rents amounting to 31.5 MVAr per pole (i.e. per half station) initially rising to 42.5 MVAr per pole if additional filters for higher frequency harmonics are required later.

To allow the filters to be disconnected without interference to the flow of power to or from the South Island system required a "load breaking isolator" capable of interrupting 42.5 MVA of capacitive current, of withstanding a through short circuit of 52,000 A, and of carrying continuously a current made up of: 50 c/s, 809 A; 3rd harmonic, 100 A; 5th harmonic, 490 A; 7th harmonic, 338 A; 11th harmonic, 194 A; 13th harmonic, 155 A; 17th harmonic, 100 A; 19th harmonic, 80 A; 23rd harmonic, 50 A; and 25th harmonic, 40 A.

The equipment finally purchased was an air-blast circuit-breaker manufactured in Japan under licence from a French firm, an adaption of a 70 kV 2,500 MVA 2,000 A circuit-breaker.

The connections from the transformer 33 kV windings to these circuit-breakers are made with oil-filled cables two 0.4 in^2 cables in parallel per phase. These were manufactured in England.

7. 220 kV OUTDOOR STATION

The layout of the 220 kV outdoor station is shown in Fig. 5 from which it can be seen that there is a ring bus on one side of which are connected the two circuits to the interconnecting transformers at the power house and on the other side three outgoing transmission lines. A spare bay could accommodate a fourth line, and there is space at either end into which the ring could be extended if still more lines were ever required.

A reinforced concrete building provides workshop and toilet facilities and accommodates circuit-breaker compressed-air equipment, 3.3 kV and 400 V switchgear for local power supplies, and a 110 V storage battery and charger to operate the closing and tripping coils of the air-blast circuit-breakers.

7.1. 220 kV Ring Busbars

The 3 in diameter copper conductors of the ring bus are supported 17 ft above the ground by insulator stacks mounted on concrete posts, and the connections from the bus to the switchgear for transformers or transmission lines are rigid copper tubes (Fig. 11).

This arrangement achieves a saving in cost, as no overhead tensioned conductors or supporting steel structures are required. Steel take-off gantries were used, however, where transmission line towers could not be placed sufficiently close.

This ring bus arrangement has, also, more flexibility for the maintenance of isolators than "the single bus with a ring connecting its ends" that was adopted at Roxburgh and Haywards (4 and 5) but it occupies some 10% more space. If there had been any great necessity to save space a reduction of about 20% could have been made by omitting the isolator on the bus side of each circuit breaker and using the two adjacent bus section isolators for isolating the circuit-breaker.

Tests have indicated that this general arrangement achieves a low level of radio interference.

7.2. 220 kV Switchgear

The circuit-breakers are of an air-blast type with sequential switch and have an interrupting capacity of 7,500 kVA. The breaker for the Islington line is suitable for high-speed single-pole reclosing, and the breaker on the line that initially connects with Livingstone is used for three-pole reclosing. A further line breaker to be installed for the line from Roxburgh will be suitable for single-pole reclosing.

The isolating switches and bus section switches are all rated at 1,200 A.

8. AUXILIARY SERVICES

Briefly: Power and lighting supplies at 400/230 V are provided as close as possible to the loads through 15 transformers which are supplied at 3.3 kV from one of the two 2,500 kVA auxiliary generators, each driven by its own 3,500 hp turbine with completely separate tailrace, penstock, and intake. Each of these generating sets has two further 3.3 kV, 400 kVA generators to supply the essential needs of the mercury arc converters. An emergency supply can be obtained from the Waitaki Power Board.

Cooling water is pumped from the tailrace by one of two 12,000 gal/min 400 hp pumps which are speed-controlled by rotor resistor taps.

