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UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

MEMORANDUM TO CHIEF DESIGNING ENGINEER

Translation from German

SUBJECT: "METHODS OF TESTING CONSTRUCTION MATERIALS TO DETERMINE THEIR SUITABILITY FOR USE IN AN EARTH DAM"

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TRANSLATOR'S NOTE

The original of this article, "Untersuchunge methoden um fertzustellen ob sich ein gegebenes Baumaterial für den Bau eines Erddammes eignet," appeared in volume III of "1er Congres des Grands Barrages," Stockholm, 1933. No great effort has been expended to make the translation perfect beyond the demands of technical accuracy because the time required for such a rhetorical revision is not felt to be justified in view of the great amount of additional material whose translation is continually in demand.

D.P.B.

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METHODS OF TESTING CONSTRUCTION MATERIALS TO DETERMINE THEIR SUITABILITY FOR USE IN AN EARTH DAM

INTRODUCTION

The response to this question, propounded by the International Commission on High Dams of the World Power Conference, and which is to be discussed at the meeting of the conference in Stockholm during the summer of 1933, must be restricted to the narrowest limits in accordance with the instructions and policies laid down by the Standing Committee of the Commission, because otherwise the temptation to bring into the report elements from a large field, such as geology, soil mechanics or foundations either from consideration of their close relation to the problem or for the sake of completeness, would be very great. For these reasons any discussion of earth as foundation has been avoided and only earth as construction material for dams has been treated. However, even though the questions put by the High Dams Commission relate only to the earth dams which impound reservoirs, all of the methods used in connection with embankments and transportation canals, power canals, and the settling basins of ore dressing plants, or with levees and dikes, are pertinent because of the similarity between the construction materials, their method of introduction into the structure, and the physical conditions during and after construction.

In Part I the following question will be taken up: What properties make a particular soil type suitable for use as a construction material in dams and which properties may prohibit its use, either entirely or in certain parts of the dam? In this connection the report must go into the design of the cross-section, as well as the type of construction which is planned, whether dry dumping, scraping, hydraulic fill, rolled fill, or tamping, etc.; for these features are intimately related to the applicability of the soil. Also the principal dimensions of the dam, particularly the height and with it the water pressure, must be considered.

Primarily only recent dam and embankment construction in Germany will be brought into the discussion.

Part II takes up the physical condition of the water in the soil.

In Part III are treated the experimental methods by means of which the important properties of the soil are disclosed and tested, both at the borrow pit and during construction, as well as in the finished structure.

PART I

THE MECHANICAL FUNCTIONS OF THE VARIOUS PARTS OF AN EMBANKMENT OR DAM

The soil to be employed in an earth dam must fulfill various functions and must therefore possess a number of different properties. As a rule either all of these properties are not to be found together in a single soil type or at least not in their full degree of perfection, or, if the soil is especially suitable, it will either not be found in sufficient quantities for a large dam or will not be adaptable to excavation by large scale methods. It has therefore become customary to assign to the various parts of the dam particular functions and to distinguish in the more recent and carefully constructed high dams:

1. An impervious part, whose function is to prevent or to reduce the penetration of the impounded water;

2. A supporting part, whose function is to withstand the vertical and horizontal forces (weight and water pressure) and to transmit them into the foundation;

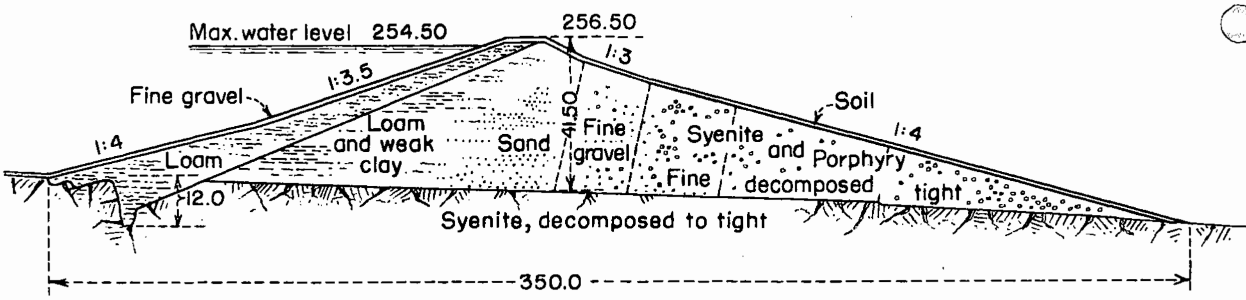
3. A protecting part, whose function is to resist the attack of flow, waves, ice, and frost on the upstream side, and of rain and wind on the downstream side. A discussion of the destructive effects of burrowing animals and the appropriate corrective measures does not belong here. In a dam so constructed, this affords the possibility of utilizing soils of very different types, each in its appropriate place.

In many cases the impervious and supporting parts of the dam consist of a single homogeneous soil which performs both functions; particularly in older dams of low height, or with flood control reservoirs seldom filled and quickly emptied. The practice of the Prussian Ministry of Agriculture in 1907, with regard to dams, recognized only "dam-earths"; the report of the Commission on Dams, of the German Association for Water Economy and Water Power for 1930, concerning the "Principles for the Design, Construction and Use of Dams" on the contrary, recognizes in addition to homogeneous dams, the dam with functional divisions.

The impervious and the protecting parts are also sometimes combined in the form of a concrete apron or slope paving.

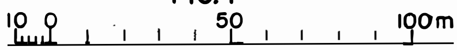
(a) The Impervious Body

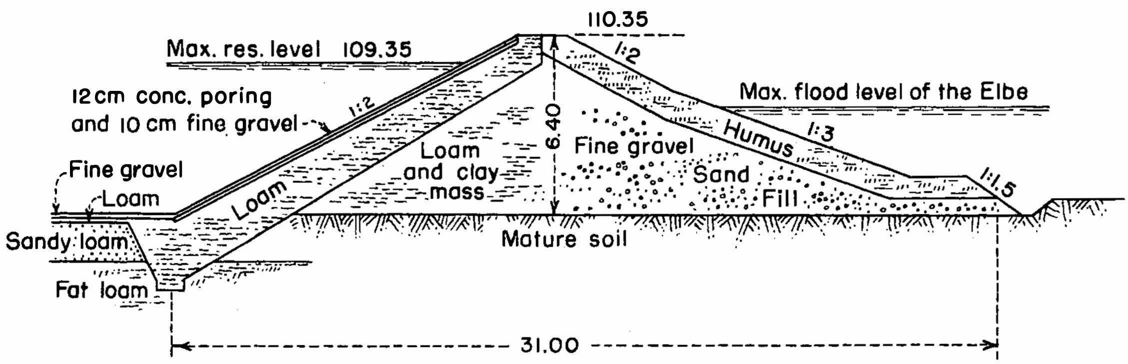
The water seal of a dam is of decisive importance. This is effected either by means of a tight soil, by means of concrete (also with masonry or cement mortar) or by means of steel sheet piling. With either the concrete or the sheet piling an additional seal of cohesive soil is generally understood.



UPPER PUMP STORAGE RESERVOIR AT NIEDERWARTHA (DRESDEN) 1927-1929

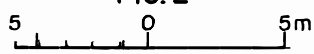
FIG. 1

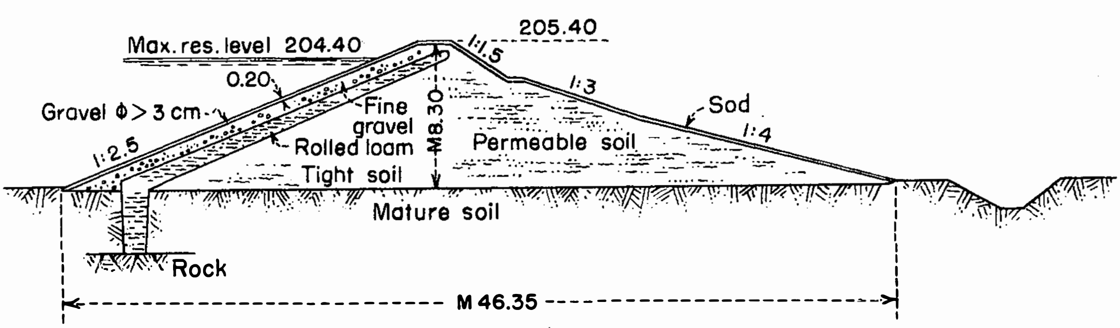




LOWER PUMP STORAGE RESERVOIR AT NIEDERWARTHA (DRESDEN) 1927-1929

FIG. 2





EQUALIZING RESERVOIR OF THE EDER DAM NEAR AFFOLDERN 1928

FIG. 3

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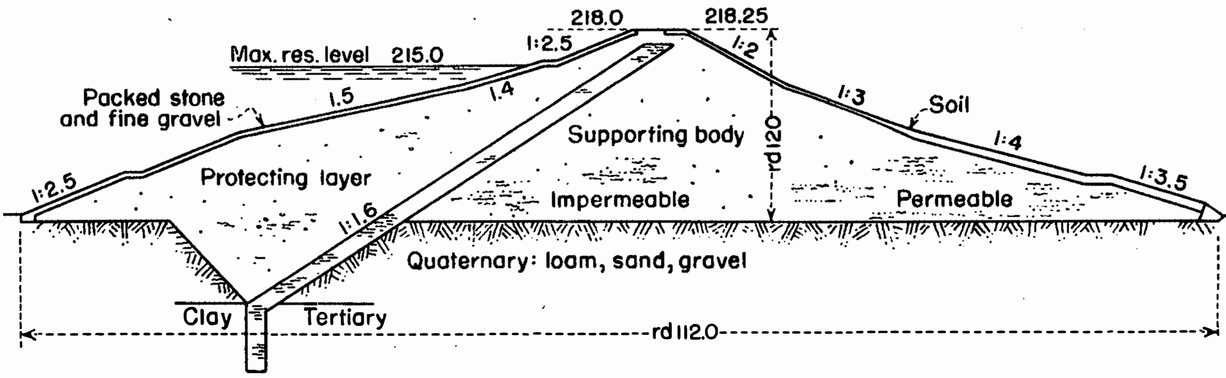
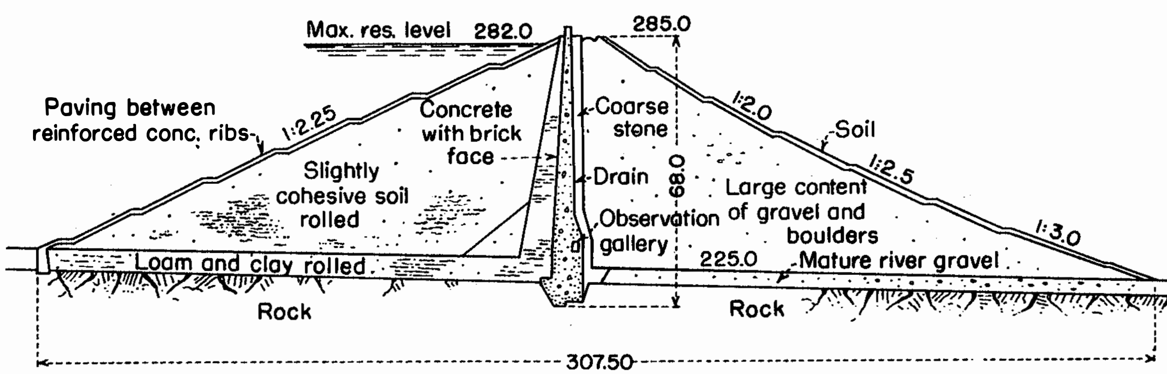


RESERVOIR OF THE GLATZER NEISSE AT OTTMACHAU 1928-1932

FIG. 4

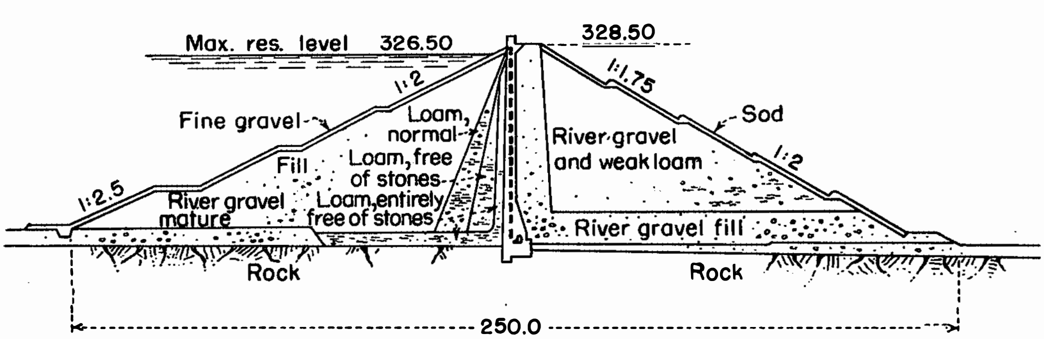
10 0 30m



DAM ON THE SORPE (RUHR) NEAR ARNSBERG 1928 (UNDER CONSTUCTION)

FIG. 5

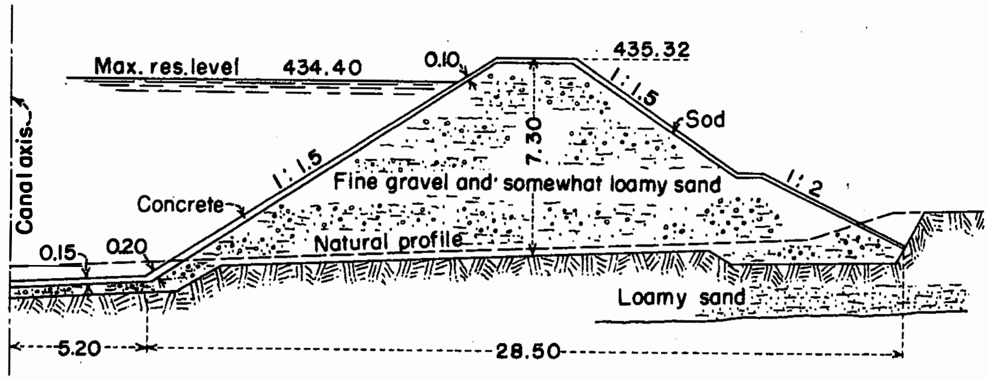
10 0 50 100m



RESERVOIR ON THE SOSE NEAR OSTERODE (HARZ) 1929-1931

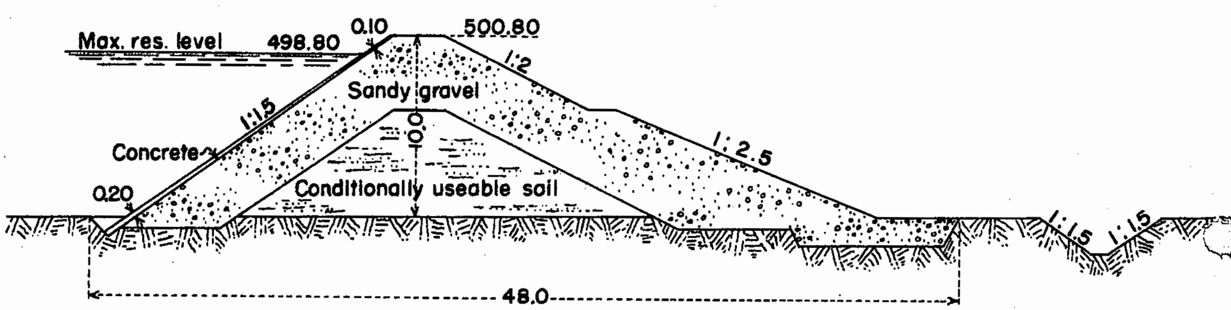
FIG. 6

10 0 50 90m



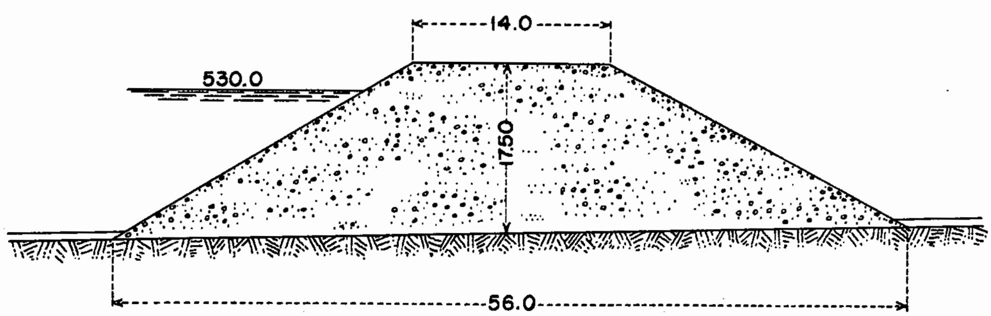
EMBANKMENT OF THE POWER CANAL ON THE MIDDLE ISAR 1926–1929

FIG. 7



STORAGE BASIN ON THE MIDDLE ISAR 1926–1929

FIG. 8



UPPER GROSSHARTMANNSDORF BASIN NEAR FREIBERG (SAXONY) 1593

FIG. 9

Where there is to be a single seal (figure 13), or where, as already mentioned, there is to be an auxiliary seal with concrete, or sheet piling is to be used, the impervious body lies on the upstream side at a flat slope under a protecting layer of sand and stone or concrete (figures 1 to 3); or where there is danger of sloughing of the impervious soil it may lie farther in the interior of the body of the dam at a steeper slope under a stronger protecting layer which serves as ballast and as resistance to sliding (figure 4); or it may be arranged in the interior at the middle of the dam as a core.

