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CONSTRUCTION OF MUIRHEAD RESERVOIR, SCOTLAND

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INTRODUCTION.

Muirhead Reservoir on the Rye Water, Ayrshire, South-west Scotland, supplies water for domestic industrial purposes to the Burgh of Paisley (population 90,000). The reservoir was formed by an earth dam, 2,070 feet long, constructed immediately upstream from an existing reservoir. During construction, movement of the earthwork occurred and, as a result, the design was modified and the height of the dam limited to about 70 feet instead of 87 feet as originally planned.

The reservoir catchment area is 3,230 acres with average annual rainfall, 65 inches. The modification reduced the storage capacity from 1,250,000,000 gallons as intended to 780,000,000 gallons and the dependable yield from 5½ to 4½ million gallons per day. This was warranted because the drop in yield was offset by works established during construction of the reservoir and by alternative means available for additional storage in any future development.

Details of the design and construction of the dam are given herein. The nature of the movement of the earth embankment is described and particulars are given of the soil mechanics investigation which followed and of the measures finally adopted to ensure stability.

GEOLOGICAL CHARACTER OF THE SITE (DRAWING 3).

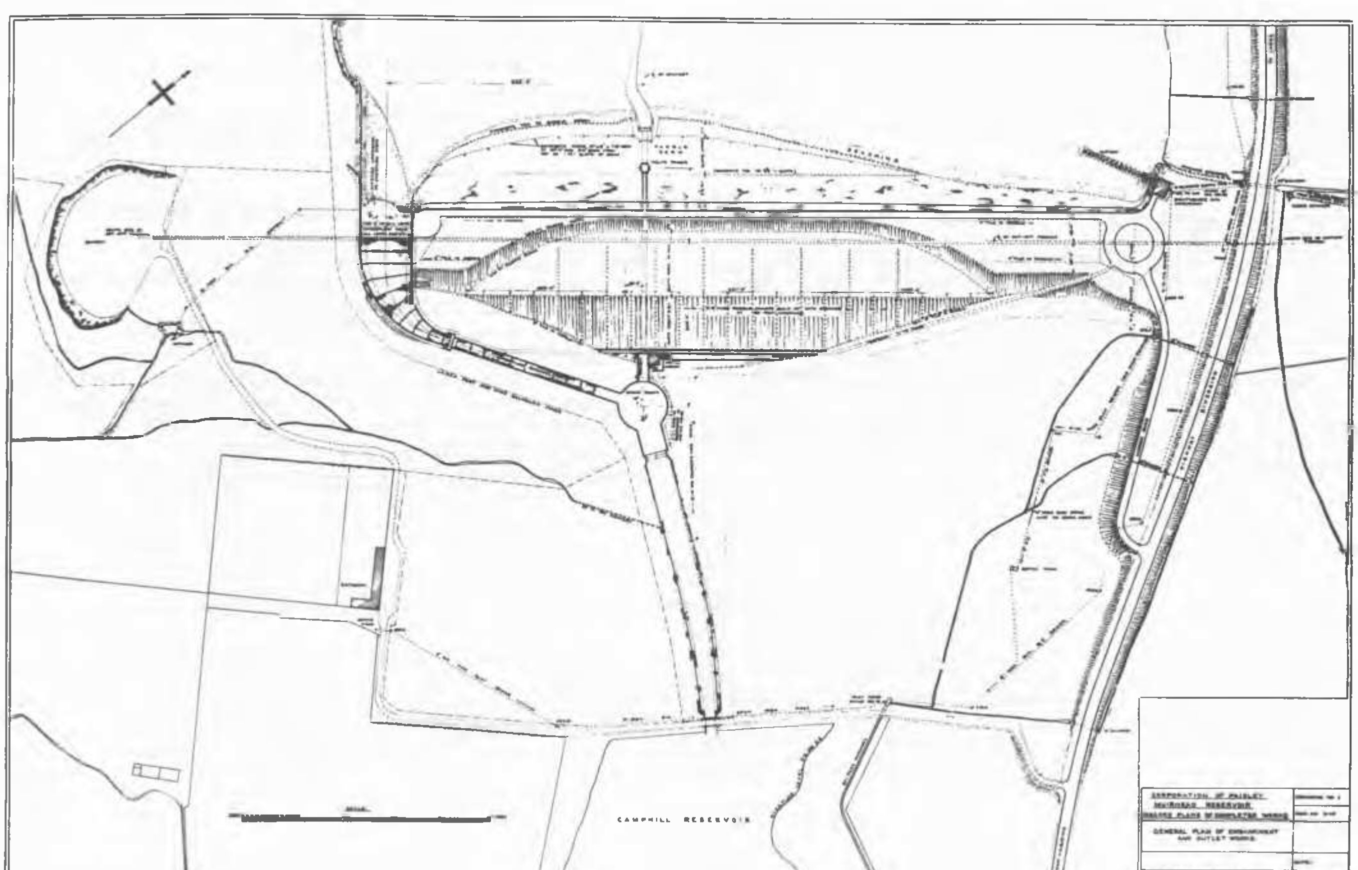
The rock formation is of volcanic origin. Whinstone (a basalt) overlies deep deposits of volcanic ash, into which are intrusions of whinstone, fractured and fissured to a varying degree. Overlying the rock is boulder clay on which the dam was founded. The embankment is of boulder clay and the same material, selected from the more clayey deposits and the larger stones removed, was soured for the puddled clay core.

