de plusieurs genres, au lieu d'un seul, de mesures contre l'infiltration. Bien sûr, un mur parafouille positif, qui est faconné dans une excavation ouverte à une couche impénétrable, et qui est remblayé d'un matériau impénétrable compacté, est la forme la plus désirable. Pourtant, quand un tel mur n'est pas possible d'un point de vue pratique, il faut considérer d'autres mesures telles que celles qui suivent:
(1) voiles injectées (Fig. 21 et Table VI. La Figure 21 montre le procédé de la tube à manchettes.);
(2) murs parafouilles en béton (Fig. 23 et Table VII);
(3) tranchées talissantes de parafouille (remblayées de terre) (Fig. 27); (4) voile d'étanchéité à l'amont;
(5) pieux en tôle d'acier (Fig. 28); et (6) drains verticaux ou puits de décompression et galeries de drainage (Fig. 29). (La Figure 30 montre une coupe transversale du barrage du Haut Aswan avec trois galeries dans le

noyau à côté d'autres mesures contre l'infiltration, y compris une voile d'étanchéité à l'amont, une voile injectée et des puits de décharge à l'aval.)

En plus des études par les ingénieurs de la stabilité générale d'une digue et des mesures contre l'infiltration, il faut considérer les hasards du fendement dans le noyau et de l'érosion interne. Il est nécessaire en outre de considérer comment le déplacement de la charge altère le fonctionnement de la digue. Des études récentes indiquent que le déplacement de la charge peut influer d'une manière significative sur le fonctionnement de la digue, occasionnant des déformations, des déplacements, des contraintes, de la pression de l'eau interstitielle, et ainsi de suite.

LA CONSTRUCTION

La construction des barrages à digues entraîne d'ordinaire la dérivation de la rivière par des tunnels ou des canaux temporaires par le moyen de batardeaux de remblai rocheux. Pourtant, on juge parfois que les méthodes de convention de construire un barrage au sec ne sont ni pratiques ni économes à cause des exigeances et des incertitudes qui se rapportent à la construction des batardeaux et à l'assèchement. Des gisements étendus d'alluvion perméable qui descendent parfois à des profondeurs dont on n'a pas la mesure, tous couverts par des eaux profondes et rapides, font les conditions les plus importantes qui ont mené au développement d'un dessin qui utilise des méthodes de construction sous l'eau. Les exemples des barrages récemment bâtis avec cette Méthode comprennent le barrage du Haut Aswan en Égypte, le barrage Flèche (Arrow) au Canada et le barrage Jarkvissle en Suède.

La méthode conventionnelle de construction épuise la région entre les batardeaux; et l'excavation de la fondation, la préparation et le nettoyage convenables suivent l'épuisement. L'étendu des excavations entre les batardeaux dépend de plusieurs éléments, mais essentiellement, c'est le minimum nécessaire pour assurer un soutien suffisant pour la digue. D'ordinaire, les excavations enlèvent tout surcharge et toutes roches altérées jusqu'au niveau où les propriétés physicaux des matériaux sont égales ou meilleures que celles des remblais rocheux ou terreux qu'on va y mettre. Le traitement de la surface du fond rocheux, la région du noyau exceptée, entraîne le nettoyage méchanique de la surface nue du fond rocheux. Dans la région de contact du noyau, cependant, il est essentiel qu'on fasse plus que l'attention ordinaire à la préparation des fondations, afin de réduire tous les accidents de terrain qui augmentent la possibilité des tassements différentiels et du fendement, ou qui empêchent qu'on fasse le compactage nécessaire; il faut, d'ailleurs, remplir toutes les fissures qui pourraient occasionner l'érosion progressive du noyau. Bien souvent, le matériau du noyau qui a des propriétés spécifiques est placé et compacté à la main contre les culées préparées ou les fondations ou tous les deux. Au barrage de Terzaghi au Canada, à cause des tassements sérieux qu'on attendait dans la digue, et à cause d'autres considérations, on compacta une couche d'argile humide d'une largeur d'un pied contre les culées avec des compacteurs opérés à la main. D'une façon un peu semblable, on plaça et compacta à la main du matériau du noyau spécialement préparé sur les fondations et les culées du barrage de Miboro en Japon et du barrage de Mont-Cénis en France. Quelquefois, le noyau et les transitions s'évasent aux culées afin d'augmenter la longueur d'infiltration et de protéger davantage contre l'érosion. On mit ce principe en action aux barrages de Round Butte aux États-Unis, d'El Infiernillo au Mexique, et de Furnas dans l'Amerique du Sud, parmi bien des autres (Fig. 13, Diagramme A).