The problem of sealing a dam on the upstream side with a concrete or masonry apron without any impervious layer of loam or similar soil is really foreign to the question in hand; it will be treated briefly later (figures 7 and 8). It represents an exceptional practice in reservoir embankments.

The significant properties of a soil suitable for producing a water seal are:

The greatest possible impermeability.

Sufficient plasticity to form a coherent sealing layer.

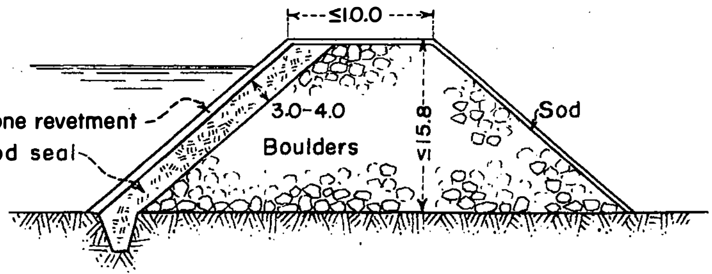
The greatest possible cohesion and internal friction to reduce the danger of sloughing.

A low water absorption capacity and low swelling so that the soil neither dissolves, flows away, or sloughs.

Low shrinkage upon drying so that no fissures form, especially during construction, before the protective layers have been applied. Also because of the possibility of settlement of the dam, the compressibility of the material to be considered, and, primarily in conjunction with thin concrete aprons, the resistance to frost is of importance.

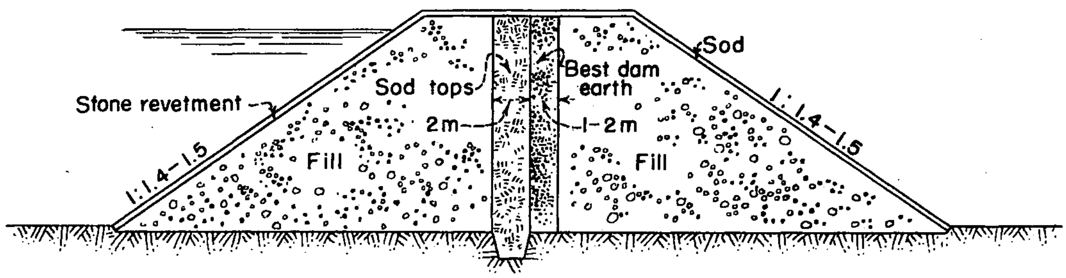
At present the so-called cohesive soils, clay, red clay, loam, boulder clay, boulder marl and marl are used exclusively for impervious seals.

The old dams of the Oberharze Mine, built in the sixteenth and eighteenth centuries (probably also those of the Saxon miners in the Erz Mountains (figure 9) utilized a special type of seal made of sod or turf blocks which proved quite satisfactory but which have not been used again in recent times.



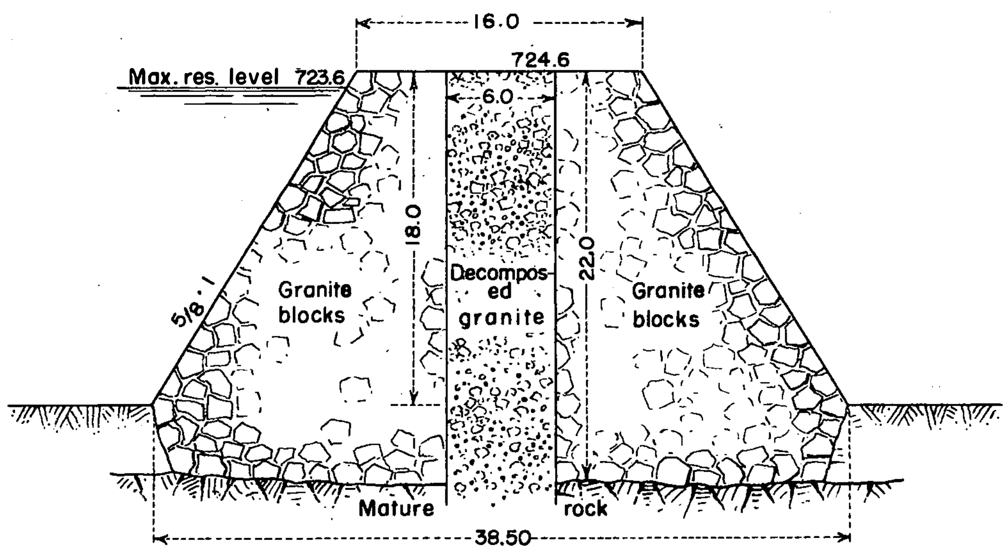
HARZ BASIN DAM, EARLY TYPE

FIG. 10



HARZ BASIN DAM, RECENT TYPE

FIG. 11

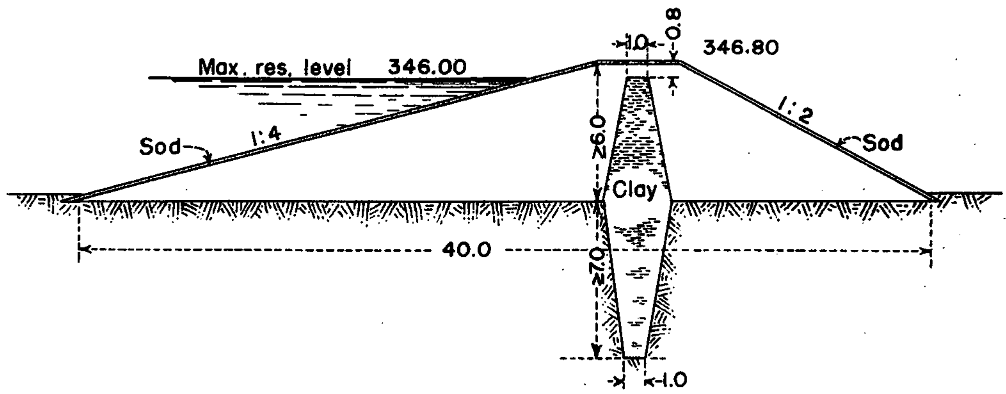


ODER BASIN DAM IN THE HARZ, 1714, 1721 (RAISED 1765)

FIG. 12

20 0 40m

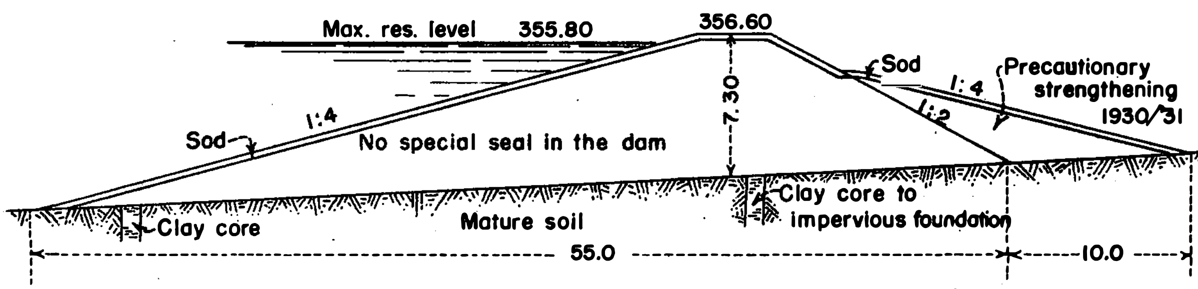
X-D-1470



FLOOD RESERVOIR ON THE HEIDE (BOBER) NEAR HERISCHDORF (LOWER SILESIA) 1904–1907

FIG. 13

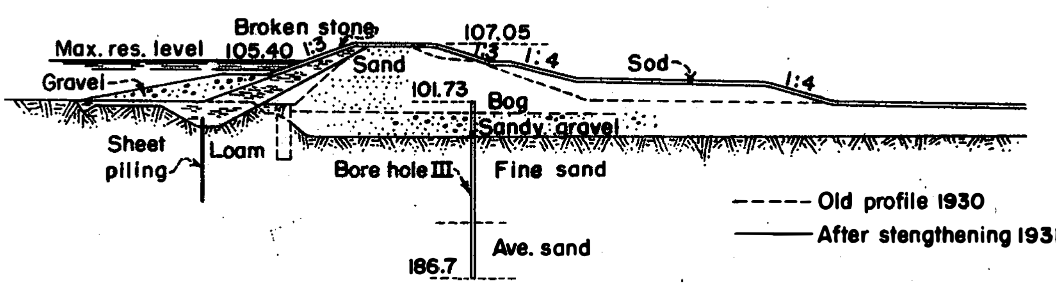
5 0 15m



FLOOD RESERVOIR ON THE LANGWASSER NEAR FRIEDEBERG QUEIS (LOWER SILESIA) 1908–1910

FIG. 14

5 0 20m

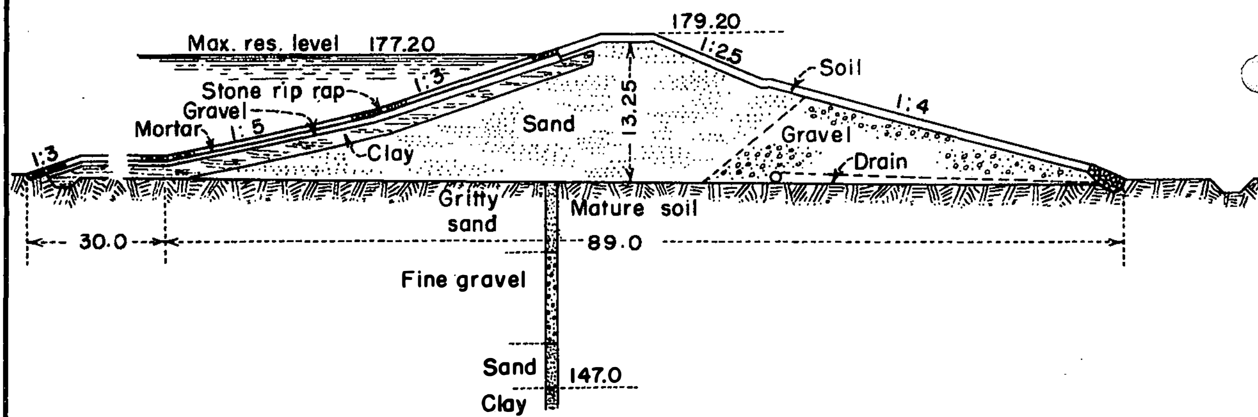


RESERVOIR ON THE KÜDDOW NEAR FLEDERBORN (BOUNDARY BETWEEN POLAND AND WEST PRUSSIA) 1931

FIG. 15

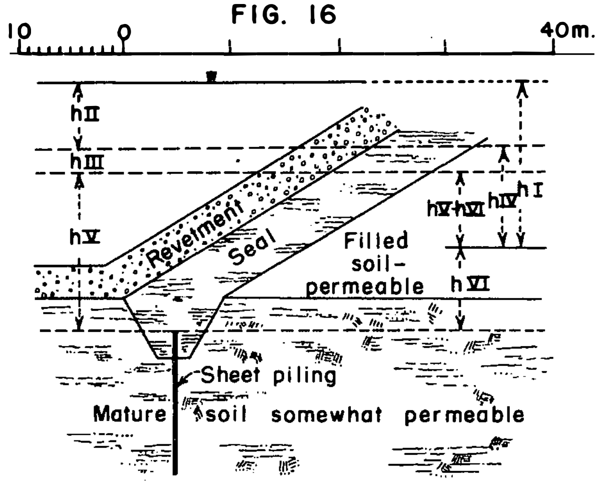
10 0 30m

X-D-1471



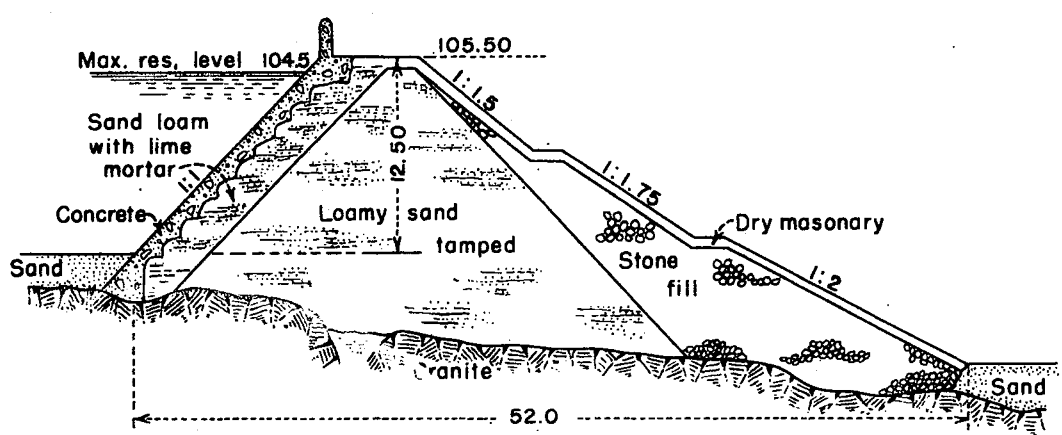
RESERVOIR ON THE MALARPANA NEAR TURAWA (UPPER SILESIA.) DESIGN 1932

FIG. 16



hI Pressure drop in the seal.
hII Pressure drop in the mature soil on the upstream side up to lower edge of cut-off.
hIII Pressure drop in the joints of the sheet piling
hIV Pressure drop in the downstream side from the lower edge of the cut-off to the tailwater.
hV Excess pressure on the top of the cut-off, upstream.
hVI Excess pressure on the top of the cut-off, downstream.
hV-hVI Resultant pressure at the top of cut-off.

FIG. 17



SCHIESSROTHRIED DAM (VOGESSEN) 1888

FIG. 18

5 0 20m

[In the older dams the heavy sod layer was placed on the upstream side (figure 10); in the later dams it was placed in the interior in a trench which reached into the impermeable foundation (figure 11). For the rest the dams consisted of rubble and talus. A 17-meter high dam in the Oder basin constructed between 1714 and 1721 (figure 12), exhibits the only departure from this practice by utilizing downstream from the sod core an additional fixed core of decomposed granite, a product of kaolinizing decomposition, which exhibits certain cohesive properties; the remainder of the dam is of dry masonry with moss packing in the joints. The tight root mass of the sod which held the earth together effected the seal and thus protected the dam from piping (9)].

The upper end of the impervious layer must be carried high enough above the highest reservoir level - including allowance for flood conditions, piling up from the effects of wind, and wave action - so that any considerable quantities of water may be prevented from climbing over the impervious layer through capillary action in the protecting apron; this capillary rise can be definitely broken by means of a layer of coarse-grained soil.

Wherever possible the impervious layer should be connected at the bottom and the sides with either an impervious or a very slightly pervious foundation or abutment in order that water will not seep under or around the dam; in addition to the loss of water from the reservoir any considerable movement of water through the adjacent foundation or abutment can lead to the destruction of the dam through rupture or sloughing of the downstream face. The connection to the impervious layer of the foundation or the abutment where the depth of permeable overburden is not great, should be made by means of a trench filled carefully under water with the impermeable soil. (compare figures 1 to 4, 13 and 14). Where the depth of the pervious overburden is greater or where large quantities of water would penetrate into the trench, steel sheet piling is indicated (figure 15), provided the presence of large boulders would not prevent their tight junction with the impervious foundation. In this case an impervious blanket or apron is implied. If the junction with an impervious foundation is not obtainable, then the path of percolation must be extended so that the flow gradient and with it the quantity of flow is reduced. This can be accomplished either by driving sheet piling or a clay wall deep into the foundation, or by placing an impervious apron of impervious soil on a long slope on the upstream side (figure 16). Yielding soils (bog, for example) must be removed from under the impervious apron and from under the dam itself in order that settling cracks which would open short channels with a severe gradient under the dam shall not occur as the result of the earth or water loading. The choice as to whether sheet piling or an apron is to be used is also a question of cost, but where ample impervious material is at hand the apron will in general be cheaper. Investigations as to the impermeability and the formation of the foundation and observations of the gradient of the natural ground waters at the site of the dam will disclose whether or not an apron can be made to suffice in consideration of the increased excess water pressure of the reservoir.

If any weaknesses are exhibited either in the slope or at the foot of the slope on the upstream side, repairs can be readily effected when the reservoir is empty. Much more difficult would be any repair operations if the impervious clay layer were combined with sheet piling or a trench in the middle of the dam. In massive dams with cores, which will invariably be founded on rock, the inspection galleries always present will affort opportunity for the plugging of leaks after construction, with no great difficulty whether in masonry or in a fractured rock foundation. With sheet piling, care must be taken that no vertical channels along the lock joints which would conduct water from a point of high pressure to one of lower pressure be left open; for example, from the lower end of the piling to the head remaining in the impervious layer (figure 17).

Injuries to the impervious layer can take place in various ways:

1. With the sloped impervious layer, the protecting apron and the impervious layer may slough away together as a result of the leaching and dissolution of the upper surface.