DESCRIPTION OF WORKS AND METHOD OF CONSTRUCTION.

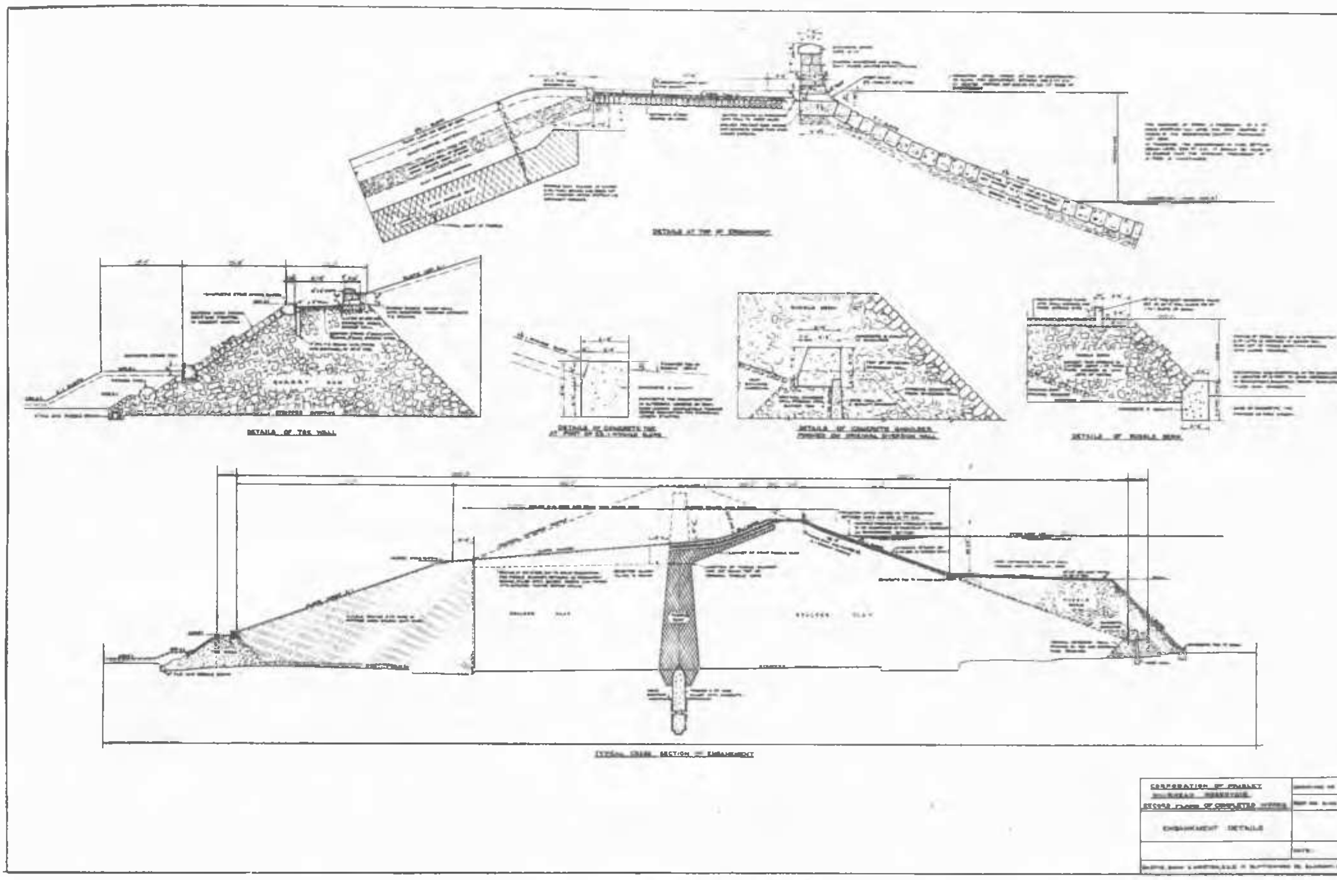
General.

Details of the works as constructed are shown on Drawings 1, 2, and 3 , The plant Schedule (Fig. 1) and the Layout Plan (Fig. 2) explain construction arrangements and the Chart (Fig. 3) gives the programme of operations for the principal items of work.