Il y a une grande diversification dans les procédés de placement et de compactage des matériaux du noyau, dû en grande partie à la grande gamme de matériaux qu'on emploie, mais dû aussi aux conditions de climat et à d'autres éléments. Les épaisseurs des niveaux pour les matériaux du noyau en plusieurs des barrages les plus importants varient entre 15 et 50 centimètres environ, bien qu'on emploie parfois des niveaux plus épais, par exemple, les niveaux de 91 centimètres qu'on choisit après les essais sur le terrain au barrage de Slottmoberget en Norvège. Effectivement, on place et compacte les noyaux de la plupart des massifs à zones en terre et en remblai rocheux à un teneur en eau optimum. En Suède, où la saison de construction est courte et pluvieuse, on place et compacte des moraines envasées à un teneur en eau d'environ 4 pour cent de l'optimum. Des exceptions distinctes à la discussion ci-dessus sur les teneurs en eau sont les barrages construits aux États-Unis par le Bureau de Réclamation. La gamme des teneurs en eau dans ces barrages-là varie entre 0.7 pour cent au-dessus de l'optimum et 2.5 pour cent audessous de l'optimum.

Comme les genres des matériaux, le matériel qu'on emploie pour compacter les zones des noyaux de barrages est variable, bien qu'on emploie le plus souvent les rouleaux statiques à pieds de mouton et les rouleaux compacteurs à pneus. Aux barrages d'Oroville et de Portage Mountain (W.A.C. Bennett), on utilisa de lourds rouleaux compacteurs à pneus, qui pesaient de 90 à 100 tonnes, pour compacter les matériaux du noyau: sable et gravier envasés et sable envasé. Pour compacter le Noyau d'argile à Mangla, on employa un rouleau compacteur à pneus, dont chaque pneu était suspendu indépendament des autres.

Quant aux transitions, on emploie d'ordinaire les rouleaux vibrants pour le compactage par un certain nombre de passages d'un tracteur rampeur, ou tout simplement en employant le matériel de portage et de deploiement. En bien des cas, on essaie d'atteindre une densité relative de 70 pour cent du moins.

Autrefois, les zones de remblai rocheux se composaient de remblai basculé bien lavé au sluice. Peu après 1950, pourtant, il devint évident qu'il serait pratique de placer et compacter le remblai rocheux en des niveaux relativement peu épais, en utilisant des rouleaux vibrants. On trouva qu'il était possible de réaliser, par un tel proces, un remblai rocheux à propriétés de fonctionnement supérieures à tout ce qu'on avait eu auparavant. En outre, il était possible avec ce procédé d'utiliser de la roche qui n'avait pas été convenable aux remblais rocheux; donc, on profitait d'un point de vue économique de l'emploi du remblai rocheux compacté. Il semble raisonnable de dire que, dans les barrages les plus récemment construits, la plupart du remblai rocheux est compacté.

En général, on met le remblai rocheux compacté, dans la plupart des barrages, en des niveaux de moins de deux mètres, mais on utilise parfois des niveaux plus épais. Du remblai rocheux placé d'une épaisseur de plus de 3 ou 4 mètres environ appartiendrait à la catégorie du remblai rocheux "basculé". Comme dans le compactage des zones du noyau et de la transition, on emploie une grande variété de matériel de compactage pour compacter le remblai rocheux: les tracteurs rampeurs, les rouleaux compacteurs à pneus, et les rouleaux vibrants en sont des exemples. Au présent, le poids statique des rouleax vibrants varie entre 3.5 et 15 tonnes environ mais il est possible qu'on voie bientôt des rouleaux plus lourds. L'usage de laver les remblais rocheux au sluice est très variable aussi, d'une agence à l'autre. Bien des choses dépendent des propriétés physiques de la roche qu'on y met. Il arrive parfois que la pollution ou la contamination de la rivière à l'aval du barrage par l'écoulement du sluice est un problème très important à considérer. Les zones de remblai rocheux compacté dans les barrages de Mont-Cenis, Trangslet, Messaure et Akosombo furent lavées au sluice, le dernier avec le rapport entre l'eau et la roche à 1/2: 1. De l'autre côté, le remblai rocheux aux barrages plus récents, Round Butte, El Infiernillo, Gepatsch, Carters, Summersville et Mangla, ne fut pas lavé au sluice. Dans plusieurs pays, y compris des pays scandinaves, l'on place le remblai rocheux dans des sections déterminées du barrage, sous des conditions hivernales. Dans ces sections, on bascule le remblai rocheux et le compacte en niveaux avec des rouleaux vibrants. Afin d'éviter le risque de la congélation, on limite la hauteur du remblai hivernal à 10 mètres chaque année. Des études continuées des tassements et de la compactabilité du remblai placé en été et celui qu'on place en hiver démontrent qu'il y a très peu de différence de compactabilité entre les deux.

Le contrôle du placement des matériaux de la digue est nécessaire pour assurer que les propriétés du sol ou du remblai rocheux soient compatibles avec les hypothèses du dessin. Depuis peu d'années, l'augmentation des taux du placement des matériaux exigent une revue critique des techniques de contrôle et d'analyse des renseignements accueillis par les essais sur le terrain. Pour le contrôle de la densité des sols à grains menus, il paraît qu'on emploie le plus souvent l'essai du cône de sable. De l'autre côté, pour le contrôle des matériaux de digue à grains grossiers, il y a une tendance de faire les essais de densité en place aux grands diamètres (2-1/2 à 6 pieds), les mesures de volume déterminées d'ordinaire par une méthode d'échange d'eau. En outre, les hauts taux de placement nécessitent une détermination rapide du classement du matériau. Par exemple, aux barrages d'Oroville et Portage Mountain, on développa une méthode rapide de contrôle de gradation, qui entraînait le lavage au crible et le pesage par la voie humide des matériaux pour constater le pourcentage des grains qui dépassaient une ou deux dimensions critiques au crible. En général, l'analyse des renseignements du contrôle de construction, et une évaluation de l'acceptabilité du terrassement emploient des méthodes statistiques qui se rendent compte des variations au hasard dans les pratiques à l'égard des terrassements.