9. CONCLUSION

Brief mention must be made of the magnitude of the task faced by the N.Z. Electricity Department's construction forces to install the first half of the station equipment in time to allow five

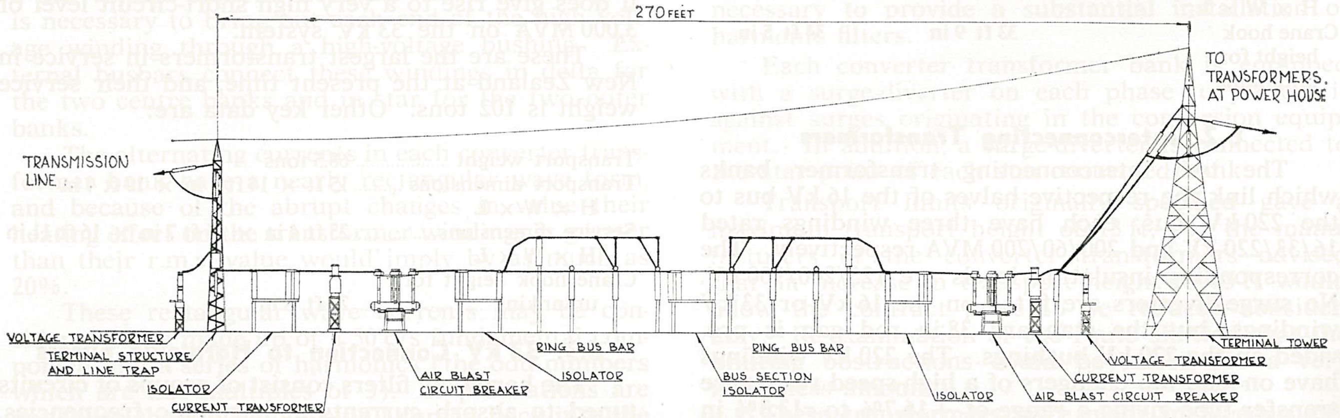

Fig. 11: Cross-section of 220 kV outdoor switching station.

Design of Benmore Earth Dam

O. T. JONES*
B.E., (MEMBER)

The paper describes aspects of design for the earth dam at Benmore. The dam, of rolled fill with central core and shoulders of river gravels and rockfill, is 360 ft high. General considerations of site conditions and design requirements leading to the form of the dam are discussed. In describing methods of testing material for design data and the stability analysis carried out the author outlines a method of "semi-total-stress" analysis to allow for pore-pressure changes during earthquake loading. Modifications in design arising from construction experience are outlined, installations for dam performance measurements are described, and their results to date are given.

1. INTRODUCTION

THE siting of the 540 MW Benmore hydro-electric station in the gorge of the Waitaki River upstream of Otematata was determined after site investigations of two reaches of the river: (a) Blackjack's Point, or the "Upper Benmore" site upstream of the Ahuriri junction, and (b) the "Lower Benmore" site downstream of the Ahuriri.

The upper site was well suited for a concrete dam, whereas the lower site suited an earth dam. The progressive reduction in earthmoving costs during the period of the investigations was one of the main factors in the selection of the lower site for development.

The layout of structures adopted for the site is shown in Fig. 1. The valley is dammed by the earth dam, with a 2,700 ft crest length, and the 400 ft long concrete gravity dam which incorporates the penstock intakes.

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2. SITE CONDITIONS

At the damsite river gravels up to 80 ft deep covered the basement rock in the valley bottom, while the lower sides of the valley were covered with as much as 30 ft of talus. On the upper slopes rock outcropped with localised pockets and layers of talus. Details of the geology have been described by Oborn and McKellar (1) and the engineering foundation investigation by Ballintine (2).

In brief, the basement rock comprised interbedded greywacke and argillite. The general attitude of the bedding planes was near-vertical (frequently overturned), with the strike variable but somewhat parallel to the river. Although no recent faulting was evident the rock had been subjected to considerable tectonic deformation producing localised folding, crush zones, and minor faulting, particularly in the weaker argillite. Although highly jointed (the argillite intensely so) the rock seldom showed signs of open joints.

Generating and Electrical Equipment—contd.

months' testing of the d.c. equipment before the scheduled commissioning date, April 1, 1965, in spite of a number of adverse circumstances, the most important of which were:

(1) Turbines, generators and transformers were all much larger than any previously installed in New Zealand.

(2) The programme included much unusual equipment—16 kV busbars and switchgear, mercury arc converters with a great variety of associated indoor and outdoor equipment which, even with expert assistance from manufacturers, involved the Department in a great deal of additional work.

(3) The period between the Government's final decision to proceed with d.c. transmission in March 1961 and the commencement of trial operation in November 1964 necessitated quick preparation of specifications and setting early delivery dates. However, the unusual nature of most of the equipment and the fact that much of it required development beyond existing designs led to delays which still further concentrated construction work into the last few months, thus aggravating the already difficult task of assembling and using effectively a workforce sufficiently large and appropriately skilled.