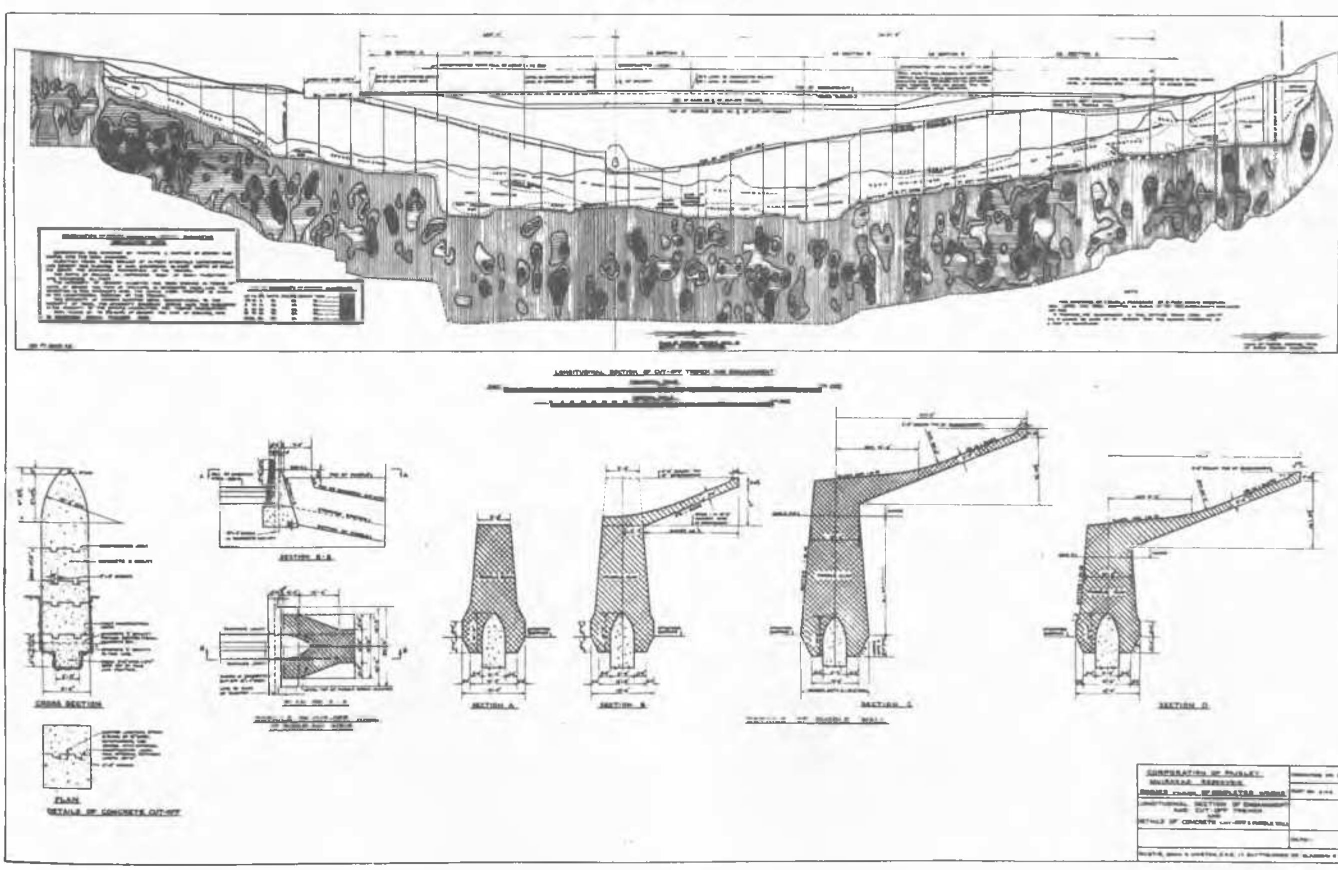
The original design of the dam was on orthodox lines. The outline in so far as it did not conform with the ultimate profile was as shown by dotted lines and omitting the upstream berm in the typical cross section in Drawing 2. Stone rubble walls formed the toes of the slopes. The upstream face was pitched with stone and the downstream slope sown with grass. Drainage was provided in the outer half of the downstream side by a 2-feet layer of quarry



DRAWING 1



DRAWING 2



DRAWING 3

ITEM	USE	NOTES
	CUT-OFF	
7 - 3 ton Derrick Cranes 85 feet jibs.	Handling excavation and concrete.	Located along trench 160 feet centres.
3 - 3 ton Loco Cranes.	do.	One crane used at Culvert.
4 - 2 Tool Compressors 120 cub.ft. per min. each.	Rock drilling.	
2 - 2 inch injection pumping units.	Cementation.	
	EMBANKMENT	
10 Excavators - total capacity 7½ cub. yds.	Borrow Pits.	6 of 4½ cub.yds. capacity prior to April 1941.
75 - 5 ton end-tipping Lorries.	Transport from Borrow Pits to Embankment.	21 lorries prior to April 1941 75 do. April-Oct. 1941. 40% of fleet under repair due to heavy duty.
2 feet gauge rail track 20 Diesel Locos(20-40 H.P.) 170 Cub.Yd. Waggons.	do.	In use April-Aug. 1941.
3 Euclid Waggons , ecch 13 cub.yd. capacity.	do.	Aug.-Sept. 1941.
2 Athey Waggons each 12 cub.yd. capacity.	do.	Sept. 1941.
2 Winding Engines.	Muck haulage to West end of bank.	One used later in removal of excavations from top of bank, one lowering rubble to berm.
2 Dragline Excavators each 5/8 cub. yd. capacity.	Dressing slopes.	do. do.
4 Bulldozers - 1/D8, 2/D7 and 1/D6.	Spreading and compaction of earthwork.	April-Sept. 1941. 2/D8 prior to April 1941.
	QUARRY	
3 Crushers with conveyor belts, riddle screens and hopper. 1 - 16"x12", 14 H.P.Unit. 1 - 12"x 6", 12 H.P. " 1 - 10"x 5", 10 H.P. "	Crushing for concrete aggregate, stone under pitching, etc.	Quarry located beyond West end of dam.
1 - 6 Tool air compressor 250 cub.ft. per min.	Rock drilling.	
1 - excavator 1½ cub. yds.	Loading rubble for Berm.	
	CONCRETE WORK	
10 Concrete Mixers 10/7 to 14/10 capacity 2 Mortar Mixers.	General	Portable units sited as required.
	LIGHTING	
1 - 34 K.W.250 v.generator Power Unit 55 H.P.	External lighting.	
1 - 7 K.W.100 v.generator Power Unit 14 H.P. 54 Cell Storage Battery.	Works Camp and Offices.	