L'INSTRUMENTATION

Depuis 1960 environ, l'instrumentation se développe et s'installe dans un grand nombre de barrages importants pour mesurer les déplacements horizontaux, les déformations, les tassements et les contraintes. Ces instruments variés et d'une grande importance fournissent, parmi d'autres choses, d'importants renseignements quantitatifs à l'égard du rapport entre les propriétés des matériaux de digue et les déplacements, déformations et contraintes qui arrivent dans les digues pendant diverses périodes critiques, par exemple, pendant les tremblements de terre. Parmi les genres principaux d'instrumentation qui sont actuellement disponibles pour installer dans les digues et les fondations des barrages à digues, il y a: (1) le piézomètre (Fig. 39); (2) le dispositif du déplacement interne (Figs 40 à 50); (3) les mesures de la surface (Fig. 53); (4) les mesures de contrainte (Fig. 52); et (5) les enregistreurs des tremblements de terre (Fig. 53). Il y a une concordance générale de ce que l'instrumentation qui donne des renseignements dignes de foi pour assez longtemps est préférable aux systèmes compliqués qui ne durent pas longtemps. On voit par l'expérience que l'installation de l'instrumentation ne gêne pas les activités de l'entrepreneur, et, si on la manie d'une façon convenable, on peut réduire les dégâts au minimum. Si on considère les avantages énormes d'un' système bien réfléchi et bien choisi d'instrumentation, les frais, qui varient en général entre 0.1 pour cent et 1.5 pour cent du prix de la digue toute seule (la variation dépend des problèmes du barrage particulier, sont petits.

DÉFORMATIONS DANS LES BARRAGES EN TERRE ET EN REMBLAI ROCHEUX

On instrumente un grand nombre de barrages au présent pour mesurer les déformations internes qui paraissent pendant la construction. Ces déplacements sont souvent plusieurs fois plus grands que ceux qui arrivent après qu'on a fini la construction. Les figures 55 à 65 présentent des renseignements typiques sur les barrages de Mammoth Pool (Fig. 55 et 56), El Infiernillo (Figs. 57, a 61), Netzahual-coyotl (Fig. 62), Muddy Run Embankment (Figs. 63 et 64) et La Villita (Fig. 65).

Plusieurs trouvailles qui résultent de ces mesures suivent:

(1) Il n'y a aucun glissement au point de contact entre ou le noyau ou l'extérieur et la culée.

(2) Les accidents de terrain ont peu d'effet sur les déformations internes, mais ils occasionent des zones localisées de grand effort de cisaillement.

(3) Il y a un rapport entre le tassement vertical et la déformation horizontale, ce qui éclaircit un peu la cause du fendement de la crête.

(4) Il est possible que la compactabilité du remblais rocheux de bonne qualité soit égale à celle d'un noyau en argile compactée.

Dans le dessin pour l'instrumentation des nouveaux barrages, on met plus d'importance maintenant sur l'instrumentation pour mesurer les déformations longitudinales et parallèles à l'axe dans les 15 mètres supérieurs de la digue, déformations adjacentes à la partie supérieure de chaque culée. Lorsque les chambres de pressions sont plus dignes de foi qu'elles ne l'étaient auparavant, on installe plus de ces dispositifs.

PROBLÈMES DU DESSIN ET DE LA CONSTRUCTION

La Comité ASCE sur les Barrages en Terre et en Remblai Rocheux soumit un rapport en 1967 qui cataloguait les dix problèmes suivants, sur lesquels il y a un besoin urgent de nouveaux renseignements. Les détails de la liste suivent, selon l'ordre de priorité. La liste ne contient que les problèmes propres aux digues, sans embrasser ceux des culées ou des fondations.

Problèmes d'Importance Majeure

1. Les propriétés de résistance et de changement de volume du gravier et des matériaux de remblais rocheux sous de grandes pressions resserrantes
2. Les fendements dans les digues
3. Les méthodes de compaction du remblai rocheux et des graviers grossiers

Problèmes d'Importance Moyenne

4. Les éléments de glissement du remblai rocheux sur les surfaces de contact de la fondation
5. La prédiction des pressions interstitielles dans le sol cohésif compacté
6. Le fonctionnement dynamique des digues dans les régions des tremblements de terre (y compris les exigeances à l'égard de la partie supérieure du barrage, qui surmonte l'eau)
7. Les mesures de la contrainte et la déformité dans les digues

Problèmes Relativement Peu Importants

8. Le contrôle du compactage des remblais rocheux et graviers
9. La protection des pentes dans les barrages en terre
10. Des mélanges à bon marché pour l'amélioration des propriétés du sol.

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