2. As a result of the penetration of water into the impervious body its shearing resistance may be so reduced that a sliding surface is formed in the impervious material itself, particularly if there are sudden changes in the reservoir level. Tamping and roller laminations sometimes define the surfaces of such slides.

3. Through transposition of the soil. Each impervious layer must act as a filter, for a completely impervious blanket can scarcely be expected in artificially placed soils; also the tightest soils are also the soils with the smallest angles of friction and are therefore the most susceptible to sloughing. (Where it can be avoided they are therefore never used.) The water which is forced into the soil on the upstream side under pressure must debouch on the downstream side with its pressure completely relieved and with so small a velocity that it will be impossible for it to carry the finest particles farther into the neighboring and supporting mass. The voids of this supporting layer must not be so large that portions of the impervious layer may be pressed into the foundation or into the mass, for such transposition of material would produce cavities.

On the contrary, a favorable condition from the point of view of the permeability and therefore from the point of view also of stability, may result if because of either transposition of soil particles or the chemical separation of dissolved materials, or because of the deflocculation of irreversible colloids (hydrate of iron oxide, silicic acid, or humus compound) a spontaneous seal on the upstream side is produced.

Careful moistening of loamy and sandy soils with lime mortar and tamping in very thin layers will produce a particularly strong and a dense body for the dam (figure 18).

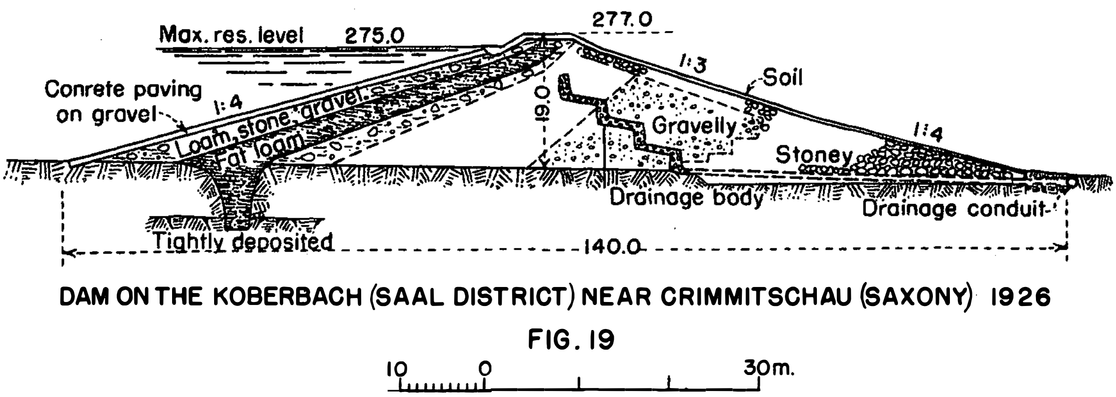
The thickness of the impervious layer will depend upon its permeability; furthermore, this must be increased toward the bottom in proportion with the water pressure. The impervious seal placed as a core will require the smallest quantity of material since this type is most protected and will be further consolidated by the settlement of the dam. Nevertheless half of the body of the dam will be saturated. For this reason the path of the water penetrating the core seal and the foundation will be considerably shorter for the same breadth of base of the dam than if the impervious blanket were placed upon the upstream face.

In dams of great height the soil in the deepest portions is under greater pressure, for the most part under much greater pressure than in the borrow pits. The water which is naturally present in the soil as well as that which either intentionally (as in hydraulic fills) or unintentionally (rain) is added to the material during construction, insofar as it is excessive, must be given time and opportunity to drain off; otherwise the over-saturated soil will act upon its environment - for example, core walls, pipe galleries - as a viscous fluid weak in friction or shear and will become susceptible to sloughing or piping (compare below).

The speed with which the construction of a dam progresses thus has a considerable bearing upon its safety.

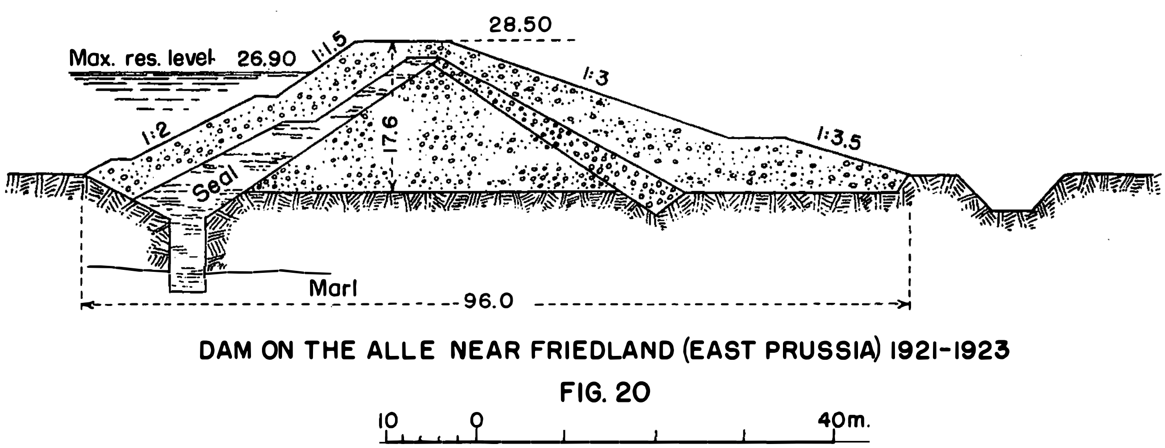
(b) The Supporting Body

Whereas the impervious body of the dam must permit the penetration of as little water as possible, the supporting body on the contrary must permit such seepage water as passes the impervious layer as well as any rain water which has penetrated the dam, to fall rapidly towards the bottom, and must conduct it underground away from the site of the dam. (A dam constructed throughout of uniform "impermeable" soil will be considered later.) Then the major portion of the supporting body will remain dry and capable of resisting horizontal forces. Sufficient shearing strength and permeability is therefore an important requirement of soils for supporting bodies. Their compressibility must be small; therefore they must be compacted as densely as possible during construction in order that subsequent settlement - particularly



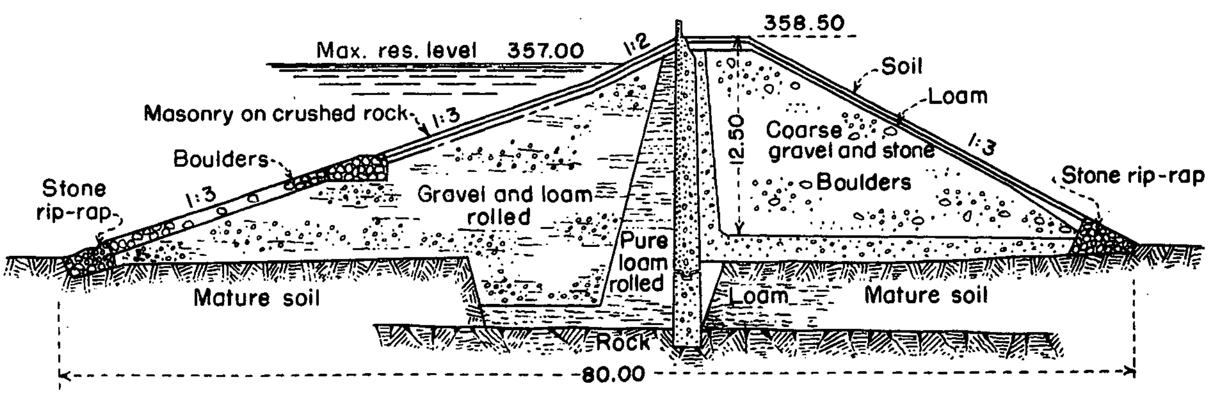
DAM ON THE KOBERBACH (SAAL DISTRICT) NEAR CRIMMITSCHAU (SAXONY) 1926

FIG. 19



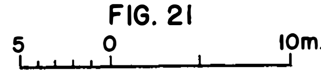
DAM ON THE ALLE NEAR FRIEDLAND (EAST PRUSSIA) 1921-1923

FIG. 20



EQUALIZING RESERVOIR FOR THE SAAL DAM NEAR BURGHAMMER 1931

FIG. 21



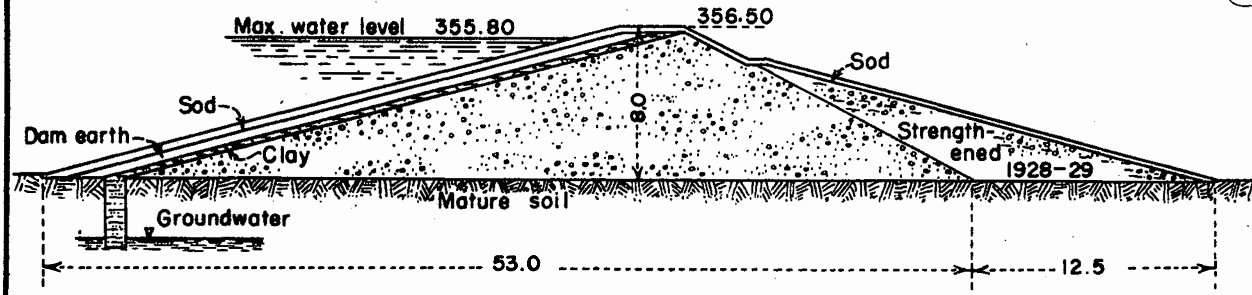
unequal settlement - which might endanger the various structures in the dam (weirs, outlets, core walls, sheet piling) may be avoided. The elastic condition of cohesive soils is therefore very disturbing since it may vitiate the initial artificial compaction in part. Gravelly and sandy soils retain their compaction. A high density increases the resistance to horizontal forces; the particularly light organic soils (peat mold, marshland, and such) may be eliminated for this reason. The differences in density of the useful mineral soils are fairly small; they depend primarily upon the density of the deposition and upon the water content; the specific gravity of most of the soil-forming minerals is almost the same (about 2.65).

If sufficient soil of the necessary permeability is not available for the entire supporting body, then at least the downstream side and the deeper parts should be made of this material, while the upstream part and the upper part may be constructed of the less permeable soil. "Percolation layers" in the interior of an otherwise only slightly permeable supporting body have been used only occasionally in Germany (figures 19 and 20). On the other hand foundation drains have often been built either of coarse stone filler (in trenches) or of drain pipe with filter type back fill (compare figures 16 and 20) or by excavating the tight loam layer over a permeable gravel layer under the downstream toe of the dam (compare figure 6) when conditions such as are frequently found in river valleys exist. Foundation drains can not, however, be permitted to wash out any of the soil without danger.

In dams with concrete cores any water that may penetrate the core is collected on the downstream side and carried harmlessly downward through dry masonry (compare figure 5). This stone back fill, however, must not be of such a nature as to yield under the influence of horizontal forces, for the concrete core which is not designed for bending stresses would then carry the complete load. Dr. Ing. Collorio, director of the Harz Water Company, proposes a double wall of steel sheet piling with a gravel filling to conduct the water downward.

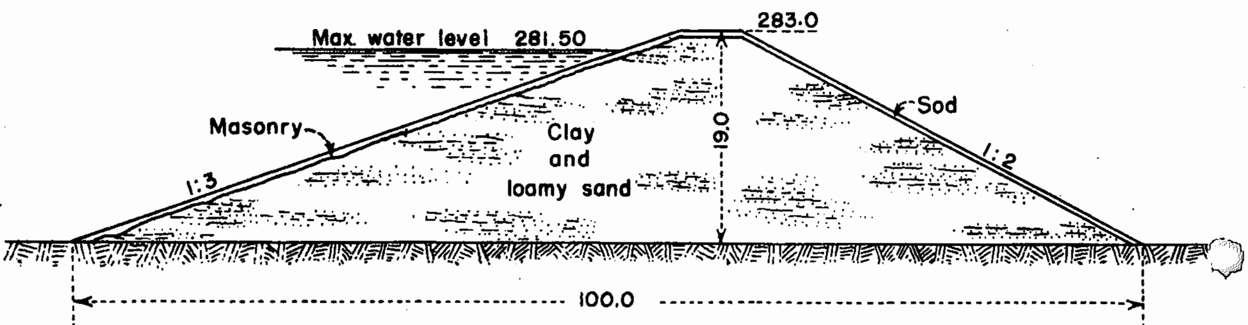
(c) The Protecting Layer or Revetment

On the downstream side sod in its native soil is sufficient to conduct rain water to the toe of the dam without the formation of injurious fissures and gullies. High embankments are made with berms on which the water is collected in small ditches and led away. Sprinkling water should be permitted to penetrate the soil only so far as may be necessary for the growth of the sod. For this reason the upper surface should be kept smooth and plain. As to the properties of the soil, no particular requirements need be mentioned except that it shall provide the proper nourishment for the grass. For the most part the downstream toe is equipped with a reverse stone filter to prevent the debouching seepage water from producing piping or leaching (figure 21).



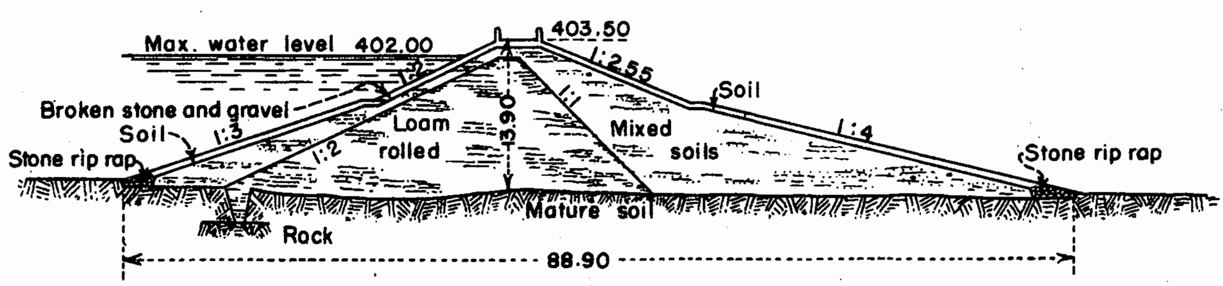
FLOOD RESERVOIR IN ZACKEN NEAR WARMBRUNN (SILESIA) 1906-1909

FIG. 22



FLOOD RESERVOIR ON THE KATZBACH NEAR
SCHONAU (LOWER SILESIA) 1907-1911

FIG. 23



FLOOD RESERVOIR ON THE KATZBACH NEAR
OBERKAUFFUNG 1931

FIG. 24

X-D-1474

Similarly the crown of the dam - assuming that it has a gentle slope - and occasionally also the upper part of the upstream slope beyond the reach of the water may also be covered with sod (figure 16). If the reservoir is purely for flood control purposes and is only rarely filled it may be sufficient if the dam is constructed of good material, to cover the entire upstream slope with a dense blanket of sod (figure 22), or with a pavement as is the practice in river and sea dikes (figures 23 and 24).

Over as much of the dam as is permanently under water, and particularly in the range which is subject to fluctuating water levels, a revetment is required which will provide protection against the attack of waves, drying out of the impervious layer, frost, and ice. Protection against current movement is generally important only in the vicinity of flood-water outlets in some dams, whereas on the other hand it may frequently be necessary where power intake canals are immediately adjacent to the dam.

The usual form of a protecting layer consists of a blanket of material laid up in the manner of the reverse filter (compare figures 3 and 16) in the following order: coarse sand, gravel or stone chips and crushed rock, and a filler of fines or of mortar. The whole serves also to load and hold down the impervious layer. The more rounded rock (rubble or river gravel) rolls easily into holes or cavities that may be formed by the impact of waves or from the ice. A filler of sharp-cornered broken stone, particularly if it be carefully wedged with stone slivers, is more stable; however, unnoticed cavities may be formed under the arched deck from the washing action. These then become danger points in the impervious layer. This is even more possible with pavement on sand and pavement on mortars. On the other hand the smooth pavement is least subject to the attack of ice. The monolithic concrete apron is treated in the next section on dams of uniform materials (compare page 9). As construction material for the protection layer on the upstream side, a coarse sand which will not slough and stones of all kinds so long as they are water and frost resistant, are well adapted. The greater their weight the more valuable will they be. In stones there will be found a much greater difference in densities than in soils (basalt 2.8 to 3.3 tons per cubic meter, diabase, diorite about 2.9, granite, syenite, gneiss, porphyry about 2.8, grey wache 2.5 to 2.8, slate 2.7 to 3.5, sandstone 1.9 to 2.9, limestone 1.5 to 3.0 tons per cubic meter). If the impervious layer lies more nearly in the interior of the dam or in the core of the dam, then the filter type of protecting layer is not so important; the protecting layer acts then primarily as a load or as a surcharge and as an equilibrant against sloughing of the saturated dam body; a high shearing resistance is therefore important (compare figure 4).

(d) The Dam of Uniform Material

In the dams in which the different functions are not separated into the different portions of the dam, that is, dams made of uniform soil throughout, three cases must be distinguished: either the soil is highly impervious, weakly pervious, or very permeable.