MUIRHEAD RESERVOIR.PLANT SCHEDULE

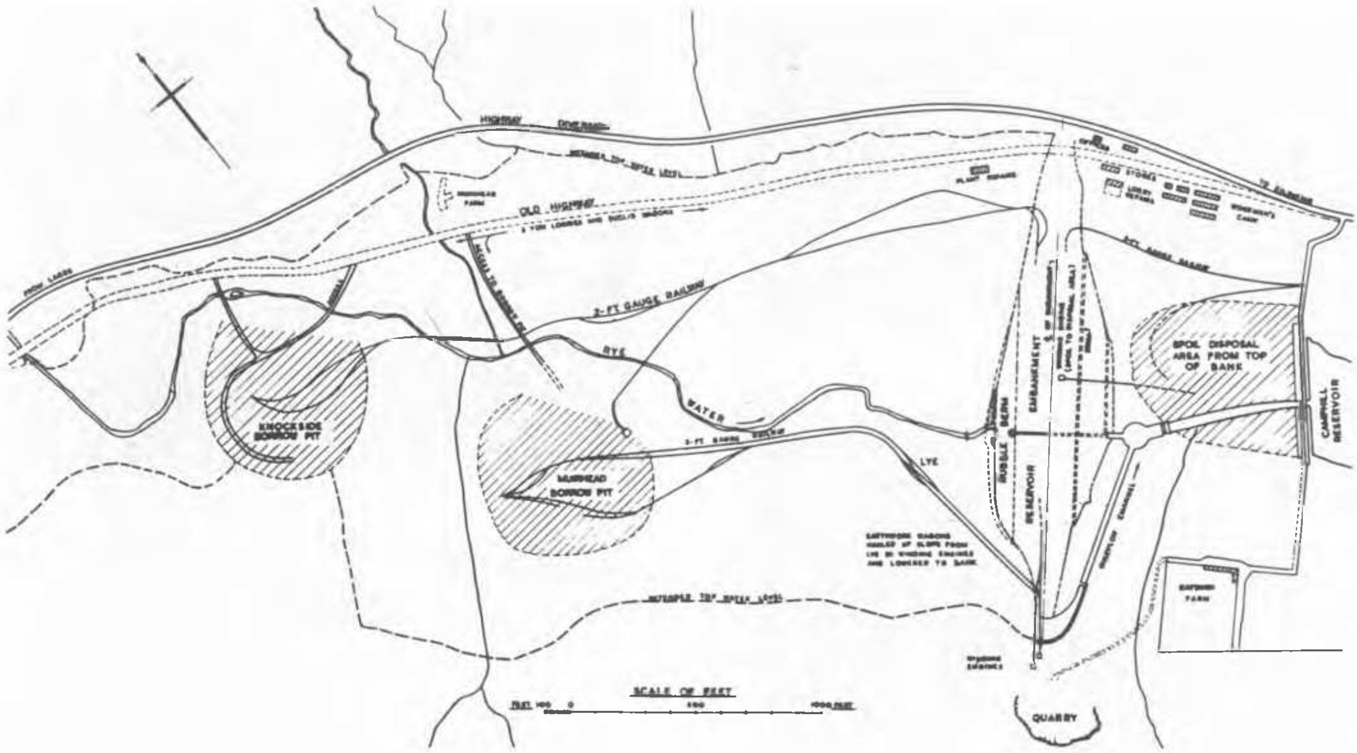
FIG.1

debris at the base and vertical drystone walls spaced 50 feet apart.

A concrete culvert, founded on rock and connecting with a forebay and tailbay built into the toes of the embankment, served to divert the Rye Water clear of the works. The draw-off from the reservoir is controlled from a valve tower which, at its base, is monolithic with the culvert. The overflow channel is on natural ground and the weir, at the South end of the dam, was built on top of the cut-off wall.

Cut-Off Wall and Cementation.

Exploratory bores to determine the required depth of cut-off across the valley gave no conclusive results because of the confused nature of the volcanic formation, the intrusions of whinstone following no definite order. The cut-off trench, 6 feet wide, was excavated to a firm strata throughout its length of 2280 feet. The average depth of trench was 60 feet below ground and 24 feet below rock surface level. Below the trench, the cut-off was completed by cementation. The volcanic formation was estimated to be 700 feet or more deep and it was decided provisionally that the depth of cut-off below rock surface should be approximately equal to the height from the latter to



Muirhead reservoir.Lay-out of construction plant

FIG.2

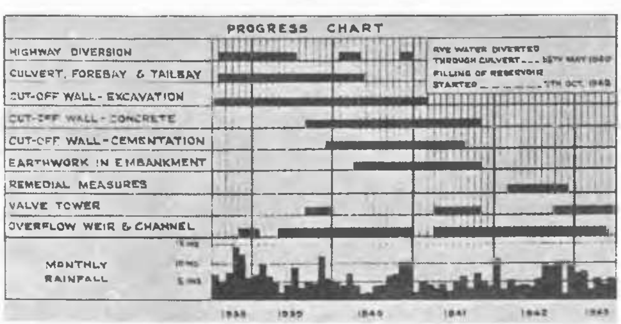


FIG.3

the top water level of the reservoir, provided that, before final grouting, the leakage in a 10 foot length of borehole in untreated rock did not exceed 1/10th cubic foot per minute.As finally completed, the depth of cementation below the bottom of the trench was 100 feet in the central portion and 60 feet in the wings.