(4) Although the powerhouse site was generally favourable, failure of some exposed rock batters hindered Ministry of Works construction forces in making working area available.

It has been both a privilege and a satisfying experience for the author to have been one of the many persons in this and other countries by whose joint efforts this rather unusual power station has come into being.

The author thanks the members of the power stations section of the N.Z. Electricity Department design office for their very considerable assistance in the preparation of this paper, and the General Manager for permission to present it.

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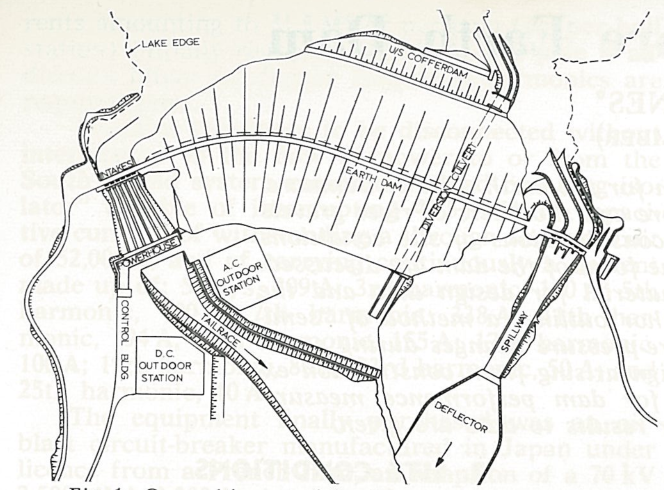


Fig. 1: General layout of the Benmore power project.

NOTATION

b	Horizontal width of slice
c'	Cohesion in terms of effective stress
$'c'$	Cohesion in terms of semi-total-stress
F	Factor of safety
F_{min}	Minimum factor of safety
h	Height of wave (crest to trough)
i	Angle of slope of dam face
K'	Dimensionless constant (weight of rip-rap)
k	Coefficient of permeability
l	Length along slip surface
n	Porosity
q	Horizontal seismic acceleration factor
S	Specific gravity of rip-rap
u	Pore pressure
u_s	Excess pore pressure with respect to toe water level
V_w	Rate of drawdown of lake
W	Total weight of soil plus any free water vertically above the slip surface
W_s	Weight of soil (saturated and/or moist) vertically above the slip surface
W_1	Total weight of soil, plus any free water at the head of the slip surface, vertically above the slip surface and above toe water level
W_2	Buoyant weight of soil vertically above the slip surface and below the toe water level
W_r	Weight of stable rock for rip-rap
α	Slope angle of slip surface
γ_s	Density of saturated soil
γ_w	Density of water
σ	Normal stress on slip surface
σ_3	Minor principal stress (e.g. confining pressure in triaxial cell)
ϕ'	Angle of internal friction in terms of effective stress
$'\phi'$	Angle of shearing resistance in terms of semi-total-stress

The closely-spaced fine joints had been filled with low-strength secondary minerals such as calcite. Permeability values (from borehole tests) ranged between 10^{-7} and 10^{-3} cm/s.

The overlying river gravels consisted of well-graded gravels containing stones up to 9 in, and generally had less than 5% passing the 200 mesh sieve. Lenses of siltier gravels with up to 7% passing the 200 mesh sieve occurred here and there throughout the deposit. The stones were well-rounded hard greywacke.

The talus on the abutments formed heterogeneous deposits of coarsely-graded argillite detritus containing variable contents of silt-clay weathering products and wind-blown sand.

3. CONSTRUCTION MATERIALS

Preliminary investigations (2) indicated that the most promising sources of construction materials for the earth dam were:

(a) Silty gravels from a basin 2½ miles from the damsite (area A).

(b) River gravels from the damsite area and from a river terrace deposit one mile upstream (areas E and F respectively).

(c) Rock-fill obtained from excavations for the diversion, spillway, intake, and powerhouse structures.

3.1. Impervious Material

The main source of impervious material (area A North) contained silty weathered gravels being residual products of argillite, greywacke and sandstone. The material was somewhat stratified and variable, and material near the surface tended to contain fewer fines, but mixing during borrowpit excavation was expected to rectify this.