Very impervious are fat clays, fat loams, clay-like muck and marl, so much so that they may be utilized in dams without joints or layers. Much has recently been published concerning the formation of more or less dense laminations during rolling, the so-called "roller skin" and of the danger of their presence (14). In contact with the water in the reservoir these soils rich in colloids may absorb certain quantities of water (compare page 12). Therein lies the danger of their slipping or sliding. This consideration is less important for reservoirs only briefly filled, such as the sea dikes which are often composed of marl; for the movement of water in very dense soils is extraordinarily slow so that in this case the body of the dam never achieves complete saturation. Another danger of very fat soils is that of the formation of fissures as a result of very slow and long-continued drying. This, however, can be prevented with an adequate covering layer. Also the rather large settlement of such cohesive soils should be noted. The moisture conditions and the changes in the moisture conditions of dams with the lapse of time may be investigated by means of ground water observations. This method which is useful in permeable soils is impossible in very dense soils since these exhibit no distinct ground water surface and no pronounced saturation line (see page 16). Because these dense colloid soils are very unreliable in their relationship with water, special and more dependable seal become necessary. The dam is then no longer a uniform dam. Fat clays and loams may be diluted with additions of sand, a process which requires very careful work.

In the weakly permeable soils of uniform dams which act as slow filters, the grain distribution must be so favorable that the finest particles are not washed away; the voids of the coarsest soil framework must be well filled with fine grains. The saturation line which in this case is easily distinguishable must debouch very low (on the downstream slope), a feature which is achieved by the proper design of the dam cross-section; the quantity of seepage water (and thus the loss from the reservoir) must be small. The toe of the dam cannot be permitted to soften. The entire mass of such a soil must be uniform throughout so that during construction no permeable veins and pockets may be formed. The practice of dumping lends itself to this condition, for the coarse particles roll to the foot of the heap, and it is very difficult in practice to correct the effect of such deposition. Only in rare cases can such thoroughly uniform fill material of the most favorable proportions be obtained from the borrow pit by mass production methods. Neither do the most recent depositing machines which gather up and redistribute the loosened soil afford complete safety against the formation of such laminations, veins, or pockets in the body of the dam.

Through the deposition of fine sedimentary material on the slope of the embankment or through inundation, or as the result of the precipitation of irreversible soil colloids (iron oxyhydrate, silicic acid, or humus compound) the body of the dam which acts as a slow filter will gradually and eventually seal itself.

If such a dam of the same impermeable type is constructed by hydraulic methods it is possible that the unavoidable sorting of the soil according to the grain size may cause the deposition of the finer parts on the upstream side and of the more coarse and permeable parts on the downstream side. This method is not yet used in Germany for reservoir dams; however, in combination with a hydraulic dredge, transportation from borrow pits by means of wagons, and hydraulic distribution and sorting at the dam the method is well adapted to the construction of high canal embankments (16).

To construct a highly permeable dam contradicts the implicit purpose of the structure; namely, to impound water. Permeable dams may be used for clarifying or settling basins in connection with ore dressing plants and are designed to filter the water brought to them laden with mineral particles. Such dams cannot, therefore, be tightly constructed. Their height is often considerable, but the free depth of water nevertheless is quite small since the lower part of the reservoir is filled with the precipitated sediments. The body of the dam consists of coarse stone fragments affording considerable friction. Consideration must be given to the sliding effect of the percolating water because of its steep gradient. For such embankments of highly impermeable material designed to transmit the water any special seal can of course not be permitted.

In other lands dams of this type are frequently constructed of boulders or gravel, but in Germany seldom; for example, the sewerage and equalizing basins of the Middle Isar; more often perhaps for embankments of large power canals (compare figures 7 and 8). For reservoirs, the seal consists of a concrete apron placed upon the slope either with or without a cutoff wall of sheet piling; for canals, a concrete shell generally encloses the entire cross-section. In many cases an additional seal of a layer of cohesive soil is added. In this event the dam can scarcely be considered as one of uniform material.

Because of the unpreventable settlement of filled soils there always exists the danger that fissures and open cracks which may admit large quantities of water may be produced in the inflexible concrete apron. Also large concrete surfaces can scarcely be made watertight. The percolating water may transport the finer particles of the body of the dam further toward the interior and downward and thus produce cavities under the concrete apron so that after a certain amount of the progressive extension of these cavities the concrete crust may collapse under water pressure. So much water may then penetrate the dam that it may be quickly destroyed. The possibility must also be considered that because of the presence in the otherwise permeable material of the layer of relatively impervious (containing loam) material a region may be formed under the concrete shell which can retain the water. As long as the reservoir surface remains constant this will not be a source of danger; nevertheless if the reservoir level should drop suddenly the concrete crust may be broken off by the excess water pressure on the under side. This may initiate the destruction of the dam or embankment.

The hydraulic smoothness of the concrete paving of the embankments of power canals is of great importance in conserving the hydraulic head. In navigation canals the concrete shell must be protected from injury with a thick layer of sand so that the reduction which would result in the resistance to the passage of ships without this layer is of no consequence.

PART II

THE PHYSICAL PROCESSES TRANSPIRING IN THE BODY OF THE DAM

(a) Soil and Water in General

Before discussing in detail the particular properties of soils suitable for the impervious layers, the supporting body and the protecting layer of the dam, and the methods of testing, certain general remarks must be presented concerning the relationship existing between soil and water; for these relationships are decisive in the appraisal of the suitability of the soil for use in an earth dam. The extraordinarily large number of investigations in the field of soil science are initiated for the most part from the agricultural point of view and treat the soils as a habitat for plants, whereas from the point of view of construction materials soils have been much less investigated. From both standpoints the physical and the chemical conditions are naturally the same; in the application of the particular properties and in their significance, however, there exists a very great difference.

Every natural soil from the point of view of construction is a mixture of mineral particles and water and possibly of air. The ground water contains dissolved mineral salts in various kinds and quantities, sometimes also carbon dioxide. Because of their low specific gravity and their behavior with relation to water, soils with considerable quantities of organic origin (marl, peat, muck, etc.) can generally be ignored in connection with dam construction as has been mentioned; on the contrary organic materials which may be dissolved in water (humus acids and the like) may exert an important influence (compare page 21).

It is convenient to distinguish two principal types of soils: granular, noncohesive; and sticky, cohesive. Between the two are all possible types of the transition forms, each according to the proportion in which the two types are mixed. In connection with the granular types of soil are met the simple manifestations of capillarity - in fact for the more coarse soils the effect of water except for uplift may be neglected; on the contrary careful colloid research must be applied to cohesive soils with their large proportion of colloidal particles in order to obtain a knowledge of their phenomena.

The testing of soils with regard to their grain size distribution is therefore fundamentally important.

Soil science distinguishes the following sizes of grains in accordance with the internationally accepted Atterberg classification (6):

	Diameter	Diameter
Pea gravel, gravel (rounded), rock (angular)	over 20 mm.	
Grits, gravel (rounded), and fragments (angular)	20 - 2 mm.	20 - 6 mm. coarse 6 - 2 mm. fine
Sand	2 - 0.2 mm.	2 - 0.6 mm. coarse 0.6 - 0.2 mm. fine
Meal sand (Swedish "Mo")	0.2 - 0.02 mm.	0.2 - 0.06 mm. coarse 0.06 - 0.02 mm. fine
Silt (German "Schluff")	0.02 - 0.002 mm.	0.02 - 0.006 mm. coarse .006 - 0.002 mm. fine
Colloid clay (Gumbo)	$<$ 0.002 mm. $< 2\ \mu$	2μ - 0.1μ (microns) 0.1μ - 0.001μ (millimicrons)

Particles smaller than 0.001μ = 1 mm (Nanno meter) are considered to be molecular components of pure solutions

(1) Granular Soils

In noncohesive granular soils water occurs in three different forms; namely:

1. Ground water.--This fills all the voids completely, is subjected in general only to gravity forces and during flow to friction, and is analogous to open water with a gradient dependent upon the resistance to flow (width of void, volume of void, density of deposition). It may also be transmitted from points of higher elevation through underground channels under pressure (artesian water). The surface forces of the soil particles are not noticeable.

2. Capillary and Tensile Water.--*This fills the finer voids of the soil above the ground water level either completely or in conjunction with air and water vapor, is held above the ground water level against the air to a height corresponding to the so-called capillary height and in contact with the air forms menisci which produce tensile stresses in the water. In relation to the ground water level it therefore exhibits negative pressures. In addition to the gravity forces, the friction, and the surface tension opposed to the air, there occur also in this "tensile water" to a certain extent the surface forces of the soil par-

* Some experimenters distinguish further between these two forms of water (6).

ticles; these surface forces hold the water fast in a layer thin in comparison w th the dimensions of the soil particles whereas otherwise it would move freely in the void. Additional water which may be added flows into the ground water as "percolating water."

3. Adsorbed or Hygroscopic Water interspersed with air in the zone above the tensile water.--This is subjected only to the forces of circumstance and to the attraction of the soil particles, but not noticeably to the gravity forces and friction. The layer of water immediately next to the soil particles is in a compressed state and can therefore be separated only with great difficulty (partly by severe heating); the total thickness of the water shell depends upon the chemical nature of the soil particles and upon the electrolyte content of the water as well as upon the external pressure; the air in the soil is in equilibrium with the vapor pressure.

In addition to the fluid and the vapor formed the water may also take the form of ice (see page16).

(2) Cohesive Soils

Cohesive soils are distinguished by their content of soil particles of colloid size (one micron to one Nannometer). This proportion in the mixture fixes almost conclusively the magnitude of the boundary surfaces between the solid phase (soil particles), the fluid phase (water), and the gaseous phase (water vapor and air). The mutual behavior of the phases of the system rests, however, upon the boundary surfaces alone

Zunker (6) gives the following table of developed boundary surfaces for one gram of solid material of specific weight 2.6 grams per cubic centimeter for spherically shaped particles. The overwhelming importance of the finest portions is well illustrated.

TABLE OF DEVELOPED BOUNDARY SURFACES

Size Classification	Diameter Limits mm.	Average Diameter mm.	Phase Boundary Surface cm.^{2}/g.
Sand	2 - 0.2	0.53	44.6
Meal sand	0.2 - 0.02	0.053	446
Silt	0.02 - 0.002	0.0053	4460
Colloid Clay	.002 - .000001 (= 2 μ - 1 nm	0.0000077	2,990,000 (= 229 m.^{2})

A sharp delineation between the coarse-disperse system of granular soil types and the colloid system of cohesive soils does not exist because of the infinitely various proportioning; the properties overlap. Whereas the soil particles down to the size of the silt fraction are still so large that they cannot be stimulated into motion by heating of the molecules of water, the finer particles of the clay fraction are subject to Brownian movement and produce osmotic pressure. Whereas with the silt also, the voids, even with the most dense possible compaction of the soil particles, are extremely large in relation to the magnitudes of the molecules of water and dissolved materials so that for the most part the water is free to move and thus the soil is more or less permeable, for the finest clays in their most dense state of consolidation the size of the void is no longer so significantly large in comparison to the dimensions of the soil particles. A very considerable part of the void water is securely anchored by the electric fields of the soil material and can move only with difficulty; the innermost layers of the water molecules are subjected to pressures of thousands of atmospheres. The permeability of very pure, densely deposited, fat clays may completely vanish and in any case is very small. With clay and loamy soils the considerable part of silt or sand in addition to the colloidal portion increases the breadth of the voids and with it the freedom of the water or the permeability. Also, it is of considerable importance whether a cohesive soil exhibits its individual grain structures, whether the colloid forces are deflocculated or dispersed, and whether the particles are densely deposited or exhibit a honey-comb structure with larger cavities in an otherwise dense soil mass. These things depend upon the manner in which the soil is deposited, upon the electrical capacity of the soil particles which may be influenced by the electrolysis of the soil water, and sometimes upon the lime or salt content. The chemical nature of the soil material because of its various chemical values or electrical capacities influences also the density of the laminated and firmly held shell of water about the soil particle and therewith its freedom of movement. Thus the univalent sodium carries a much thicker shell than the divalent calcium or the trivalent aluminum. Silicic acid in the form of quartz can retain the divalence of the water molecule in a satisfied condition only on the faces, edges, and corners of its crystal lattice and with very little force or in thin layers. The silicates of clay on the contrary, on whose corners and edges free charges - and these predominantly negative - of anions of the space lattice occur, and normally adsorb the positive water molecules and draw to them also the negatively charged cations of soil solutions (3) which on their part again lay on water to the extent permitted by their free excess charges (hydration). To completely satisfy the cations requires very considerable quantities of water; concentrated soil solutions may partly withdraw this blanket of water from the colloid particles (dehydration).

The amount of this water and water vapor (hygroscopicity) is used as a measure of the developed surface of the soil. This, however, is only acceptable under limited conditions since the quantity of this blanketed water as has been said depends appreciably upon the number of the sorbant particles in a unit volume, their chemical nature, and the number of ions appended to them. If the determination of the hygroscopicity is made according to the method of Mitscherlich (5) (compare page 21) the method is above criticism, for maximum hydration is always achieved and all the water which is required for the release of the entire heat of hydration will be blanketed upon the particle (3).

Colloid cations and anions of soil adhere to each other under the attraction of their opposing charges and form larger units with corresponding reduction of their surface areas and thus of the quantities of "water-blanket"; they coagulate or floc; these coagulants can be irreversible and form concretions in the soil; for the most part they are reversible and the soil will re-acquire its original quantity of water. Through the introduction of univalent highly hydrated cations (for example, sodium) through more highly valent less hydrated cations (such as calcium and aluminum) the soil particles may be drawn together with a loss of water which may lead to the crumbling of the soil; conversely the introduction of highly valent cations through univalent cations will produce separate grains (peptisation), and in the presence of ample quantities of water will lead to the dissolution of the soil. Slightly hydrated colloid particles are protected against floccing through the blanketing accumulation upon them of the highly hydrated colloid particles (protecting colloids; for example, colloid silicic acid, and humus substances).

These briefly sketched remarks have an important bearing upon the strength properties and the permeability of the soil since the soil in a dam is permeated with solutions of various kinds and under certain circumstances may be intentionally influenced by the addition of lime mortar or lime powder. To the same extent these chemical relations are important in laboratory tests.

Many investigators (2) attach considerable importance to the form of the particle in colloid clays as influencing their behavior; thin flakes such as those of clay, mica, talc, etc. yield much more under the influence of pressure than the more round particles such as quartz; in relation to their weight they have a larger surface and larger surfaces of contact; they tend therefore to deposit themselves in parallel laminations so that their plasticity and cohesion are augmented. Nevertheless the electro-chemical behavior of the element of cohesive and granular soils must be recognized as the principal cause of their various physical properties.

Water in the form of ice may have a unique effect in cohesive soils sometimes even in the impervious layer under the slope, whereas these phenomena do not exist in granular soils. In clays the expansion of the water during freezing does not lead to a uniform expansion of the soil. On the contrary the freezing water builds itself up in the places where the ice crystals first begin to form. Here larger ice crystals quickly form, forcing the clay particles aside farther and farther from the region - to a greater extent in proportion to the amount of under cooling - and other water particles draw in and are deposited as ice crystals. In addition to the increase in volume and to the loosening effect, there remains, after thawing, a network of fine and larger channels extending into the interior of the clay (12). There exist ice layers and upward pressures, and, during thaws, an increase in the water content and therewith a softening of the soil.

(b) Pressure and Pressure Changes in Soils Containing Water

The soil water must achieve equilibrium between the external load and the internal pressure of a poly-disperse system. If not, movement begins, partly in the water and partly in the soil. The equilibrium condition is called the "natural water content" of the soil.

(1) Granular Soils

In granular soils the sorbant capacity is very weak, the hygroscopicity is small, and the "water-blankets" are subjected to small pressure; the individual soil particles, separated only by a very thin shell of water, are in very close contact and therefore in a fairly stable position. This does not mean, however, that they have the most dense arrangement or the smallest volume of voids. Particularly in the case of very irregular grain forms, for example, needle-like, plate-shaped, or knobby grains there may exist a very bulky deposition with much greater void volume, according to the method in which the soil was formed. If the soil changes from the bulky formation to a more dense, then it settles with a reduction in volume which has been often observed. This change in the relative positions of the particles can be brought about through flowing water but particularly through vibration (shaking, tamping or oscillating). An increase in the static external pressure does not have so great an influence upon the rearrangement of the particles and upon the compaction; also the shrinkage of the already very weak cells of water with the increase in the pressure causes only a very insignificant increase in the density of the soil. On the other hand the increase in pressure may fracture the soil particles and produce elastic deformation which may after all produce an increase in the density of the soil. The volume of the water itself or its density will remain practically unchanged for any practical range of increase in pressures.

When soil is compacted either as a result of an increase in the pressure or by vibrating, the water becomes excessive; there is more present than is necessary to fill the pores. In granular soils it generally flows quickly away. Until this happens it is subjected to an excess pressure (a condition of over-saturation or of stressed void-water); the friction of the solid particles is reduced; the soil tends to flow or to float; it takes on the properties of a heavy fluid. Appreciable resistance to this flow is offered only by very fine and uniform soils; wherefore these are subject to sloughing. With a reduction of pressure granular soils take on very little water, it swells very weakly; since only the elastic compression and to some extent the water in the shell of the grain is recovered a compaction produced by the rearrangement and the fracture of the particles remains.