The procedure followed was to drill primary holes spaced 10 feet apart and secondary holes intermediate with these where found necessary, so that locally the borehole spacing averages 5 feet centres. The grouting holes (all 2-inch diameter) were drilled to a depth of 10 feet and, after grouting under pressure, were re-drilled through the set mortar in the borehole and extended for a further 10 feet, this procedure being repeated until cementation to the full decided depth was accomplished.

The ratio of cement/water by weight (C/W) varied from 0.7 to 0.07 and injection pressure from 100 to a top limit of 600 lbs. per square inch. Drilling totalled 27,000 feet and 3,600 tons of cement were injected. To test for leakage the borehole were connected by pipe line to a small water tank on the hillside 60 feet above the reservoir. An external sight glass on the tank indicated any drop in water level. The leakage did not in any case exceed the predetermined limit and in many boreholes complete water-tightness was obtained.

Details of the cut-off and a longitudinal section indicating the intensity of cement injected are shown on Drawing 3 . The trench was filled with mass concrete to ground level; pourings were limited to 4 feet deep and 60 feet long. In the bottom 18 feet depth of concrete, vertical pipes were embedded, 20 feet apart, through which holes were drilled and grout injected into the rock to seal the trench bottom.

<u>Earthwork</u>.

The specified procedure in forming the embankment was that the material should be deposited, spread and compacted in layers not more than 12 inches deep, the layers to be inclined from the outer slopes towards the puddled core.

The rate of forming the embankment is shown in Figures 4 and 5. Operations commenced in April 1940 and the average rate of placing was about 3,300 cubic yards per week until April 1941, at which date approximately 170,000 cubic yards had been placed, or about 30 per cent. of the total anticipated quantity of 580,000 cubic yards. Wartime industrial needs made it essential to accelerate progress, and to achieve this the existing contract was terminated by agreement and a new contract was let with another firm of contractors who had more adequate resources in both labour and plant. Thereafter an average rate of about 16,000 cubic yards per week was maintained for the ensuing 5 months until mid-September 1941 when a total of approximately 500,000 cubic yards had been deposited, the maximum quantity deposited in one week reaching the peak figure of