Ninety per cent of the samples tested contained more than 6% passing 200 mesh (see Fig. 2); the −36 fractions had plasticity index values between 0 and 20 and liquid limits between 16 and 40.

It was estimated that 1,000,000 yd³ of unsuitable materials would have to be stripped to waste and that 6,000,000 yd³ of suitable material was available if the water table could be drawn down by a system of cut-off drains. Lest this borrowpit should prove insufficient, another area (A South) was proved for quality and additional quantity.

3.2. Pervious Gravels

The river gravels available in areas E and F were generally similar to those at the damsite, except that area F was overlain by silty gravels resulting from intermixing with local outwash material.

Average groundwater level was several feet above river level and the rock base of the deposits was generally well below river level. The total quantities available (down to rock level) were estimated at over 17,000,000 yd³ of which 2,500,000 yd³ would be unsuitable strippings.

3.3. Rock-fill

It was expected that the 1,500,000 yd³ of rock-fill available from nearby foundation excavations would provide a material having high permeability and frictional qualities, but when rock excavation was

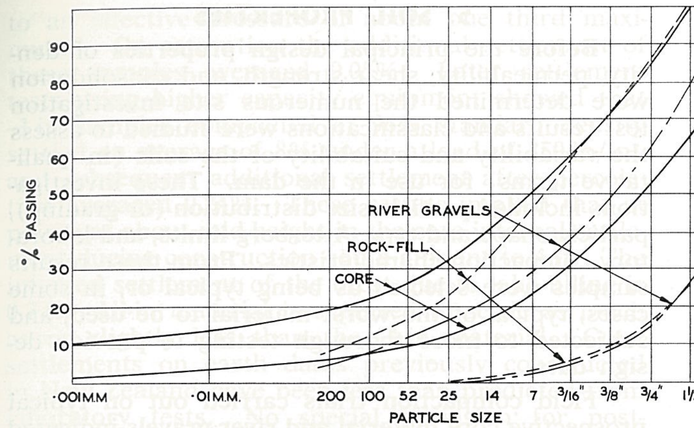


Fig. 2: Grading curves of available materials.

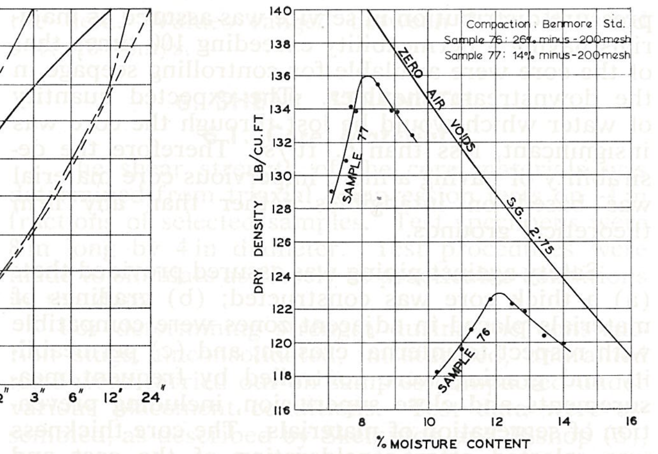


Fig. 3: Laboratory compaction curves on −¾ in core materials.

under way it yielded rock-fill whose grading was even finer than the river gravels; much of the in-situ rock was rippable by heavy tractors. This breakdown in the rock occurred particularly in the argillites and appeared to be due to the presence of the microscopic jointing and the weak vein infill material.

4. GENERAL ARRANGEMENT

4.1. Functional Requirements

The dam would have a narrow operating range of water levels (normal: el. 1,178 to 1,180 ft, design flood level el. 1,183 ft) and a freeboard of 7 ft above design flood level. The lowest level to which it would be possible to draw down the lake would be approximately el. 1,100 ft; this would probably occur very infrequently for special maintenance on the higher-level hydraulic structures.

To minimise interference of the dam slopes with the spillway and diversion entrances and with the intake forebay and powerhouse, the dam axes through each abutment were asymmetrically inclined upstream and were joined by a 1,600 ft radius curve. To keep the diversion culverts as short as possible the upstream cofferdam was incorporated in the shoulder of the earth dam and for this reason construction of the cofferdam was subject to some design control. At the downstream toe of the dam an extensive berm was provided as a service area.