As long as the water is not forced back into the material (condition of under-saturation) tensile forces exist in the void-water which compress the soil; at the boundaries with the air and the soil, concave menisci are formed; also air enters the soil. The soil thus possesses a slight transitory cohesion. A withdrawal of the water by evaporation produces the same effect as a reduction in pressure.

The permeability of the soil and the velocity with which the water moves vary with the void ratio and according to the size of the voids.

The finest granular soils (dust, reworked loess, and silt), which in the pure state exhibit only very weak cohesion, can be extremely permeable. It is only exceptionally therefore that they may be used as sealing material since, as has been mentioned, under excess pressure they tend to slough; they must therefore be deposited between more permeable layers of material which will act as filters, a process which in practical construction may lead to difficulties. In dams of very great thickness their use may be considered. Chinese river dikes have been built of such loess where the material was convenient.

(2) Cohesive Soils

In cohesive soils, on the contrary, the sorbant capacity of the particles is great, the blanketing water shells are thick, and the pressure in them is very considerable. In order to shrink the water shells, very great increases in pressure are necessary. The water which is squeezed out can drain away through the fine outermost voids only very slowly, the more especially since the breadth of the pores is reduced by the blanketing water (condition of over-saturation of the soil). For the most part, the increase in pressure is transmitted perpendicularly to the walls of the void by the compressed void-water; the pressure between the surfaces of contact of the soil particles, and with it the internal friction, no longer therefore increases in the same ratio with the external forces as do the tangential stresses. A tendency to slough therefore exists in the body of the dam; that is, there exists in large masses of soil which tend to act together a tendency to move along distinct surfaces of slip. If in the course of time the excess water has escaped, then the soil has consolidated, its particles have grown closer to each other, and its volume has shrunk. The new equilibrium condition corresponds to the "natural water content" at the increased pressure. The quantity of the expressed water corresponds, during the initial time interval, to the amount of the shrinkage; during the second time interval water may escape through evaporation and the penetration of air in the soil without any noticeable increase in the shrinkage. In comparison with the original condition the shearing strength in the new condition of equilibrium has been increased.

If the external pressure is again released, then, as a result of the hydration of the colloid particles and of the elastic recovery, the soil again regains its water, the particles separate themselves from each other; the soil swells under the actuation of considerable internal pressure; in some cases it softens or becomes fluid in accordance with the nature of the elements in the soil and of the soil salts.

As long as equilibrium has not been achieved between the reduced external pressure and the complete saturation of the sorption complex of the soil through an increase in the quantity of the "water-blankets," tensile forces (osmotic pressure) are produced in the void water which tend to hold the soil particles in a more tight mass and thus cause the so-called "apparent cohesion" (condition of under-saturation of the soil).

In many cases the soil does not recover as much water when the pressure is released as was originally expressed; there remains a permanent increase in density as a result of the intensified attraction between the surfaces of the particles; in comparison with the original condition the pure cohesion has been increased.

Such changes have frequently been produced by natural evolutionary processes in the soil (burying under a deep deposit, geological faulting, ice pressure). Cohesive soils in the compacted state are often particularly impermeable and resistant to the attack of flowing water (for example, the application of boulder clay to the Hindenburg Dam Sylt-Festland, or in the sea dike of the Zuider Sea). Artificial working also increases the density of cohesive soils (tamping and rolling), although only to a certain degree since there is opportunity for lateral flow because the duration of artificial pressure is only short so that excess water has no time to drain away, and since upon release of pressure the natural water content in the volume may be recovered.

Such soils appear to be somewhat elastic.

If the cohesive soil was initially deposited in a loose state, severe shrinkage may be expected. Any considerable shrinkage caused by drying produces shrinkage cracks.

The addition of water to air-dried cohesive soils may lead to their fracture since the water penetrating with considerable energy may compress the air and produce a disruptive effect. Also the swelling of the external layers leads to a progressive scaling. Shrinkage cracks will not close again during a succeeding swelling if the soil in shrinking experienced a permanent compression.

As has been mentioned the permeability and the velocity of water in fat cohesive soils is very small; in the finest clays it practically vanishes. For this reason the condition of dangerous susceptibility to sloughing produced by over-saturation may endure for a long period, whereas on the other hand in the condition of under-saturation with its accompanying apparent cohesion the soil may remain stable for a long time.

In very fat clay soils there is no very well defined ground water boundary which can be measured in observation wells; although the soil may contain a large proportion of water its entrance into the air-filled space of the well is obstructed by its capillary surface tension; thus the concept of a saturation line is fruitless. This is worthy of note in connection with "ground water observations" in dams and foundations of such materials.

PART III

METHODS OF TESTING

1. During the exploration prior to construction the rock and soil affecting the dam are investigated as to their quantity, the degree of difficulty with which they may be obtained, and with which they may be prepared for use, and with regard to their suitability for the various parts of the dam; the detailed design of the dam can only be supported upon a basis of such results.

The accompanying problems of cost, etc. are not pertinent to this report.

2. During the construction of the dam continuous tests must be performed upon the material, it must be sorted and systematically distributed in the body of the dam, and in some cases portions must be excluded. The treatment of the material during placement must be carefully watched (mixing, placing, regulation of the water content and the like). Here quick tests are appropriate.

3. In the finished dam the vertical and horizontal movement of the soil and of the superstructure must be observed (settlement, consolidation, sloughing, bulging, cracking); likewise the behavior of the water in and under the dam (observations on water level, saturation, swelling, uplift) must be observed. Finally, there may be measurements of the soil pressures in the interior of the dam, temperature measurements and color and salt solution tests to determine the origin of the ground water, etc.

(a) Technical Testing Methods During Exploration and Construction

(1) Trenches, Shafts and Boring. The extraction of Soil Samples

The choice of a damsite from the geological (1), morphological, and hydrographical viewpoints is based upon the type of soil, its strength, its physical properties and upon its distance and elevation, as determined by the investigation. This covers the following points:

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(1) Leppla, A; Geolog. Vorbedingungen d. Staubecken (Geological Consideration Preliminary to Dam Design), Wasserbau und Wasserwirtschaft, 1908. Leppla, D.= Geolog. Voraussetzungen f.d. Errichtung von Talsperren in Deutschland (Geological Prerequisites to the Construction of Dams in Germany), Deutsch, Wasserurrkraftanlagen, 1924. (Geological Prerequisites to the Construction of Hydro-Electric Plants.)
Terzaghi, K.V.; Ueber den Einfluss untergeordn. geolog. Einzelkeiten auf die Sicherheit von Dammbauten (Concerning the Influence of Minor Geological Details upon the Safety of Dams), Wasserwirtschaft, 1930.

1. The difficulty of excavating the soil.

2. The strike and dip of the layers.

3. The ground water conditions.

4. The extraction of soil samples for laboratory testing.

The borrow pits are decided upon by means of trenches and bore holes. Despite the high cost, which is aggrevated if timbering and the exclusion of water are necessary, trenches and shafts are preferable since with these conditions mentioned under parts I, II, and III above may be directly tested in the thrust against the timbers and on the bottom of the shaft. Here too a mixing of the types of soils during the extraction of the samples need not be feared. The position of the strata and their water-bearing propensities may be clearly determined. Also the extraction of soil samples including carefully oriented samples in their undisturbed state is possible anywhere in the shaft or trench. The soil profile based upon scattered excavations should be confirmed and extended by boring, particularly in glacial deposits since in these the soils frequently change very rapidly in their distribution.

Concerning the fourth line of investigation noted above, in connection with the extraction, packing and shipping of soil samples for the purpose of performing laboratory tests, bulletins have been published by the experiment station in Berlin and by the "Society for Construction Methods";* These are briefly reviewed and somewhat extended here:

Soil bits of various kinds are used for extracting undisturbed samples from bore holes. John Olsson (2) describes a cylinder operated by hand with which soil cores of about 64 centimeters length and 4 centimeters diameter may be extracted. Under favorable conditions it may be used to a depth of about 10 meters. J. Ehrenberg** has constructed a cylinder which can be used within the 137 millimeter casing of a hole made with the usual rig, drilling shaft, etc., and which needs merely to be exchanged for the ordinary auger, to secure undisturbed samples from cohesive soils to any desired depth (a depth of 25 meters has already been attained). The ten-

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* Die Gesellschaft fur Bauwesen" - Trans.
(2) J. Olsson; Kolvborr. Tekn. Tidskr., Feb., 1925.
** J. Ehrenberg; Gerate zur Entnahme von Bodenproben usw. (Equipment for Extracting Soil Samples, etc.), Bulletin No. 15 of the Preussische Veruchsamstatt fur Wasserbau und Schiffbau, Berlin, 1933 - Trans.

centimeter thick core may be taken in lengths up to 60 centimeters. The sample is contained in a two-part brass shell with which the cutting pipe is equipped. This brass shell can be easily removed and serves also as a container for the shipment of the sample. For testing, the sample is easily freed from the two-part shell. During the breaking off and the raising of the sample it is held fast in the cutting pipe by means of a self-closing air valve above. The pressure of the hydraulic jack required to punch out the sample is measured and provides a measure of the bearing capacity of the soil in case it may be contemplated as a foundation.

Burkhardt (3) describes an apparatus with which the undisturbed cross-section of the subsoil may be secured and the material to be investigated brought to the surface so that the necessary investigation of the stratification and strength, of the ground water level, of the elevation of the rock, and of the eventual value of the material as revealed by its quantity, proportioning, grain size distribution, and purity may be made without uncertainty (this has been used to a depth of about 23 meters). The equipment is particularly adapted to the removal of samples from beds of conglomerate, rubble, or like stones.

The Committee on Foundation Research of the German Society for Construction Methods has prepared rules for the uniform description of soil strata. As a result, instead of the confusion which has previously existed, only a few clear and concise classifications which permit a unified and comprehensive description are now in use.*

(2) Laboratory Soil Tests

Laboratory tests on soils, insofar as they are concerned with their use in dams, are primarily directed along two different lines:

(a) To determine their stability in the slope, and
(b) To determine their relationship to water and air.

A certain overlapping exists between (a) and (b).

The decision as to whether a soil is suitable for the construction material in an impervious layer, in a supporting body, etc., is based upon the results of these tests.

Under (a), the shearing strength of the soil must be determined since from this is derived the maximum slope at which a soil will stand in a dam of given height and of the anticipated hydrostatic and hydrodynamic properties.

The shearing strength comprises the internal friction and the cohesive strength (tensile strength, or cohesion, or coherence). The friction is that portion of the resistance to sliding, distortion or fracture which exists between the irregular surfaces of contact of the soil particles and which is dependent upon the pressure upon these faces, and upon the nature of the faces themselves.

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(3) Burkhardt, E.; Zur Aufschliessung des Untergrundes (For Foundation Exploration), Bautechn., 1931, No. 17.
* Obtainable from the Deutsche Gesellschaft fur Bauwesen (German Society for Construction Methods), Berlin N.W. 7, Ingenieurhaus.

The pure or apparent cohesive forces depend upon the surface attraction of the soil particles and upon the viscosity of the highly compressed "water-blankets" (compare page 15). So long as the water content remains unchanged the cohesion is independent of the surface pressure. A reduction of the water content as the result of an increase in pressure indicates a loss of the less-compressed portions of this water and therefore an increase in the cohesion.

γ Grain Size Distribution

Friction and cohesion are closely related to the grain size distribution of the soil. Every investigation must properly begin with the separation of the soils into their component parts. This is indispensable in the description of a soil. Through this classification carried out by means of sieves, by means of elutriation, or through sedimentation, the grain size structure (distinct grains or crumby structure), the mineral content, etc. of many types of soils may be observed.

A mechanical analysis is given on page 26 of this report. The proportion greater than 20 millimeters was determined by direct measurement.

Cohesionless sands and coarse "meal" sand up to about 0.06 millimeters diameter can be separated by means of sieves. A sieving machine is best adapted to this purpose since by this means the personal element (as in hand sieving, particularly with a large number of samples) may be avoided.

The sieve consists of a wood frame or of sieving containers made of drawn brass pipe which are drilled with pinholes or equipped with a testing mesh up to size Din 1171. Sieves with round holes are made with hole diameters up to 0.5 millimeters. Wire mesh for testing sieves may be obtained in certain sizes between 6 and 0.6 (Din 1171) milometers clear mesh opening; that is, from 1 to 10,000 openings per square centimeter.

The fine meal sand, silt, and cohesive soils after being prepared (by being washed with a brush in distilled water and if necessary with the addition of an electrolite or treatment in a vibrator, etc.) are sorted into their respective grain sizes, elutriation or sedimentation methods. The elutriation method with a rising stream of water is satisfactory for the separation of grain sizes down to 0.01 millimeter, or, that is to say, 10 microns. Sedimentation in still water is used for the determination of grain size proportions in the range between 20 and 1 micron; this work must be performed with a temperature as nearly constant as possible in a thermostatically controlled room since temperature variations have a pronounced effect upon the result.

The most difficult part of the sedimentation or elutriation method is the preliminary treatment of the sample; its dispersion into its elemental components. Very fine soils rich in electrolite frequently offer a very great resistance to this separation. Laborious washing of the soil with distilled water and sometimes with the aid of a Pukall candle is often required. Whereas for some soils a dispersion machine may be of advantage in preparing the sample, with others this procedure leads to almost immediate floccing, a coagulation of the particles into balls or groups.

The large difference in the results of analyzing one and the same soil by the various methods results largely not from the insufficiencies of the methods themselves but from the diversity of the methods of preparing the samples. It may be appropriate to mention the technique here (4). It should be noted further that the velocity with which the particles descend has been demonstrated to depend upon the specific gravity on the form and upon the position of their centers of gravity; so that the cohesive soils comprising as they do mineral fragments of such various elements and forms and treated by means of the elutriation and sedimentation method are not separated according to their grain sizes but as Ramann (5) has declared, according to "their hydraulic properties."

The apparatuses of Schoene, Kopetzky, Schulze-Harkort and others are used in Germany for the elutriation process. For the sedimentation method are used the Wiegner apparatus, which has been improved in many ways, an apparatus designed by Sven Oden, the sedimentation cylinder of Atterberg, and the pipette apparatus of Koettgen, as well as one by Koehn.

The results of a grain size determination are given as percent of the dry weight comprised by particles of particular diameters, and plotted either as curves of successive percents or more generally as curves of accumulative percents. It is convenient to plot the grain sizes on a logarithmic scale in order to magnify the interval occupied by the very small but highly important fraction. Figure 25 shows such curves for various types of soils.

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(4) Report of the Second Convention of the Subcommittee for Agricultural Soil Research, Kulturtechniker, 1925 - Zunker, F.; Gebrauchsanweisung zur Bestimmung der spezifischen Oberflache des Bodens (Experience in Determining the Specific Surface of Soils), Kulbertechniker, 1925 - Hahn, V. v., Dispersoid-analyse (Dispersoid Analysis), Dresden-Leipzig, 1928 - Oden, Sven, Über die Vorbehandlung der Bodenproben zur mechanischen analyse (Concerning the Preparation of Soil Samples for Mechanical Analysis), Bul. 16 of the Geol. Inst. of Upsala, 1919 - Schubert, H., Einfluss der Vorbehandlung der Boden auf die Ergebuisse der mechanischen Analyse (Influence of Preparation on the Results of Mechanical Analysis of Soil Samples), Kulturtechniker, 1929, No. 3 - Gessner, H., Die Schlammanahyse (Sedimentation Analysis), Leipzig, 1931 (contains extensive reference).

(5) Ramann, E., Bodenkurde (Soil Science), Berlin, 1911.