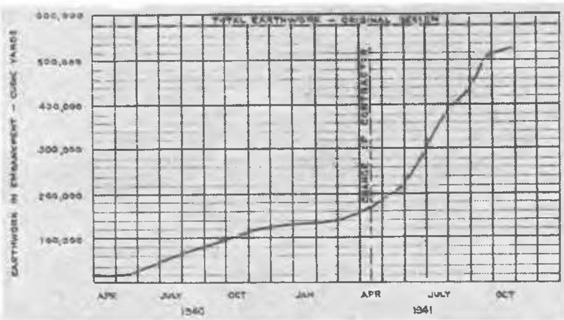


FIG.4

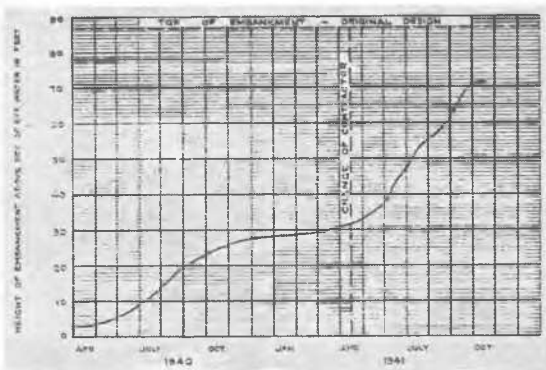
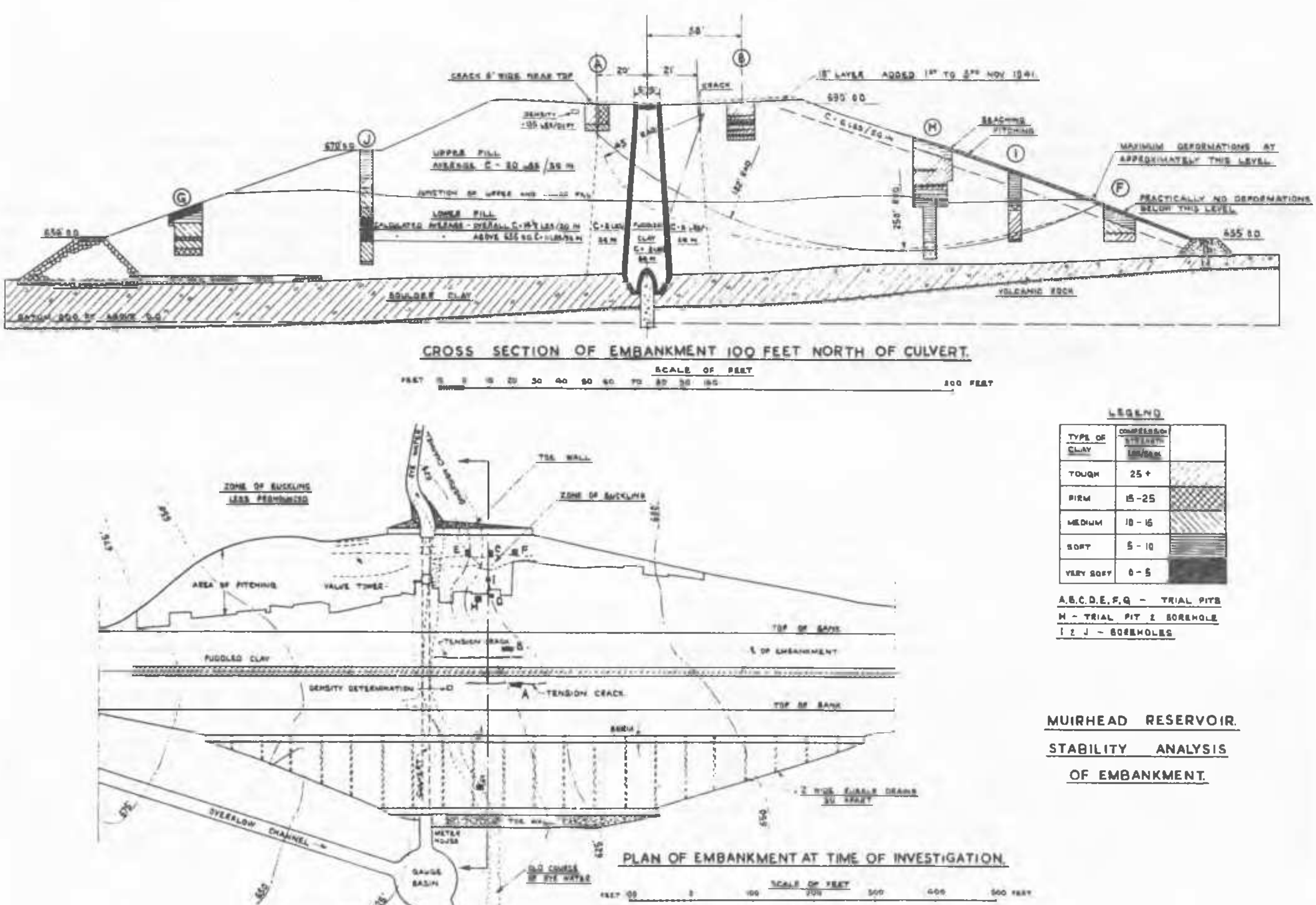


FIG.5

Muirhead reservoir. Arched deformation of stone pitching

FIG.6



DRAWING 4

25,000 cubic yards. At this stage the top of the bank had reached an elevation of about 690 O.D. or 18 feet below the intended top finished level of the dam.

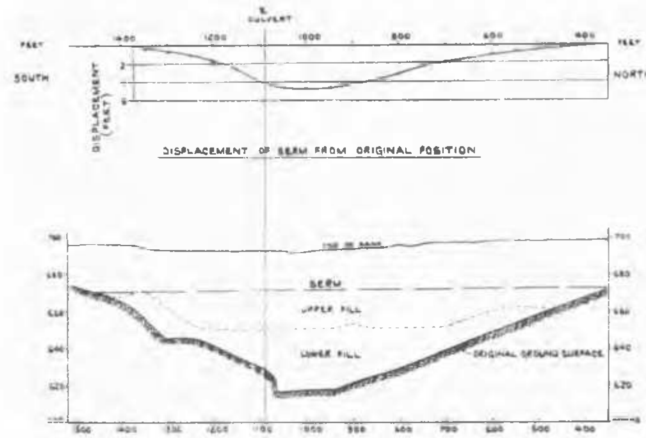
Superficially, the embankment had every appearance of a substantial, thoroughly compacted job, but on 16th September a limited displacement of the stone pitching was observed on the lower part of the upstream face and three days later the pitching at one or two places had risen in the form of an arch (Figure 6). The placing of material on the central portion of the embankment was suspended pending a survey to determine the nature and degree of deformations. On 22nd September two cracks were observed on top of the bank, one on each side of the puddled core parallel to and about 20 feet distant from the centre line of the dam (Drawing 4).

INVESTIGATION OF MOVEMENT.

A survey across three sections of the embankment revealed that displacement of the earthwork had occurred practically in a horizontal direction, the extent varying almost directly with the height of the embankment (Figure 8). The toe walls remained stationary.

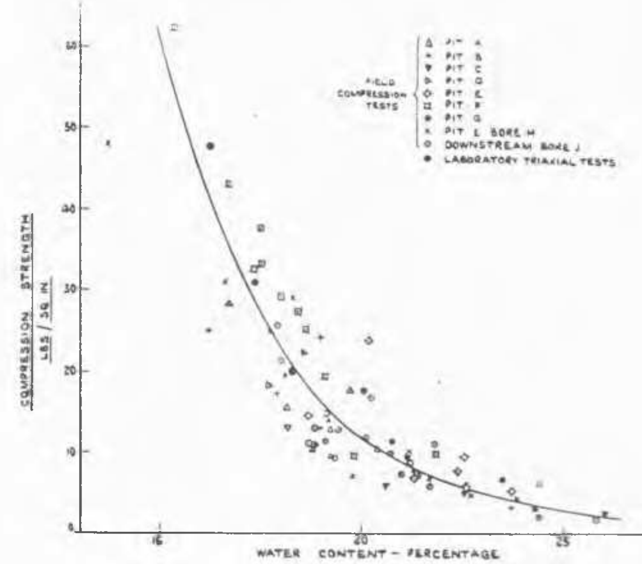
Investigation was concentrated on the cross section 100 feet north of the culvert where the height of the dam was greatest and deformation most pronounced. Shallow and deep observation points were established and bores and trial pits sunk to obtain samples for testing (Drawing 4) Movement of the deep observation points proved to be substantially the same as that of the adjacent surface point. The average movements of points E and F (Figure 7) are typical of the others.

