GEOTECHNICAL INVESTIGATIONS FOR EMBANKMENT DAMS
EXPLORATIONS GEOTECHNIQUES POUR DIGUES

A.L. LITTLE, B. Sc. (Eng.), F.I.C.E.

Soil Mechanics Consultant to Messrs. Binnie & Partners, London, England.

SYNOPSIS Conventional site investigation methods, including geophysical methods, are inadequate for embankment dams. The paper discusses their shortcomings and recommends that geotechnical investigations for embankment dams be divided into three parts: Preliminary, Design and Construction. The emphasis in each stage is on different aspects but the investigation cannot be considered as completed until construction is also finished. Geological investigations constitute an important part of the whole.

INTRODUCTION

Embankment dams are the largest and most expensive structures built by man. Mangla dam, completed in 1967, contains 83 M cu. yds. of fill and cost £M35; Tarbela dam, now under construction will contain 175 M cu. yds. and is estimated to cost £M85. Utmost economy of design and construction are essential whilst, at the same time, since failure has such far reaching effects, safety is of overriding importance. Consequently, the closest investigation is required of details of geology and other site features which might, in a lesser structure be of little importance. Experience has shown that the conventional site investigation involving boring and "undisturbed" sampling is inadequate for embankment dams. This paper considers some of the shortcomings of past investigations and makes suggestions for an ideal arrangement.

PURPOSE OF SITE INVESTIGATION

For any structure, the purpose of the site investigation is to provide information on the geology and engineering properties of the ground for design and construction. So far as embankment dams are affected, these are geological arrangement of strata, strength, compressibility and consolidation characteristics, permeability, use as construction material. It is necessary to investigate the strength and permeability of the foundations in fairly sophisticated terms entailing a detailed consideration of the effect of stratification and of local geological variation and to evaluate the quality and quantity of available fill materials. The investigation should also be orientated to disclose hazards likely during construction such as instability of temporary excavations.

Considerations of reservoir watertightness are confined to the dam site.

STAGES OF SITE INVESTIGATION

Ideally, an investigation consists of three stages:

I Preliminary. The general features of the site are established; existing geological information is collected and, as far as possible, checked on site. Probable borrow areas and other materials sources are located. At this stage a limited number of boreholes/drillholes may be sufficient, supplemented by a few test pits in borrow areas. Some laboratory tests will be required on fill materials and possibly on foundation samples as well.

Geophysical methods of investigation (e.g. seismic refraction studies to determine depths to bedrock) are sometimes advocated for this stage. Perhaps the author's experience has been unfortunate but out of eight geophysical surveys at dam sites of which he has knowledge, only three have been successful or partially successful. It is his opinion that geophysical surveys do not at present give information in sufficient detail to be useful for this type of investigation.

At the end of the Preliminary stage it is common to produce a feasibility report on the whole project.

II Design. For this stage much more information will be required on foundation conditions and on behaviour and available quantities of fill materials. Pits will be necessary in the foundation of the dam with block samples. An intensive geological investigation of the dam site area will be necessary together with fairly detailed consideration of the regional geology. More laboratory tests will be required on fill materials and a careful assessment of the quantities available. Depending on the nature of the deposit, more material should be "proved" than is required for the dam.

III Construction. In the author's experience, no embankment dam has been constructed without changes, sometimes radical changes, in the design. These changes are usually due either to insufficiency of borrow material of the kind expected or weaknesses in the foundation undisclosed by the previous investigations. He believes that this is the common experience in dam building and that construction usually exposes shortcomings in design. Occasionally the shortcomings are first shown by a more or less serious failure during construction; usually the designers are more fortunate and the difficulties are detected in time to make adjustments.