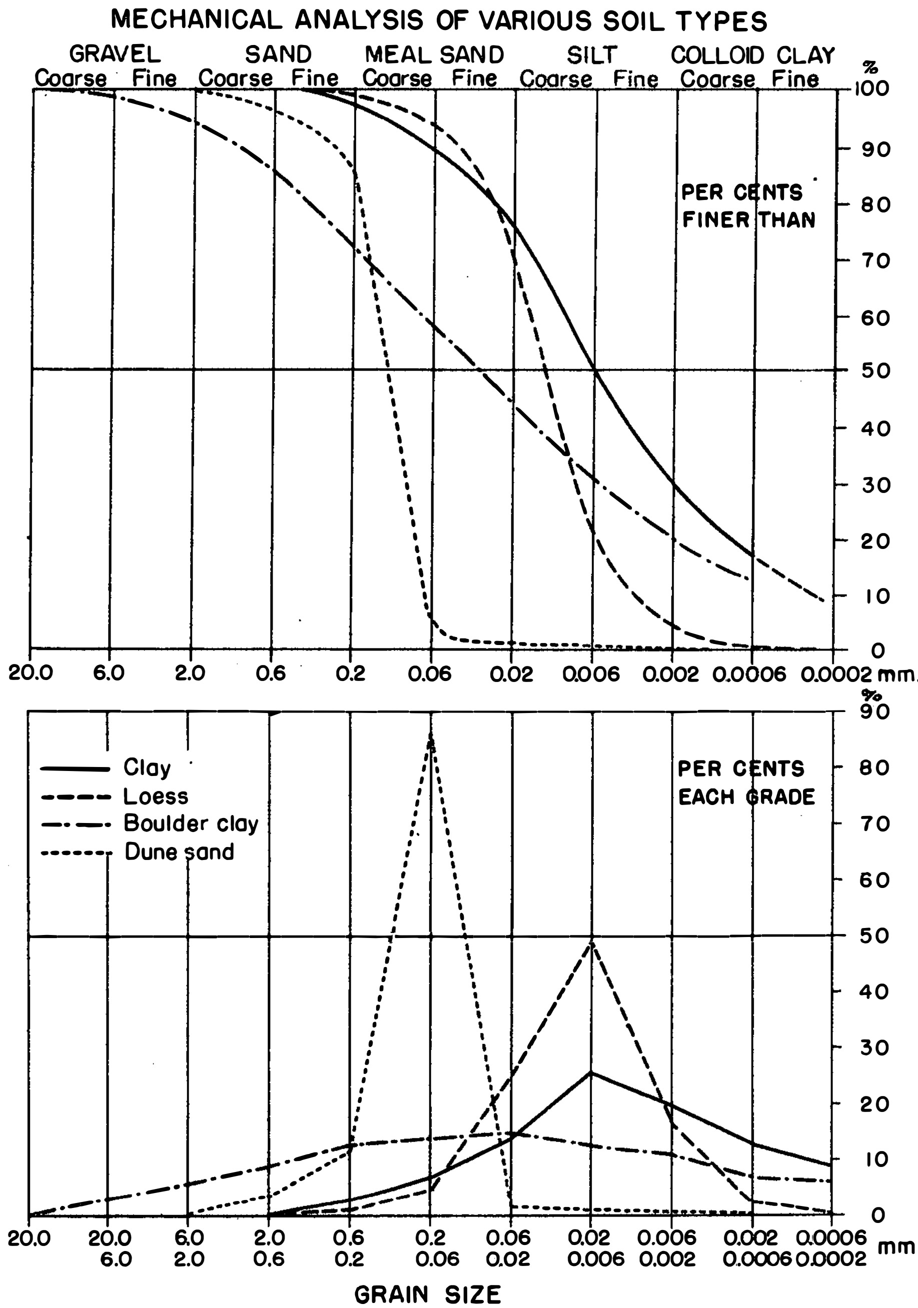

FIG. 25

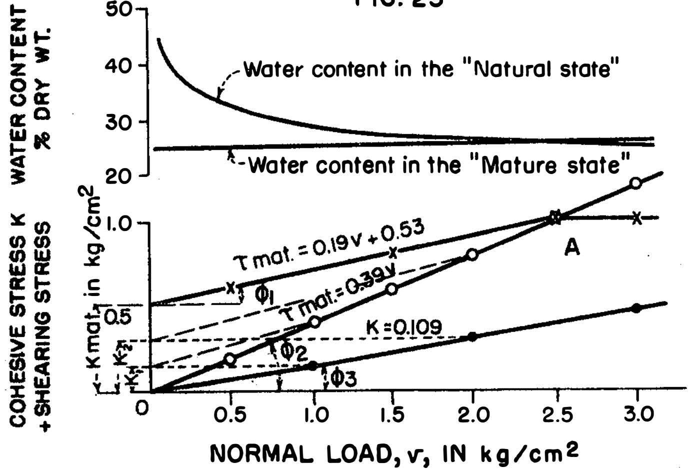

FIG. 26

SHEARING STRESS AND WATER CONTENT IN RELATION TO THE NORMAL LOAD

β Shearing Resistance

Perhaps the most important part of any soil investigation for earth dams is the determination of the shearing resistance. Soil types without tensile strength such as those free of loam or completely dry, or wholly saturated sands will take up natural slopes corresponding to their angles of friction. Soils exhibiting a tensile strength, however, in a condition of under-saturation, may tend up to a certain height corresponding to their cohesive strength to stand on much steeper slopes and even sometimes overhang. Since, however, with a constant water content the portion of the shearing resistance dependent upon the cohesion increases only with the first power of the height whereas the mass forces (weight and water pressure) and also the frictional resistances increase with the second power of the height, the influence of the cohesion upon the maximum slope diminishes with increasing height so that the slope necessary for the stability of an earth structure becomes increasingly flatter with the increase in the height and the slope finally becomes dependent upon the weight and the angle of friction alone.

In the determination of the shearing resistance the pressure and the water content must be shown to correspond with a probable or perhaps even possible conditions in the dam. As a rule there are to be determined:

1. The shearing resistance of the soil in "the mature state," and with a constant water content.

2. The shearing resistance of the prepared soil at the "natural water content corresponding to the soil pressure" (compare pages 13 and 14). In this way the portion of the shearing resistance to be ascribed to cohesion and that resulting from internal friction may be distinguished.

In plotting (figure 26) as an example the value of the shearing resistance τ_{mat} is first given (indicated with x) as it may be obtained for samples in the mature state with a constant water content (here about 25% of the total material). Second the coefficient of shearing resistance, τ'_{nat} , for the natural water content is plotted as determined in the following way.

After mixing with water the soil is introduced into the testing frame. Its original structure is thus destroyed as is also more or less the case in actual construction. In this testing frame the soil is loaded for several days with 0.5, 1.0, 1.5, 2.0, 2.5, 3.0 kilograms per square centimeter under water. For each of the various "conditions of natural water content" which are thus established the shearing resistance under the appropriate normal load is determined. In the graph, (figure 26) these points are indicated with an "o." From the two series of points may be determined the curves of shearing resistance (τ'_{mat}) and (τ'_{nat}). After each separate test the water content of the soil sample is determined and the water content curve plotted.

The τ_{nat} curve makes an angle ϕ_1 with the horizontal, and on the axis of ordinates intercepts the value of k_{mat} which is a measure of the cohesion of the soil in the mature state.

The τ_{nat} curve, since the soil has been thoroughly worked and saturated, intersects the origin of coordinate and makes an angle ϕ_2 (angle of shearing resistance) with the horizontal. The tangent of this angle, ϕ_2, is the coefficient of shear for the natural water content.

In cohesive soils the tangent ϕ_2 must still be divided into the portions describing friction and that describing cohesion. To determine the cohesive strength several samples of the soil must be preloaded with for example, one kilogram per square centimeter and sheared under a reduced load (that is in the condition of under-saturation), and a further series preloaded with two kilograms per square centimeter and likewise sheared under subsequently reduced normal load. In this way the dashed lines which intersect the axis of ordinates at the values k_1 and k_2 which, after values 1 and 2 of the normal load have been plotted, givenpoints for the x_{nat}- curve which makes an angle ϕ_3 (angle of cohesion) with the horizontal. Thus the cohesion becomes v times the tangent ϕ_3; the remainder, v (tangent ϕ_2 - tangent ϕ_3), is a frictional resistance.

In figure 26 the τ_{nat}- curve is a straight line. With cohesive soils rich in electrolyte a departure may be more often observed. As a result of the crumbling of the soil, tangent ϕ_2, in the lower range of normal loading somewhat below one kilogram becomes increasingly greater and attains here a value which approaches the coefficient of shear for a meal sand (compare figure 27, Soils I and II). The cohesion diminishes with the crumbling; nevertheless the change is very small; it lies within the range of error of the apparatus. With normal loading of somewhat more than one kilogram per square centimeter the formation of crumbs or lumps no longer has any apparent effect (compare also figures 26 and 27).

Figure 26 illustrates how the "mature soil" exhibits a larger shearing resistance than that in the "state of natural water content" in the range left of A as a result of its greater cohesion (caused by previous kneading under the pressure of mountains, ice floes, etc. and by the surface tension in the void water as the result of under-saturation); it shows also how thereafter the cohesive strength k_{nat} vanishes as a result of the disturbance in the texture and the saturation, and how the shearing strength of this soil falls off to τ_{nat}. With normal loads of 2.5 kilograms per square centimeter, the water content and the shearing resistance in the mature and in the preworked states are equal. With higher loading the "mature" soil will revert with the expression of water to the "natural water content." As has been mentioned on page 15, with cohesive soils this may take considerable time. During this time the shearing resistance is less than in the "state of natural water content" (note the path of the τ curve to the right of A).

The shearing stress required to produce sliding is, with the natural water content:

$$\tau = \mu v$$

In this $\mu = \tan \phi_2$ is the coefficient of shearing resistance.

In soils free from air the void ratio may be determined with the aid of the water content curve in saturated cases, and with it may also be determined the degree of compaction which the soil may have attained under the pressure.

Likewise to be deduced from the curve of water content is that water content with which cohesive soils may be most efficiently placed in dam fills in order that they may have sufficient compaction to withstand the subsequent loading and thus to prevent settlement and sloughing.

The tests described here are carried out by the Berlin Experiment Station with the assistance of the apparatus designed by Krey; in this connection reference is made in the following to certain literature on the subject (6).

The soil compaction tester of the Experiment Station for Hydraulic and Marine Construction (figure 29) likewise permits the determination of the shearing resistance even of rocky soils and with samples of large dimensions. The lateral fluid pressure is so reduced that the sample is fractured under the vertical pressure.

Equipment for determining the shearing strength has been designed by many others, for example, Mueller-Breslau (7), Nils

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(6) H. Krey, Erddruck, Erdwiderstand (Earth Pressure, Earth Resistance), 4th edition, Revised by J. Ehrenberg, Berlin, 1931.
B. Tiedemann, Die Bedeutung des Bodens in Bauwesen (The Significance of Soils in Construction), Handb. d. Bodenkunde (Handbook of Soil Science), edited by E. Blanck, vol. 10, 1932.
(7) Mueller-Breshau; Erddruck auf Stutzmanern (Earth Pressure on Retaining Walls), Stuttgart, 1906.

Westerberg (8), Streck (9), Stiny (10), Backofen (11), Casagrande (11a) and others.

The tests described above under α mechanical analysis and β shearing resistance, must be supplemented by:

γ Determination of the specific gravity;

δ Determination of the plasticity limit;

ε Determination of the hygroscopicity; and

ζ Determination of the lime content.

γ Specific Gravity

The determination of the average specific gravity of a grain element is made with the aid of a picnometer or with the apparatus designed by Erdmenger-Mann, as has been described in all text books dealing with soil science. The determination is only approximate since as a result of adsorption phenomena errors are introduced; but it is sufficiently accurate for engineering purposes. As a rule in mineral soils with a quartz content predominating a specific gravity of 2.65 will suffice for the calculation of the volume of voids and the density.

δ Plasticity

Atterberg thought to divide the soil types into plasticity classes according to their plasticity numbers. By plasticity number was defined as the difference between the water content (in percent of the dry weight) at the liquid limit and the water content at the plastic limit of a clay. The liquid limit is the upper limit of the

(8) Westerberg, N.; Jordtrycki kohasionara jordarster (Pressure of Cohesive Earths), Tekn. Tidskr., Stockholm, 1921, nos. 3 - 5.
(9) Streck, A.; Die Festig keitseigenschaften bindiger Boden (Strength Properties of Cohesive Soils), Deutsche Tiefbauzeitung, 1928, no. 33.
(10) Stiny, J.; Zur Schubfestigkeit der Boden (On the Shearing Strengths of Soils), Geologie u. Bauwesen, 1929, no. 1.
(11) Backofen, K.; Eine geotechnische Studie (A Geotechnical Study), Zentr. Bl. d. Bauwerwaltung, 1930, no. 18.
(11a) Casagrande, A.; Proc. A.S.C.E., Nov., 1931.
(11a) Kulka, H.; Versuche zur Bestimmung d. Scherfestigkeit verschiedener Sande u. Tone (Experiments to Determine the Shearing Strengths of Sands and Clays), Bauingenieur, 1932, p. 431.

plastic state. A soil pat about 4 centimeters in diameter and 1 centimeter thick is placed in a porcelain dish and divided into two parts by means of a special grooving tool so that the lower edges are barely separated. The water content of the soil is then increased until the two halves of the soil cake will just flow together to a depth of one millimeter along the line of separation under the effect of repeated tapping (12). Casagrande has designed a mechanical method for determining the liquid limit which eliminates all subjective errors (13). The Experiment Station V (Berlin) has adopted this device.

The lower limit of the plastic state is described as the plastic limit. The soil sample is placed on a piece of absorbent paper and rolled with the bare hand into a thread about 4 millimeters in diameter, broken in two, and rolled again. The plastic limit is the water content at which the soil crumbles into small pieces when further rolled.

The plasticity index or number can only be used for a rough characterization of the soil and for classification into soil groups (14). Thus, for example, the Finnish Geotechnical Commission gives the following plasticity indices:

Weak clays -	Plasticity no.	6 - 11
Moderately fat clays -	"	16 - 19
Fat clays -	"	20 - 26
Extra fat clays -	"	27 - 37

(12) Atterberg, A.; Die Plastizität der Tone (The Plasticity of Clays), Intern. Mitteilungen fur Bodenkunde, vol. 1, 1911, and vol. 2, 1912.
(13) Casagrande, A.; Report on an Investigation of the Methods for Determining Atterberg's Liquid Plastic, and Shrinkage Limits of Soils, their Physical Meaning and Practical Application. (See also Casagrande, A.; Research on the Atterberg Limits of Soils, Public Roads, Oct., 1932 - Trans.)
(14) Redlich-Terzaghi-Kampe, Ingenieurgeologie (Engineering Geology), Berlin, 1929.

ε Hygroscopicity

As has been mentioned the quantity of water which may be driven off from a dry soil in the form of water vapor (hygroscopic water) is a certain measure of the total surface of the individual particles (see page 14).

According to the method of Mitscherlich the soil rich in hygroscopic water is dried over phosphor-pentoxide at a temperature of 20 degrees centigrade over a ten percent sulphuric acid with about 96 percent of its full vapor pressure until a constant weight is reached. The hygroscopicity in percent of the dry weight of the soil is thus determined from the loss in weight (15). For similar mineral types this method yields good comparative values for the surfaces and thus for the fineness of the soil. In humus soils the mi-cellular surface within the humus substance is of some importance. "Soil types with the same hygroscopicity are not necessarily alike throughout in their other physical properties" (16).(See pages 13 and 14).

ζ Lime Content

The determination of the content of lime carbonate is important for the appraisal of the physical properties of the soil; it serves to identify the soil according to its formation and type. Loam and clay soils with a calcium content are classified as marl (loam marl, clay marl, calcium marl, etc.). In soils rich in clays calcium diminishes the cohesion (particularly Aetz calcium) whereas alkaline carbonate (and alkaline hydrate) increase the cohesion. Calcium produces deflocculation and crumbling in clay colloids; the alkaline colloids strengthen the salt condition of the colloids and lead to the stability of the individual particles. The marls as a result of their crumb-like structure are looser and more permeable than the corresponding clays and loams. According to various experimenters the soil crumbs attain a magnitude of from 0.01 to 0.03 millimeters (the order of magnitude of silt to fine meal sand). Because of its wide distribution in nature, lime carbonate is one of the most important sources of crumb formation. Determination of the content of calcium carbonate is made very simply with a Scheibler apparatus. Further chemical properties of the soil may be determined by chemical analysis (see later).

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(15) Mitscherlich, F.A.; Bodenkunde f. Land - und Forstwirts. (Soil Science for Agr. and Forest Economy), Berlin, 1923.
(16) Zunker, F.; Das Verhabten des Wassers zum Boden (The Relation of Water to Soil), Handbuck der Bodenlehre, vol. VI, Loc. cit.