Initial observations established that the berm on the down-stream face had moved outwards about 2 feet and the pitched slope of the upstream face above the level of the arching nearly 4 feet. Measurements, repeated after an interval of one week, showed that the displacement of the berm had increased to 4 feet and of the pitched slope to 4 feet 6 inches. About one month after suspension of operations on the central portion of the embankment, movement had ceased and during 1st to 3rd November additional material giving a compacted thickness of 1 foot 6 inches was added cautiously. As a result the embankment resumed movement, again practically in a horizontal direction,



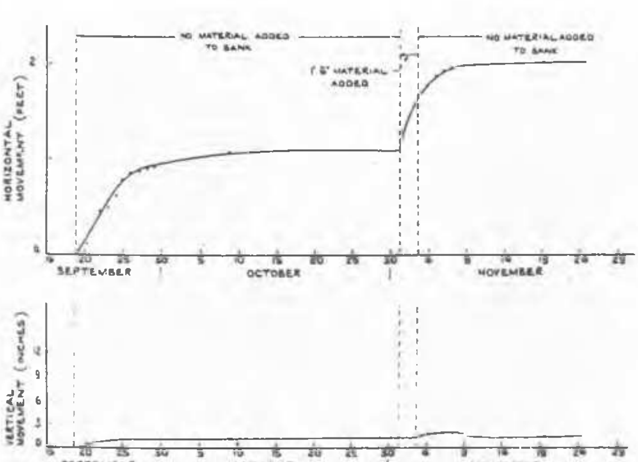
Longitudinal section along a line 110 feet down stream from centre line (IE vertically through berm)

FIG.8



Relation between compression strength and water content

FIG.9



Average movements of observation points E and F

FIG.7

but a slight fall in level of the points at the top of the bank and a slight rise in level at points 40 feet up the slope from the toe walls suggested that a deep seated slide of the rotational type was in progress. It was apparent that the bank had no reserve of safety and that remedial measures would have to be undertaken.

FIELD AND LABORATORY TESTS.

The properties of the embankment material were examined, particularly with reference to density and shear strength. About 225 compression tests were made on the site, using a portable compression apparatus. 1) The water content was determined and triaxial and equilibrium shear tests made in the laboratory.

Natural boulder clay in borrow pits weighed 140 lbs. per cubic foot, had a water content of 16.5 per cent. and compression strength of 40 lbs. per square inch. The liquid limit was

42.6 average and plastic limit 17.8 average. The Proctor maximum dry density was 116 lbs. per cubic foot. The mechanical analysis of the material is given in Figure 10.

The drop in strength with increasing water content was very marked. At 18.5 per cent. the strength was 20 lbs. per square inch and at 20.5 and 22.5 the strengths were 10 lbs. and 5 lbs. per square inch respectively. The relation between compression strength and water content is shown in Figure 9.

Triaxial compression tests showed no increase in strength with increasing normal pressure, indicating that ϕ was equal to 0, so that the immediate shear strength was equal to half the compression strength. Equilibrium shear tests showed a definite increase in strength with increasing load ($\phi = 30^{\circ}$ approx.) and a substantial drop in strength under low normal pressures.

ANALYSIS OF FAILURE.

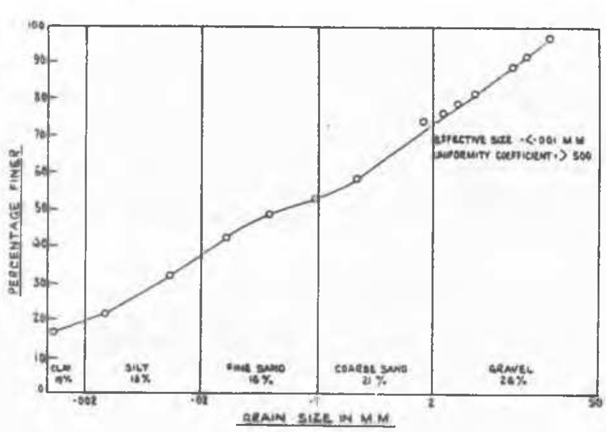
The strength of the material in various zones of the embankment, as deduced from the tests, is shown in Drawing 4. The compression strength of the core was 3 lbs. per square inch which was governed by the water content necessary to produce well-puddled clay. The strength of the embankment was low near the surface of the slopes and adjacent to the puddled core, which is explained by the difficulty of getting good compaction in these zones. The strenghh in the upper fill (i.e. material placed by the second contractor) was reasonably consistent, but the strength in the lower fill was variable and the value adopted for the latter in designing the modifications was arrived at from the assumption that the embankment had a factor of safety of unity. The foundation clay and also the clay at the base of the embankment was firm and the surface level of the latter defined the probable lower limit of the curve of shear.