PRELIMINARY INVESTIGATIONS

In 1913, boreholes were sunk at the site of the Silent Valley Dam in Northern Ireland. Construction was delayed and it was not until 1923, after the first World War, that work started. The Contractor had trouble in excavating the cut-off trench and by 1926 was in such difficulties that further investigations were made including three 16 ft. diameter shafts sunk in compressed air and further borings; these showed that bedrock was at a maximum depth of 196 ft. or three times deeper than had been previously believed. All the early boreholes had struck granite boulders in the deeper parts of the buried glacial channel and these boulders had been mistaken for bedrock. It was not until borings were taken at least 20 ft. into the granite that the true position of the bedrock was established. A new contract was negotiated and the cut-off trench excavation completed with cast iron segments (McIldowie, 1935). A somehwat similar mistake was made in the initial stages of the investigation for the River Neath bridge in South Wales where Pennant sandstone boulders were mistaken for Pennant sandstone (Carboniferous) bedrock. Fortunately, at this site further investigation was made before construction started; it was found that the rock was generally deeper, at one point over 60ft. deeper, than had been assumed from probings which had been stopped by boulders. In the later investigation, the bedrock was "proved" by drilling into it for 25 ft. with a diamond drill (Harding, 1949).

These are extreme examples but it may be difficult to decide on cut-off depths from borehole or drillhole evidence alone; the geological evidence should always be considered as well. At the site of the Ayer Itam dam, Penang, the bedrock was granite; overlying it was insitu weathered granite ("laterite"). Tropical weathering produces core boulders which can be up to 30 ft. diameter depending on the original joint spacing in the parent rock. At Ayer Itam, some of these boulders had been leached out of the overburden and had rolled down the valley sides to the stream bed. The largest boulder was 25 ft. diameter and had a tree growing on it. To allow for the possibility of meeting such boulders during drilling, it was decided to consider hard rock as starting at a point in the drillhole at which there was a notable improvement in core recovery. The drillhole was then continued for a further 30 ft. from that point. If it did not emerge again into soft material then it was considered as penetrating bedrock. This approach was found to be completely successful. A deep cut-off was designed and constructed without particular difficulty. It may be noted here that examples of the unsuccessful application of geophysical methods were in similar granite country on the mainland of Malaya. The depths to bedrock deduced from seismic surveys were quite unrelated to depths found subsequently from boreholes and pits.

DESIGN STAGE

At this stage, it will be necessary to gather sufficient information to enable tentative cross sections of dams to be analysed and to prove that sufficient quantities are available of each kind of fill material required.

Limits on expenditure usually require this stage of the investigation to be largely in terms of boreholes supplemented by pits. In some cases, a few insitu tests may be possible. For example, if the proposed fill material is relatively untried, it may be advisable to build a low test bank of it to establish its performance when compacted. Exceptionally, such trial banks are provided with piezometers and other instruments to monitor their performance (Little and Vail, 1961) but, more usually, the fill is placed and, after compaction, samples are taken for laboratory tests.

During the investigation stages of the Brianne dam (Carlyle, 1968), some doubts were expressed about using the cleaved mudstone, which when excavated appeared in the form of platy fragments about $\frac{1}{4}$ inch thick and 6 to 12 inches diameter. An inspection of several German dams in the Rhur valley showed that rather inferior material to this had been sometimes used for fill. As a result, it was considered that the mudstone could probably be used and a trial bank was constructed at Brianne which showed not only that the material could be successfully compacted but that, after compaction, its permeability remained extremely high. Subsequent laboratory tests in a large diameter triaxial machine showed that it had an adequate shear strength. The dam is now under construction.

The strength of stiff (overconsolidated) fissured clay foundations has always been a subject of controversy, doubts being expressed by some engineers about the effect of the fissures on strength. Investigations by Skempton and La Rochelle at Bradwell had shown that the apparent strength (c_u) of the London clay calculated from large scale slope failures was only 55% of that measured on intact lumps. The problem became urgent during investigations for a proposed dam at Maldon, Essex. Accordingly, large scale (2 ft. square) insitu shear box tests were done. These confirmed the reduction of strength previously observed at Bradwell which was considered to be due to the effect of fissures and also to the effects of anisotropy and rate of testing (Bishop and Little, 1967). Although this particular dam was not constructed, the results were important and a similar reduction factor has been used for another dam now under construction on overconsolidated clay foundations in the south of England.