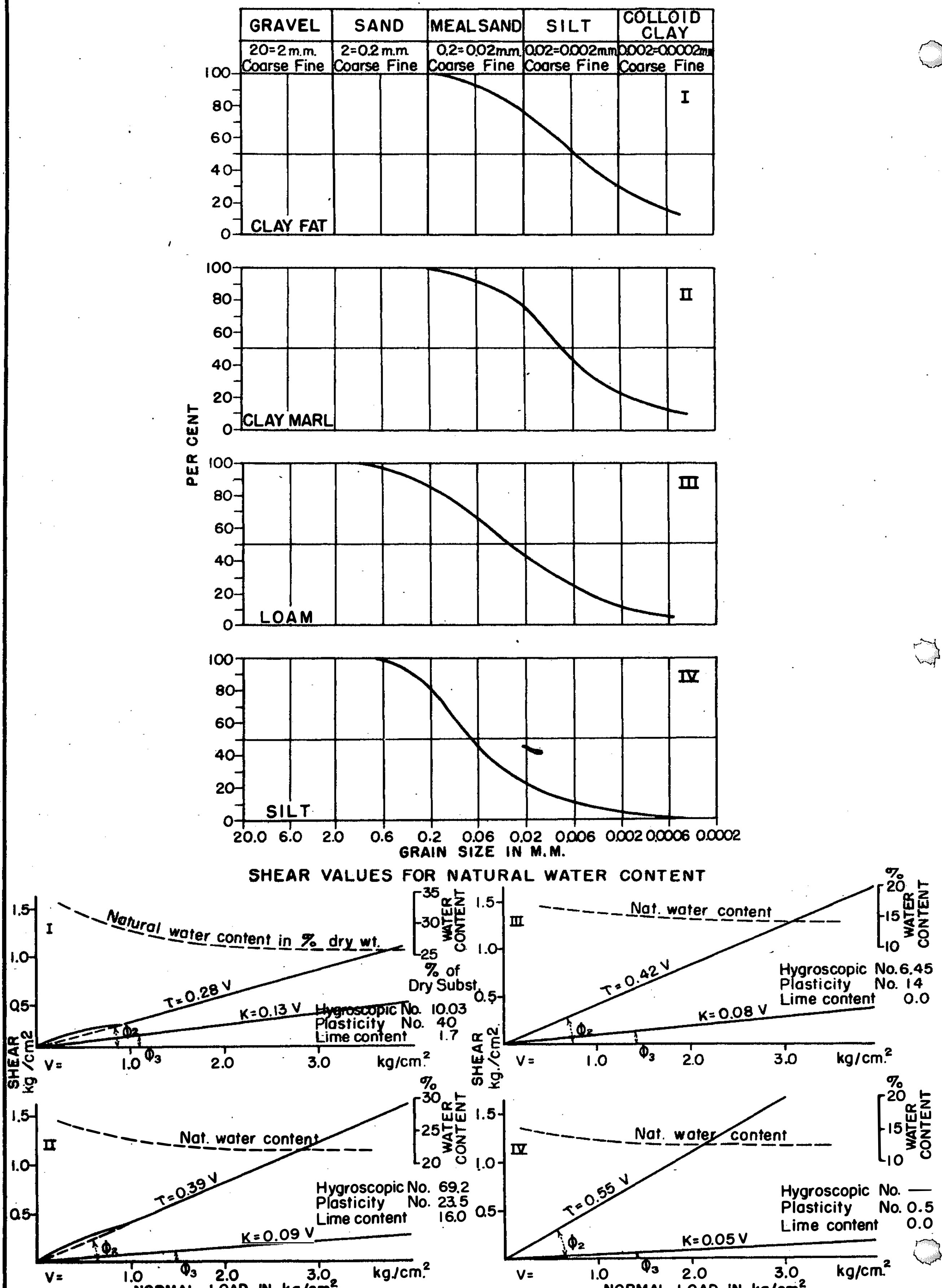
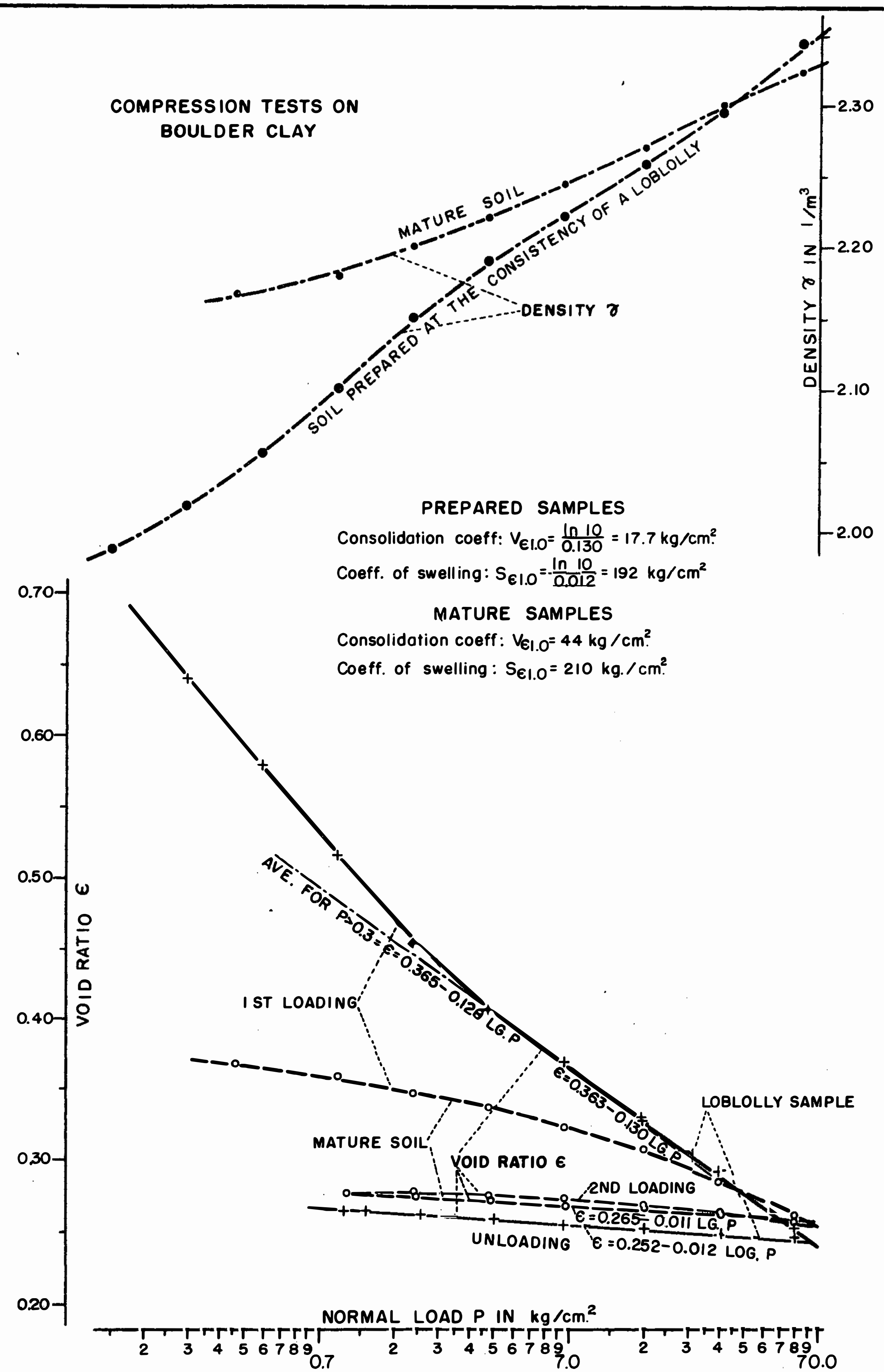


FIG. 27

MECHANICAL ANALYSIS AND SHEARING RESISTANCE OF 4 SOIL TYPES

X-D-1476



COMPRESSION TESTS ON BOULDER CLAY
MATURE SOIL
SOIL PREPARED AT THE CONSISTENCY OF A LOBLOLLY
DENSITY γ
DENSITY γ IN t/m³
2.30
2.20
2.10
2.00
PREPARED SAMPLES
Consolidation coeff: $V_{\epsilon 1.0} = \frac{\ln 10}{0.130} = 17.7$ kg/cm²
Coeff. of swelling: $S_{\epsilon 1.0} = \frac{\ln 10}{0.012} = 192$ kg/cm²
MATURE SAMPLES
Consolidation coeff: $V_{\epsilon 1.0} = 44$ kg/cm²
Coeff. of swelling: $S_{\epsilon 1.0} = 210$ kg./cm²
VOID RATIO ε
0.70
0.60
0.50
0.40
0.30
0.20
AVE. FOR P>0.3: ε=0.365-0.128 LG. P
1 ST LOADING
ε=0.363-0.130 LG. P
LOBLOLLY SAMPLE
MATURE SOIL
VOID RATIO ε
2ND LOADING
ε=0.265-0.011 LG. P
UNLOADING
ε=0.252-0.012 LOG. P
NORMAL LOAD P IN kg/cm²
2 3 4 5 6 7 8 9 0.7 2 3 4 5 6 7 8 9 7.0 2 3 4 5 6 7 8 9 70.0

FIG. 28

In the summary, figure 27, the characteristics of four types of cohesive soils are given in a single illustration:

Mechanical analysis;

The shearing resistance at the natural water content;

Specific gravity;

Plasticity index; and

Lime content.

Although similar in mechanical analysis to the clay, the clay marl exhibits significantly less cohesion but higher friction values, a characteristic which may be traced to the crumb structure resulting from the high lime content.

Consolidations

These values including the permeability and the capacity for absorbing water suffice in general for the complete determination of whether or not a soil is suitable for use in a dam and also for which part of the dam. The proportions of friction and cohesion in the soil are determined and the water content for various pressures established. If during the preloading of the sample during these tests the settlement was also measured with respect to time, then the consolidation diagram may also be plotted.

For these consolidation tests with restrained lateral expansion, V. Terzaghi has developed a very convenient apparatus described in his "Erdbaumechanik" (Soil Mechanics for Earth Construction). Figure 28 shows the results of such a test. The relation between the vertical loading and the void ratio:

$$\varepsilon = \frac{V_w}{1 - V_w}$$

(based upon the dry weight) is plotted in a simple logarithmic diagram. The consolidation index:

$$V_\epsilon = \frac{dp}{d\epsilon}$$

gives the change in the void ratio with the increase in the loading, it is the reciprocal of the direction factor (slope ?) of the pressure-void ratio curve. The settlement, Δh of a soil column of height, h, consists of the shortening, Δdh, of the individual layers, Δh, under the pressures to which they are subjected, p_{dh}; therefore

CONSOLIDATION APPARATUS OF THE PRUSSIAN EXP. STATION, BERLIN

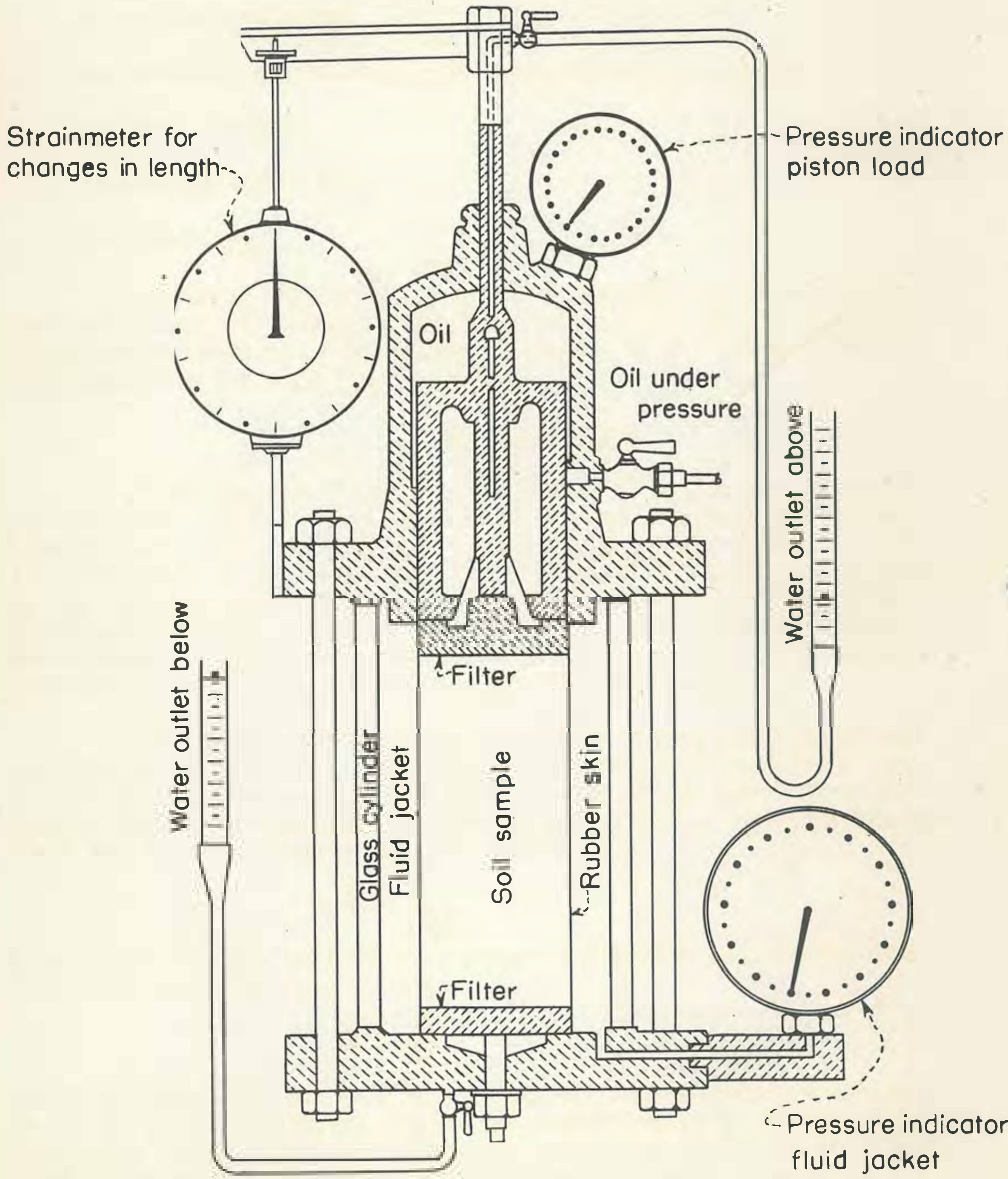
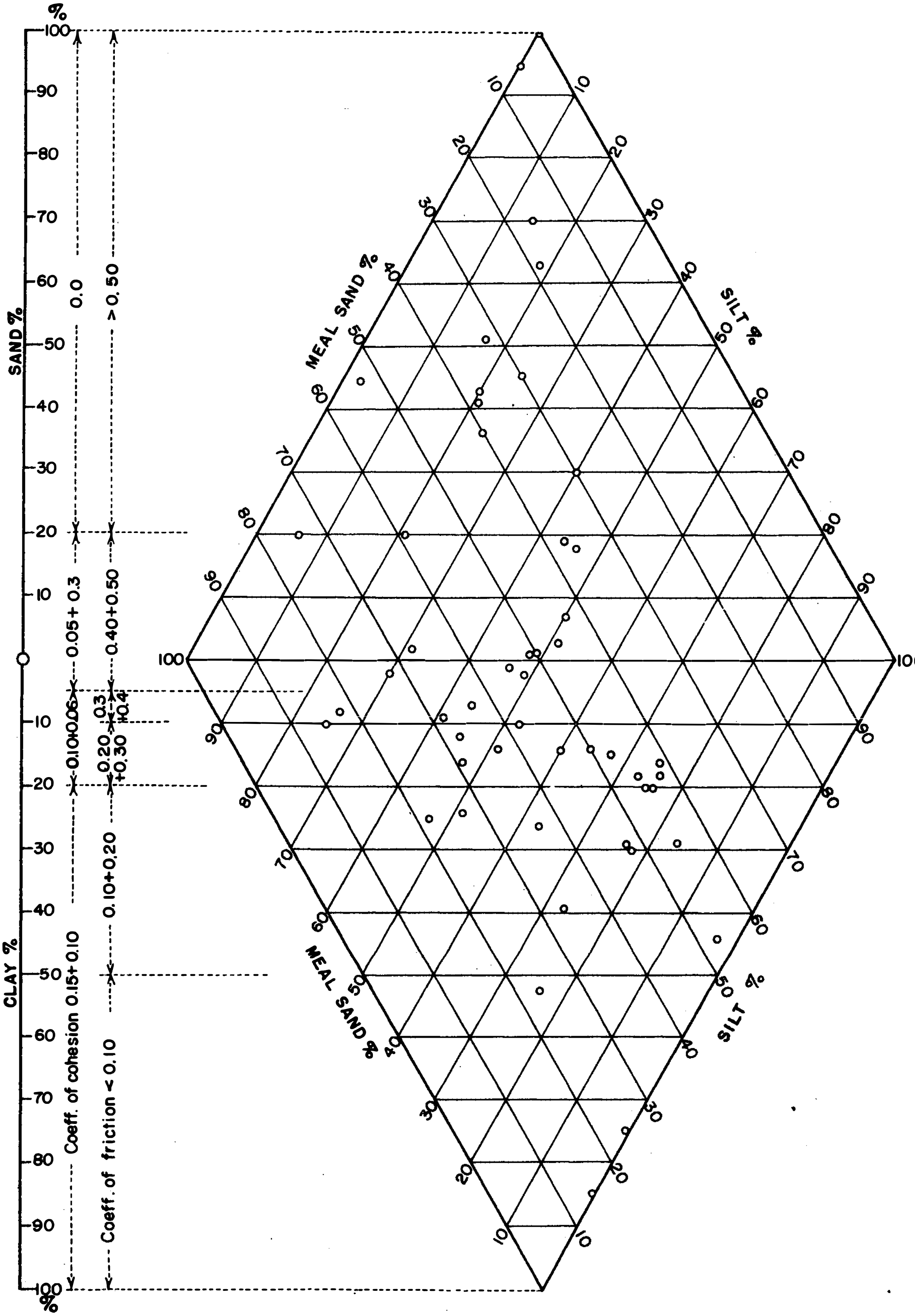


FIG. 29



COEFFICIENTS OF FRICTION AND COHESION IN RELATION TO MECHANICAL ANALYSIS, NATURAL WATER CONTENT

FIG. 30

X-D-1479

$$\Delta h = \Sigma_0^h \Delta dh \text{ and } \Delta dh = \Delta h \int_0^{Fdh} \frac{dp}{V}$$

if the pressure in the layer increases from 0 to Fdh. The consolidation coefficient, V, varies with p. The value, V, is obtained from the relation:

$$V = V_\epsilon (1+\epsilon)$$

If the load is removed from a layer of soil it expands. The coefficient of expansion, S, describes this swelling.

The Berlin Experiment Station has developed the apparatus shown in figure 29 for the same test. The compressibility of samples obtained with the Ehrenberg soil sampler described on page 22 or of samples in the reworked state is tested with it. The sample is surrounded with a rubber membrane which prevents direct contact with the surrounding fluid, but does not resist the distortion of the sample; and is surrounded with a shell of water so that the friction between the walls of the container and the sample will be obviated.