4.2. Form of Dam

Preliminary studies indicated that the most suitable form of construction for the dam would be an inner core of material from area A, flanked by pervious shoulders of river gravels and the rock-fill available from excavations (see Fig. 4). This form, with only three zones at any level, was selected for its simplicity although it was expected to lead to some wastage of materials whose permeability properties lay midway between core and shoulder requirements. The alternative of providing for a number of intermediate sub-zones was considered to be unnecessarily complicated. It was later found during construction that the latter alternative would have been the better choice (see section 12).

Foundation seepage control was provided by an open trench down to rock, back-filled with core material, and a grout curtain in the rock foundation.

The total volume of material in the dam (including cofferdam and berm) was 15,600,000 yd³, and the estimated cost of construction, including overheads, was £6,100,000 (1957 costs).

4.2.1. *Core*

The proposal to use a core material whose average permeability could not be expected to be much below 10^{-6} cm/s was adopted with some caution as very few high earth dams had been constructed with core zones of this permeability. Satisfactory pore-

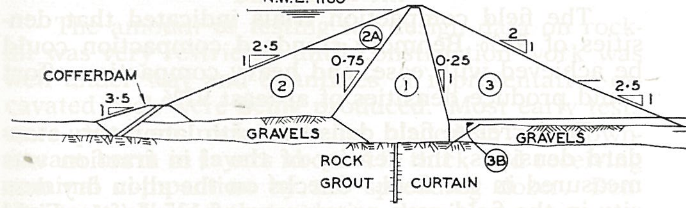


Fig. 4: Cross-section of dam, original design.

Zone	Material	Av. Permeability 10^{-6} cm/sec.	Min Av Dry Density lb/cu.ft.	Min. Av. Compaction
1	Area A (Core)	< 1	—	94% Std.
2	Gravels	> 100	140	65% R.D.
2A	Gravels or Rockfill	> 1000	115	65% R.D.
3	Gravels or Rockfill	> 100	115	65% R.D.
3B	Gravels or Rockfill	> 1000	115	65% R.D.

pressure distribution in service was assured as materials having a permeability exceeding 100 times that of the core were available for controlling seepage in the downstream shoulder. The expected quantity of water which would be lost through the core was insignificant, less than 0.1 ft^3/s. Therefore the desirability of having a more impervious core material was based on intangibles rather than any firm theoretical grounds.

Safety against piping was assured provided that: (a) a thick core was constructed; (b) gradings of materials placed in adjacent zones were compatible with respect to internal erosion; and (c) permeability and grading were controlled by frequent measurements and close supervision, including prevention of segregation of materials. The core thickness was selected after consideration of the cost and availability of core material, the stability effects, and current earth dam design practice. Had the selected slopes for the core ($\frac{3}{4}$:1 upstream, $\frac{1}{4}$:1 downstream) been much flatter the outer slopes of the dam would have required flattening to maintain allowable factors of safety against sliding. On the other hand, a reduction in the core thickness would have given unusually high hydraulic gradients for a core of this permeability.

Within the core trench it was first thought desirable to make the ratio of core width to nominal hydraulic head similar to that higher up the core, in which case much of the trench would have been over 300 ft wide. The expected cost of excavating and back-filling the core trench was high, so it was decided to reduce the width of the base of the trench to $\frac{2}{3}$ hydraulic head, provided that good quality core material was selected for placing in this region.

4.2.2. Shoulders

As production of rock-fill from excavations for other structures was scheduled to take place concurrently with most of the earth dam construction, it was intended to allow for the optional placing of rock-fill at any level in the downstream shoulder (the more accessible from most rock excavations) thus providing flexibility in construction and minimising the costs of stockpiling. In the upstream shoulder a zone of permeable gravels or rock-fill provided in the lake draw-down range allowed for absorbing rock-fill from the spillway intake. Elsewhere in the shoulders it was intended to use river gravels from the core trench excavation, and from areas E and F.

Batters for the outer slopes of the dam were selected to provide acceptable factors of safety for shear stability. During the preliminary design work it appeared that the rock-fill would have at least as high a permeability and shear strength as the river gravels while its dry-density would be about 20% lower; these assumptions were made for early stability studies.