Seepage beneath dams has caused many catastrophic failures. Hinderlider in 1933 gave a list of 293 failures of dams of all types, 52 of which (18%) were attributed to piping through the foundations. It is only necessary to recall the St. Francis dam and, more recently, the Baldwin Hills disasters to realise the importance of a proper appreciation of seepage potentialities.

The seepage pattern is profoundly affected by geological conditions in the foundations and these should be very carefully investigated, and considered in the design (Terzaghi, 1929). Although tests may be of some help, they will, in general, give overall values of permeability rather than local variations and all available evidence should be carefully examined for evidence of stratification, fissures and lenses of permeable or impermeable material which may affect the problem.

Rapid lateral changes should be assumed as a matter of course in alluvium and in fresh water bedrock deposits- marine deposits may show less variation.

At El Bosque dam in Mexico, seepages of more than one thousand cubic metres per second developed at the left abutment on first filling of the reservoir and eventually caused a major landslide 1 km. downstream. Seepages were observed as much as 12 km. away; the dam itself was in danger. Eventually the seepages were controlled by the injection of more than 118,000 cubic metres of clay-cement grout (Mooser, 1964). A rather less spectacular leakage (also through the left abutment) had developed at Dokan dam in Iraq. During construction, large quantities of cement were injected to form what was then the world's largest grout curtain. However, considerable leakage took place at one end of this curtain. It was not until a geological appraisal had shown that two faults were involved that remedial measures using a bentonite, diesel oil, cotton flock grout were successful (Clark, 1963).

These two examples serve to emphasize the importance of thorough geological examination and appraisal of the abutments and foundation of dam sites from the seepage point of view. Some borings or drillings should be put down to estimate likely water losses; for this purpose they will need to be water tested. The crudest way is to pour water down them but this can be regarded as no more than a qualitative test. Lugeon, Lefranc or pumping tests should be done at several different levels in conjunction with careful visual reconnaissance.

Seepage may be controlled by cut-offs or, where these are too expensive or water loss is not important, suitable downstream drainage measures can be installed. The investigation should be sufficiently broad to provide useful information whichever type of control is used.

One of the commonest mistakes at the design stage of investigation is to overestimate the amount of fill material available for building the dam. At Shek Pik, in Hong Kong, the presence of ribs and pinnacles of more or less unweathered rock covered by the weathered rock mantle greatly reduced the amount of decomposed rock available. This seems to be an inherent difficulty in areas of tropical type weathering. A further deficiency at Shek Pik was of free draining material for the upstream shell. Alluvium, planned to be used in the original design, was found to have suffered fairly intensive weathering. Although it might have been possible by constructing suitable drainage measures in the alluvium borrow area to have won the material in a condition suitable for use as fill, it was judged expedient to change the cross section design fairly drastically and substitute sand for the alluvium. Sand on the sea bed near the dam site proved to be very thin and limited in area, consequently, sand had to be brought in by sea from another under-sea site about 20 miles away (Carlyle, 1965).

Based on the early stages of investigation for the San Luis dam (California), a design incorporating 28 M cu. yds. of gravel was produced. Later investigation showed that the amount available did not exceed 8 M cu. yds. and to be on the safe side the designers cut the amount in the dam to 4 M cu. yds. The remainder of the gravel was replaced by rockfill (Bixby, 1966).

Hard rock may be required for rip rap or for crushing for drainage materials. Where rock is not available at reasonable cost, alternatives such as cement or asphaltic concrete or even soil cement may be used for slope protection. So far, artificial materials have not been used for drainage materials so that either natural or crushed gravel have to be found.

CONSTRUCTION STAGE

It is perhaps in the construction stage that the defects of previous investigations are most glaringly revealed. As deep excavations are made for cut-off and other purposes, the greater or less departure of the reality from that assumed in design is revealed to the astonished eyes of the designer. The conditions at Silent Valley have already been discussed and this is an experience which on a lesser scale must have been repeated on countless occasions. There is less and less excuse for this kind of error as techniques of investigation improve but at other times, conditions may be undetected because they are without precedent or their influence upon the design is not appreciated.