From these consolidation tests the relation between time and consolidation may be directly determined and plotted in the time-consolidation diagram. These tests extend over a period of several weeks with dense soils.

If the relation between the friction and cohesion coefficients and the mechanical analysis is known from a great number of tests, then an elutriation analysis may be sufficient for a first rough estimate of the soil.

In figure 30 (constructed by Tiedemann) some fifty fine soils (g ain size less than 2 millimeters) are plotted according to their mechanical analyses. The upper triangle contains the soil which consists primarily of sand, meal sand, and silt; some small percentages of clay were included in the silt; in the lower triangle the soil consisting of meal sand, silt, and clay are entered with small percentages of sand included in the meal sand. For every soil there is thus established a particular point in the interior of the triangle. The friction coefficient for these fifty fine soils and for a number of the cohesive soils and the cohesion coefficients are then determined so that zones can be set up within which the friction and cohesion coefficients of the soil are alike within narrow limits; these are to be extended by the addition of further points as time permits. This representation gives a numerical classification of the soil types and permits an evaluation of the friction and cohesion coefficients and thus of the shearing strength on the basis of the mechanical analysis alone.

Permeability

Investigations of permeability are carried out partly in the reworked or compacted state, that is, after the original structure of the soil has been disturbed, and partly in the mature state. In the latter case the sample which has been removed from the soil, or the core is placed in a glass cylinder and the space between the wall and the sample filled with a skin of paraffin or with an elastic cement solution treated with formalin. Further details are shown in figure 31 (17).

The coefficient of permeability, which has a constant value only when the velocity of the percolating water varies in direct proportion to the gradient (Darcy's law), is then the quotient of the flow through a unit cross-sectional area of the soil layer and the hydraulic gradient. The assumption that Darcy's law is valid does not hold with very permeable coarse grained soils.

Figure 32 shows the relationship between the coefficient k and the void ratio ε for clay soils.

Further Chemical Tests

Salts occurring in certain mineral soils, $CaSO_4$, $MgSO_4$, K_2SO_4, Na_2SO_4, $(NH_4)_2SO_4$, may cause injury to concrete work, pipe conduits, deck plates or aprons, etc. which may lie within the dam, if water is allowed entrance. Magnesium carbonates and magnesium sulphate may produce a so-called magnesium pitting (cement bacillus). According to Gessner (18) sulphates in the soil are injurious to cement if the hydrogen chloride solution contains more than 0.2 grams of SO_3 in 100 grams of soil or not more than 2 percent of magnesium oxide in the magnesium salt contained in the air-dried soil (19). Danger of the destruction of the cement exists with the exchange acids if in 100 grams of soil and 200 cubic centimeters of a normal sodium acetate solution, more than 20 cubic centimeters of one-tenth normal caustic potash is required for neutralization.

Concerning the relation of the soil to the air, tests should be made to determine the weathering properties of the mineral constituents of the soil. The chemical method for discovering the presence of injurious components in the water or the soil are not within the scope of this report.

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(17) Redlich - Terzaghi-Kampe, p. 329, Loc. Cit.
(18) Gessner, H.; Die Ursache Der Betonzerstörung in Mineral böden "The Causes of Concrete Deterioration in Mineral Soils," Transactions of the International Congress for Soil Science, Washington, 1927, no. 4.
(19) Grün, K.; Chemische Widerstandsfähigkeit von Beton (Resistance of Concrete to Chemical Attack), Berlin, 1928.

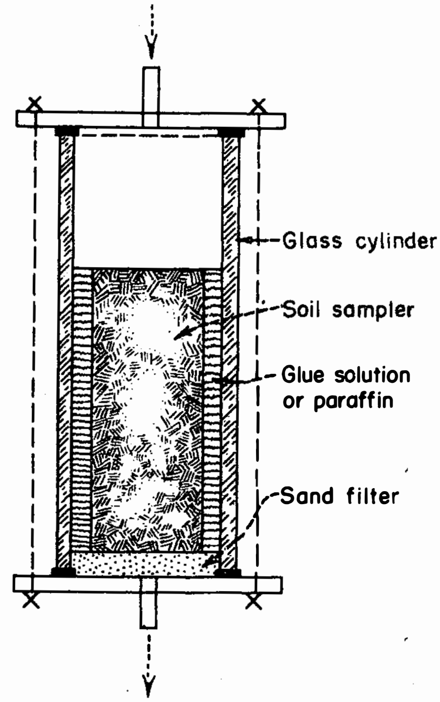
PERMEABILITY APPARATUS
Glass cylinder
Soil sampler
Glue solution
or paraffin
Sand filter

FIG. 31

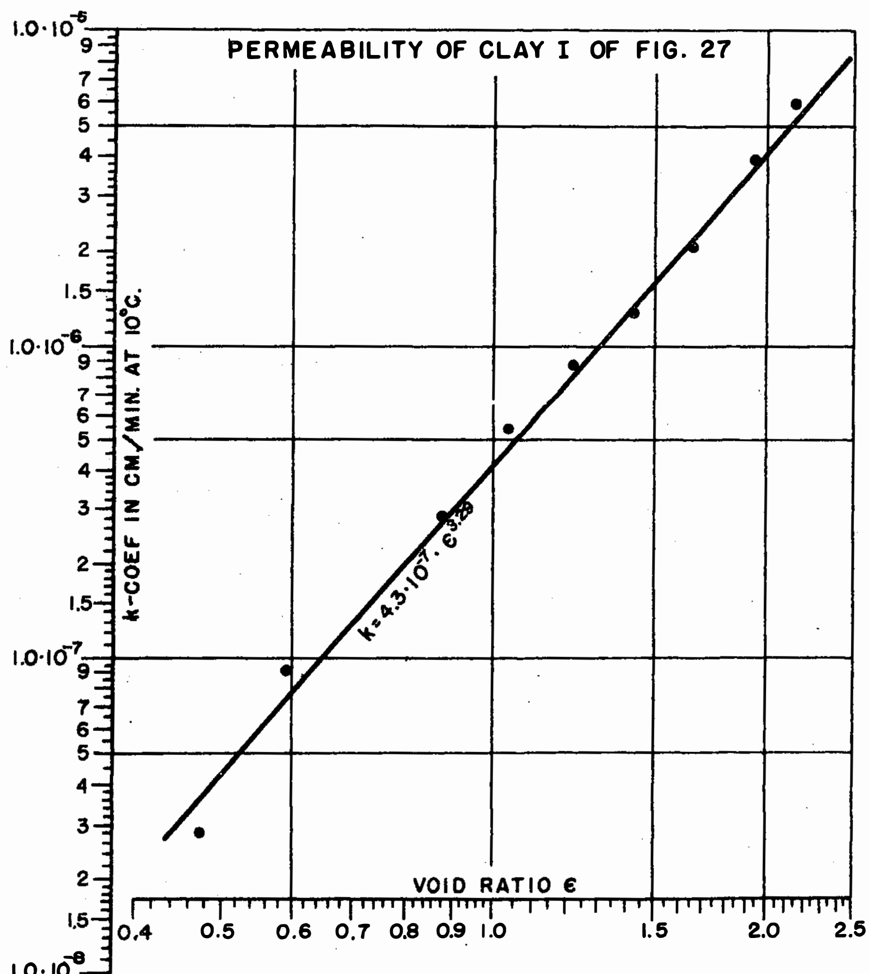
PERMEABILITY OF CLAY I OF FIG. 27
k-COEF IN CM./MIN. AT 10°C.
$k = 4.3 \cdot 10^{-7} \cdot \epsilon^{3.29}$
VOID RATIO ϵ
$1.0 \cdot 10^{-5}$
$1.0 \cdot 10^{-6}$
$1.0 \cdot 10^{-7}$
$1.0 \cdot 10^{-8}$
0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.5 2.0 2.5

FIG. 32

As yet no particular methods have been developed by us for investigating the resistance of soils to the effect of frost.

The approach to a complete investigation of a soil sample in all of the directions described above is long and time-consuming. The extent to which these experiments will be applied to soil samples will only be such as necessary to learn enough about the various soil types within the borrow pit area for the design of the structure. The sifting, the mixing, and the control of the compaction necessary during the construction of a dam must be accomplished with mass methods which are simple and which can be carried out at the construction site without elaborate apparatus. For this reason the two short methods, the evaluation of the soil according to its mechanical analysis and plasticity index, were given above.

The following is an extract from a report on construction control placed at our disposal by Dr. Collorio, director of the Harz Water Works Company (Province of Hanover).

(3) Construction Control

"The scope and organization of a construction control for the Soese Dam will be described.

"The type of dam is illustrated in figure 6. The major portion of the fill consists of river gravel of various composition; the layer on the upstream side of the core consists of loam in sloping layers. Construction control was organized as follows:

"(a) At the borrow pit (a dredge excavation) samples of various material were regularly extracted from excavations in each stratum and a quick test by sieving down to grain size 0.25 millimeters was undertaken. The disposition to be made of the soil was determined from the results of these analyses, and the construction foreman and the switchman at the dump were instructed into which track the trains to the borrow pit (dredge) should be shunted. The quick test was supplemented by a more thorough control including a full mechanical analysis.

"(b) Upon the introduction of the material and before and after it had been rammed into place with a special soil ram the compaction was measured and the consolidation calculated; if the consolidation (in percent of the height of the loose fill) was found to be sufficient, then that section was opened for additional fill, otherwise it must be further rammed (a quick test was made with a special survey crew). These quick tests were occasionally checked after certain intervals with a more thorough test; in addition, and at convenient but carefully referenced points, a soil sample of about one cubic meter's volume was excavated, the hole carefully measured, and the void volume of the soil was determined in the usual way (besides making a complete mechanical analysis). If the volume of voids corresponded to that specified by the designs, then the percentage compaction requirement was allowed to remain unchanged; otherwise a change was made. Such a case might be produced by a change in the proportioning and quality of the material.

"Conclusion

"As a result of our experience the following answer to the question propounded may be summarized:

"An efficient and economical design of a dam requires an exact testing and investigation of the construction material, just as an orderly construction of this dam requires careful construction control. The scope, nature and organization of these investigations have been described; they are no more difficult than for concrete structures, they have indeed certain external similarities to these more usual tests. The significant difference lies in the fact that the construction of an earth dam is adapted to the properties of the materials, whereas the proportioning of concrete is adapted to the construction. For this reason there is a great multiplicity of possibilities for the organization of the testing, and this explains also the impossibility of establishing hard and fast distinctions as to whether a soil is suitable or unsuitable. It will only be possible to establish such distinctions after long years of experience and then only for certain elements of particular construction forms."

(b) Testing Methods on the Completed Dam

(1) Movement in the Dam

Preparations for the measurement of the movements in the soil and of the auxiliary structures in the finished dam, as well as measurements of the percolation, must be made during the construction period by embedding reference points, constructing observation wells and observation galleries in the concrete core of the dam, and by introducing pressure cells into the interior of the structure.

Subsidence of the foundation as a result of the load of the dam and settlement in the dam itself may be measured by placing plates at various heights which are fitted with vertical shafts reaching to the surface. For protection against the settlement of the overlying strata the shaft should be surrounded with casings. The peak of the staff carries an elevation mark which is observed with a level. The observations should be begun as soon as possible during the construction of the dam and should be carried out continuously. For parts of the dam which are subjected to artificial compaction by means of tamping or rolling, only the subsequent installation of reference points need be considered for observation of the surface of the finished dam with respect to settlement, formation of cracks, and sinks (20) and then only if after careful inspection of the structure its condition is not found to be entirely satisfactory.

The horizontal and vertical movement of the core wall under the influence of percolation, etc. should be likewise observed.

The percolation through the dam may be determined by means of stand-pipes (compare page 21). In large dams with core walls and observation galleries, these facilities provide an opportunity for measuring the percolating water. The measurement of ground water temperatures may provide information as to their source.

(2) Dynamic Soil Tests

The dynamic soil tests by means of a vibrator such as Professor Hertwig (21), has developed in conjunction with the German Society for Soil Mechanics appears to serve a valuable purpose in the testing of the uniformity in the deposition of the dam. Hertwig explains how dams deposited by various methods were tested to compare their strength and density with that of the original soil. The materials consisted of sand and gravel. In connection with laboratory tests this method will probably also provide valuable information concerning the conditions of dams with cohesive soils and considerable water content.

(3) Water Content and Percolation

For quick determination of the water content of the soil at the borrow pit and in the dam after compaction, the electrical method of moisture determination may be applicable in certain cases (22); in addition of course to the extraction of the samples.

(4) Soil Pressures

In order to measure the compressive forces in the interior of the dam and the strains in the core wall caused by earth pressure and water pressure, pressure cells, of which there are several types, are required.

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(20) Laüfer, A.; Lassenbildung und deren Verheihung (Fissure Forming and its Prevention), Wasserwirtschaft, 1927, p. 560.
(21) Hertwig, A.; Die dynamische Bodenuntersuchungen (The Dynamic Soil Tests), Bauingenieur, 1931, p. 457 et. seq.
(22) Görz, A.; Über ein Tragbares Gerät zur elektrischen Bestimmung der Bodenfeuchtigkeit im Felde (Concerning a Portable Apparatus for Electrically Determining the Soil Moisture in the Field), Intern. Mitt. f. Bodenkimde, 14/35, 1924.

It has proved difficult to make such pressure cells sufficiently rugged to resist the rough treatment they experience during the construction of the dam.

Under considerably more favorable conditions than in the filled dam, pressure cells installed by O. Schaefer under the principal piers of the Niederflnow lift locks have yielded good results (H. Maihak Company, Hamburg).* The principle upon which the measurement is based is as follows: A steel sounding wire whose pitch is dependent upon the tension under which it is stretched is set at a particular tone; this pitch changes when the distance between the anchorage points of the wire - that is between points in the pressure cell or in the structure - is lengthened. The wire is set into vibration by means of an electro-magnet. The measurement of the pitch is made with a telephone receiver and a comparator wire.

The firm of Fuess-Steglitz has a pressure cell in the experimental stages designed for investigating the pressure distribution in fills and in foundations. The cell, about eight centimeters in diameter and about four centimeters high, carries a horseshoe-shaped magnet with a soft iron core on the inside of one face which is opposite to and separated by a very narrow clearance from a soft iron plate which is rigidly attached to the other face. Any change in distance of the face plates produces a change in the self induction of the magnet whose magnitude is measured with an appropriate connection and is indicative of the pressure (23).

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* "Ein neues akustisches Messverfahren usw." (A New Acoustic Measuring Method), Wasserkraft u. Wasserwirtschaft, 1934, no. 17; see also translation in possession of Mr. Lane - Trans.
(23) Cf. Pfeiffer, A.; Ein neuer Betondehnungsmesser (A New Extensometer for Concrete) Bauingenieur, 1931, p. 88.

PART IV

SUMMARY

The report takes up first the various types of earth dam cross-sections, in order to show what functions the various parts of the dam serve and to indicate the points of view according to which the soil should be tested with reference to the purposes which they are intended to serve. The presentation is illustrated with numerous cross-sections of recently constructed German dams.

A portion of the report is devoted to a treatment of the nature of the soil from the point of view of soil science and of its relation to water; and provides a clarification of concepts of the physical properties of the soil important in construction.

Likewise extensively discussed are the testing methods appurtenant to earth construction and to the preliminary exploration and construction of the dam with due reference to the many publications on the subject. The methods discussed show that it is today entirely possible in most cases to determine with complete certainty whether a soil is suitable as a construction material for an earth dam, and to predict its properties.

A very appreciable disadvantage of these testing methods is the long time required for their completion. For the current testing during the construction of a dam; therefore, methods must be devised which will permit the soil properties most important for the foregoing purpose to be determined with sufficient quickness and accuracy. Along with a more thorough investigation in the laboratory, simple determinations must be made at the construction site; both should serve to check and supplement each other. Close co-operation and a regular exchange of observations on the behavior of the soil must be sought between laboratory and construction crews.

From a knowledge of this fact the Prussian Research Institute for Hydraulic and Marine Construction (as has also been recognized for an even longer time in the Hydraulic Department for Hydrotechnical Experiments) has provided in its soil mechanics department, space in which engineers actively engaged in construction may be acquainted with its soil testing methods and where they may obtain an insight into the behavior of soils, and the necessary proficiency to perform tests at the construction site. Further, the experiment station has collaborated with the laboratories at construction sites in furnishing them with suitable apparatus for soil testing.

Finally, the experiment station begs that it be notified of all slides which may occur, settlement phenomena, etc., and that any drawings which may be made of these occurences be submitted to the station for inspection.

Theory and practice must here as well as in so many other technical fields, expand and bear fruit in many directions; only in this way can progress be achieved.

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(15) Grawell - Das Gefüge der Staudämme (The Composition of Dams), Die Wasserwirtishaft, Munich, 1921, p. 276.

(16) Tode - Das Spülkippwerfahren der 27 m hohen Dammstrecke des Mittellandkanales (Hydraulic Fill Methods on the 27 - Meter High Midland Canal Embankment) Bautechnik, 1932, p. 583.

(17) Hertwig - Die dynamische Bodenuntersuchung (The Dynamic Soil Tests), Bauingemeur, 1931, p. 476.

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The sources for Part III are given in footnotes.