The conditions of stability were examined on the basis of (a) a single-circle slip, (b) a two-circle slip, and (c) pressure from the puddled core for both upstream and downstream sides of the embankment on the lines indicated in Drawing IV. There was little evidence to prove that puddle pressure actually contributed to movement but the possibility of this was taken into account in the final design.

FINAL DESIGN.

It was decided that the embankment should have a minimum factor of safety of 1.25 against each method of failure analysed. The height of the embankment was restricted to the level established at the second stage of movement. The upstream slope was stabilised by constructing a rubble berm at the toe. The same means could not be adopted on the downstream side because costly modifications to outlet works already completed were involved and stability was procured by excavating material from the top of the embankment. The different methods adopted to stabilise the upstream and downstream sides of the dam led to an asymmetrical cross section and occasioned the unorthodox construction of the top section of the puddle wall. The completed dam is shown in photographs, Figures 11 and 12.

In removing material from the embankment, a winding engine and light gauge track with cube yard waggons were employed, and the use of heavy plant was avoided until such quantity of material had been removed as would ensure safety. Every care was taken to ensure that water could not lodge on the surface as excavation proceeded. Filling of the reservoir was



Mechanical analysis of banking material

FIG.10

Muirhead reservoir. Upstream face of dam.

FIG.11

Muirhead reservoir

FIG.12

so controlled that the rise in water level did not exceed 6 inches per day, and a 36-inch scour valve, which had been added during construction to increase facilities for discharge in case it should prove necessary to commence filling the reservoir before the dam had been completed to its full height, was disconnected when the reservoir had filled to overflow level to forestall too rapid lowering of the reservoir at any future time.

An important question arose as to whether or not the greatly accelerated rate of forming the embankment had a bearing on the movement that developed. Hitherto, it has been customary to form embankments at a much slower rate than is now the practice with muck-shifting vehicles of large capacity. By former methods considerable consolidation took place in the prolonged period of construction, whereas with modern methods the earth work is possibly in a less settled state as successive layers are added. Tests proved the upper Muirhead fill to be better compacted and of more uniform compressive strength than in the lower fill deposited at a slower rate, and, while the evidence is not conclusive, it suggests that the immediate stability procured by modern methods need be no less favourable than has pertained in the past.

ACKNOWLEDGEMENTS.

The Author desires to record his appreciation of the valuable assistance rendered by Engineers of H.M. Ministry of Works, by Messrs. Binnie, Deacon & Gourley, C.C.E., London, who were called into consultation when movement of the embankment was first observed, and to the Building Research Station of the Department of Scientific and Industrial Research who collaborated with the Engineers in the study of the site conditions. Site and laboratory tests and the relative figures and diagrams included in this paper are extracted from or are based on a report submitted by the Building Research Station to whom due acknowledgement is made. Messrs. Casey & Darragh, Ltd., Stirling, carried operations to the first stage and the work was completed by Messrs. George Wimpey & Co. Ltd., London.

REFERENCE.

1) "A Portable Apparatus for Compression Tests on Clay Soils" by L.F. Cooling and H.Q.Golder, Engineering 1940, Jan. 19th.

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SUB-SECTION IV c

EXCAVATIONS AND SLOPES

IV c 1 DESCRIPTION OF A FLOW SLIDE IN LOOSE SAND

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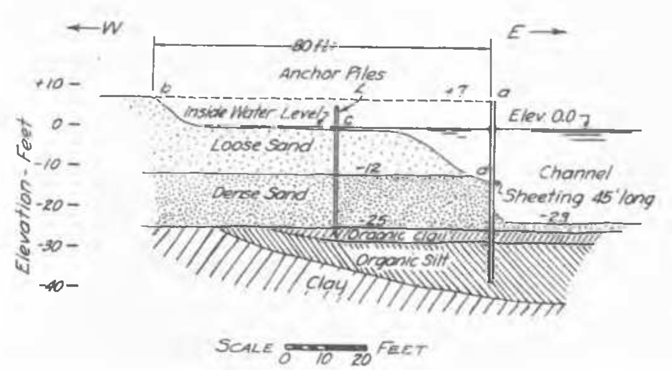
SUMMARY

This paper describes a flow slide that occurred in fine loose sand during reconstruction of a dock wall in East Chicago, Indiana in 1946. The flow occurred through a narrow opening made by the removal of a small portion of a sheet pile wall. Since no seepage pressures were involved, the primary cause of the occurrence appears to be the collapse of the extremely loose structure of the fine sand.

DESCRIPTION OF SLIDE.

Conditions prior to the slide are shown in Figure 1 which represents a cross-section through the dock wall. The outer face of the dock consisted of sheet piles 45 feet long. To the east of the sheet-pile wall was a canal with its water level at El. 0.0. The channel was maintained by dredging to El. -23. On the west side of the sheet piling the surface of the ground had a constant level at El. +7. A row of anchor piles 30 feet long was driven vertically at a distance of 40 feet from the sheet piling and was tied to it by means of steel rods at water level.

The material between El. +7 and El. -12 consisted principally of loose sand deposited by means of a hydraulic dredge. Most of this fill was pumped out of the channel during the construction of the dock. Between El. -12 and El. -25 was a natural deposit of beach sand



Conditions before slide

FIG. 1