The foundations of the William Girling dam at Chingford, North London, contained a layer of weak alluvial soil. This stratum was not removed because its effect on the stability of the embankment was not appreciated. The designers probably took the view that the layer would consolidate rapidly as it was so thin. A dam of very similar design had been successfully completed many years before on an immediately adjacent site. Unfortunately, the William Girling dam was built with plant modern at that time and which placed the fill at a much higher rate than that at which the previous dam had been constructed. When construction had reached about two-thirds of its design height, the new dam failed. Subsequently investigation and analysis showed that the failure had taken place on a surface which, because of the very strong influence of the soft layer, departed radically from the hitherto almost universally assumed circular arc trace. An analysis made after the failure confirmed that with the design and construction methods used, failure was inevitable (Cooling and Golder, 1942). This was probably the first occasion on which soil mechanics technique had been applied to a non-circular trace, although Collin a century previously had proposed cycloidal traces to represent rupture surfaces (Collin, 1846).

Following the intensive investigations for Mangla dam, the design took account, where appropriate, of the very strong stratification in the foundations. After construction started, shear zones were discovered in the bedrock. These shear zones were in the clay strata parallel to the bedding, sandwiched between layers of sandstone and were apparently caused by bedding plane slip during tectonic disturbances of the beds long after their original deposition. Along shear zones the strength of the clay had been greatly reduced and approximated to the residual condition. This discovery necessitated a radical redesign of all the earthworks on the project and certain other measures (e.g. steel lining to the tunnels and additional drainage) as well. (Binnie, Clark and Skempton, 1967; Binnie et al, 1967). The shear zones were not detected in any of the boreholes or pits in the preconstruction investigations. They were first found in exploratory adits on the Jari section of the work but a most intensive search by site personnel familiar with their appearance was needed before they were found on other parts of the project. The Sukian section was particularly difficult and shear zones here were not observed until the core trench excavation was well under way, even though considerable lengths of exploratory trench had been excavated and a search made expressly for shear zones.

The geology at Mangla was deceptively simple. Although there was a monoclinal fold some way downstream from the dam site, and steeply dipping beds at Jari, at Mangla itself the beds had a uniform gentle dip of about 10°. It was not until the geology was investigated on a regional scale that the effect of the tectonic forces which had produced the shear zones became evident.

It was fortunate that the discovery of the shear zones was contemporary with Professor Skempton's investigations into residual strength and its invluence on the stability of slopes (Skempton, 1964) otherwise their engineering significance might have taken a long time to establish. It certainly would not have been appreciated during the early investigations at a time when, for example, Canadian engineers were still puzzled by the discrepancy between laboratory values of parameters and those deduced from full scale failures at South Saskatchewan (Gardiner) dam. In order to obtain the right answers from research or investigations it is necessary to ask the right questions; questions cannot be asked about phenomena whose existence is unsuspected.

Since the discovery at Mangla, it has become apparent that shear zones have been responsible for difficulties at other dam sites, notably Roseires, Sudan; South Saskatchewan, Canada and Waco, U.S.A. (Little, 1968). The failure of Wheeler Lock, U.S.A., was also due to an "unsuspected thin clay seam" 1/16 to 3/8 inch thick near the bottom of a band of shale; it is apparent from this description that this was also a shear zone which "went undetected during extensive bedrock investigations in 1960 ... and for three months after the failure" (E.N-R., 1962).

This evidence shows that shear zones of this type are virtually impossible to detect in conventional site investigation boreholes. It is impracticable to examine every inch of core so closely as to be able to say confidently that there are no shear zones and, in any case, the driller all too often fails to recover any core at all in the vital section. As has been justly observed: the two percent of core which is not recovered is frequently more important than the ninety-eight percent which is. The only certain way of finding shear zones is in the construction excavations; even careful examination of trial pits frequently fails to disclose them. On a recent earth dam job in U.K., the conditions were right for tectonic shear zones but although their presence was suspected, it was not possible to find them until the contractor had made sufficient excavations round the site. The difficulty of finding them is not so surprising when it is remembered that, as in the case of Wheeler Lock, the offending layer may be only a few millimetres thick and this takes some finding on the average civil engineering site.

It is clear that existing techniques need development to make possible the detection of such conditions before construction begins.

CONCLUSIONS

Geotechnical investigations for embankment dams are ideally divided into three parts.

Part I, Preliminary, establishes the feasibility of the site from a geological and soil mechanics viewpoint. It investigates geology, strength, compressibility and watertightness of foundations and abutments and availability of fill materials. Experience has shown that geophysical investigations alone are not sufficiently reliable to be used in this stage.

Part II, Design stage, produces data for design on stability, seepage and quantity and quality of construction materials.

Part III, Construction stage. The designers' assumptions are tested at least in part and, depending on their correctness or otherwise, a reassessment of the design is required. It is not, however, until the last load of fill is placed and compacted that the process of design can be said to be completed; sometimes not even then.

At all stages, geological investigations should play their full part but the engineer/designer should remain in control of the overall programme.

ACKNOWLEDGEMENT

The author is grateful to the Partners of Messrs. Binnie & Partners for permission to use some information published in this paper.

REFERENCES

Binnie, Clark & Skempton, 1967. The effect of discontinuities in clay bedrock on the design of dams in the Mangla Project. Ninth ICOLD, Istanbul. Vol.I, p.165.

Binnie et al, 1967. Engineering of Mangla. Proc. Inst. Civ. Engrs. London, Vol.38, p.343 (November). Discussion, Vol.41, p.119 (September, 1968).

Bishop & Little, 1967. The influence of the Size and Orientation of the Sample on the Apparent Strength of the London Clay at Malden, Essex. Proc. Geotechnical Conf., Oslo, Vol.I, p.89.

Bixby, T.P., 1966. Influence of Modern Construction Equipment on San Luis dam. Am. Soc.C.E. Water Resources Engineering Conference, Denver, Colo.

Carlyle, W.J., 1965. Shek Pik Dam. Proc.Inst. Civ. Engrs., London, Vol.30, p.557 (March). Discussion, Vol.35, p.145 (September, 1966).

Carlyle, W.J., 1968. The River Towy scheme for the West Glamorgan Water Board. Water and Water Engineering, Vol.72, p.371 (September).

Clark, J.F.F., 1963. Discussion on Session 2, Symposium on Grouts and Drilling Muds in Engineering practice, p.112. Butterworths, London.

Collin, A., 1846. Recherches expérimentales sur les glissements spontanés des terrains argileux. Carilian-Goeury et Dalmont, Paris. Translated by W.R.Schriever. University of Toronto Press, 1956.

Cooling & Golder, 1942. The analysis of the failure of an earth dam during construction. Journal Inst. Civ. Engrs., London, Vol.19, p.38 (November).

E.N-R., 1962. Clay Seam Wrecked Wheeler Lock. Engineering News-Record, 4 January, 1962, p.19.

Harding, H.J.B., 1949. Site Investigations including Boring and other Methods of Sub-surface Exploration. Journal Inst. Civ. Engrs., London, Vol.32, 1948-49, p.111.

Little & Vail, 1961. Some developments in the measurement of pore pressure. Conference on Pore Pressure and Suction in Soils, p.75. Butterworths, London.

Little, A.L., 1968. Discussion on Waco Dam Slide by R.R.W.Beane. Proc. Am. Soc. C.E. S.M. & F. Division. March, 1968.

McIldowie, G., 1935. The Construction of the Silent Valley Reservoir, Belfast Water-Supply. Min. Proc. Inst. Civ. Engrs., London, Vol.239, 1934-5, Pt.I, p.465.

Mooser, F., 1964. A case of exceptional permeability at El Bosque Dam, Mexico. Eighth ICOLD, Edinburgh. Vol.I, p.891.

Skempton, A.W. & LaRochelle, 1965. The Bradwell Slip: a short term failure in London Clay. Geotechnique, Vol.15, p.221.

Skempton, A.W., 1964. Fourth Rankine Lecture. The long-term stability of clay slopes. Geotechnique, Vol.XIV, p.75 (June).

Terzaghi, K., 1929. Effect of Minor Geologic details on the safety of dams. Am. Inst. Min. & Met. Engrs. Tech. Publn. No.215.