Subsequent events cast doubt on the rock-fill properties and resulted in design modifications (section 12).

5. SOIL PROPERTIES

Before the principal design properties of density, permeability, shear strength, and consolidation were determined the numerous site investigation test results and classifications were studied to assess the variability and suitability of the soils (in qualitative terms) for use in the dam. These investigations included particle size distribution (or grading), particle shape and type, Atterberg limits, and laboratory compaction characteristics. From these results samples were selected as being typical or, in some cases, typical of the worst material to be used, and subjected to more thorough testing to provide design data.

Field compaction trials carried out on typical prospective core material and river gravels indicated the practicability of achieving different degrees of compaction in the field and allowed correlations to be made between field and laboratory tests for density and permeability. In the case of the coarse Benmore materials these correlations were important because limitations on size of laboratory equipment could give rise to non-representative test results.

The following sections outline some of the tests used to obtain design data. Design properties selected for final stability analyses are tabulated in Fig. 6. Shear strength testing is described separately in section 6.

5.1. Core Material

Most laboratory testing on core material was restricted to the $-\frac{3}{4}$ in fraction, which formed approximately 70% of the all-in material. For laboratory compaction tests a mould 6 in diameter by 5 in high was used. "Benmore standard compaction" was applied with a rammer giving the same energy input per cubic foot as for B.S. 1377:1961, test 10. The Benmore standard compaction gave densities approximately 1% higher than B.S. 1377. Figure 3 shows typical compaction curves for the -3 in fraction of core material. For laboratory permeability tests, where omission of coarse sizes exhibiting any particle interference could give misleading results, the -3 in fraction was tested in an 18 in diameter permeameter. Permeability test results were variable but for samples selected as representing usable material the range was 3×10^{-6} to 1×10^{-8} cm/s.

The field compaction trials indicated that densities of 92% Benmore standard compaction could be achieved with ease, and heavy compaction effort could produce densities of at least 97%.

To correlate field densities with laboratory standard densities, the density of the $-\frac{3}{4}$ in fraction was measured in the field. Checks on the all-in dry density in the field gave an average of 135 lb/ft^3. Field permeability tests carried out on the trial embankments using the standpipe method (3) gave an average permeability of 0.24×10^{-6} cm/s. At the investigation stage, measurement of the settlement characteristics of core material was restricted by test equipment limitations; $-\frac{3}{4}$ in samples compacted at about standard density showed average settlements of 5% under a vertical load of 100 lb/in^2 (equivalent

to an effective pressure of about one third maximum). On saturation the additional settlement of these samples averaged 0.05%. Later settlement tests using higher capacity equipment showed that $-\frac{3}{4}$ in samples compacted at 96% standard density settled an average of 8% under a load of 350 lb/in^2, and subsequent additional settlement after percolation averaged 0.24%. These results implied that at points at about mid-height in the core internal settlement during construction could amount to 4 ft. The inferred settlement of the crest due to lake filling is 0.7 ft. All-in material is expected to settle as much as or slightly less than the $-\frac{3}{4}$ in material. Crest settlements on earth dams previously constructed in New Zealand have been less than predicted from laboratory tests. No special provision for post-construction settlement was made in the Benmore design.

5.2. River Gravels

Compaction trials on typical river gravels confirmed that dry-densities of all-in material averaged over 140 lb/ft^3. The method of assessing degree of compaction was by relative density (4), whereby the dry-density of the -3 in material in the field was related to the maximum and minimum densities attainable on the same sample in a 2 ft^3 mould in the laboratory. The compaction trials indicated that 90% relative density could be achieved with four passes of a $3\frac{3}{4}$ ton vibrating roller and that two passes could produce over 70% relative density.

Laboratory permeability tests on the gravels were done on -3 in fractions in the 18 in diameter permeameter. Results ranged between 10^{-1} and 10^{-3} cm/s. Later standpipe field tests on materials being placed in the upstream cofferdam indicated that 10^{-3} cm/s was likely to be a more typical permeability where lenses of silty gravels could become mixed with the main body of gravels.

5.2.1. *Gravel Foundations*

In-situ density tests in the gravels at the damsite showed a wide scatter of densities but the average relative density was 65% and average dry-density 140 lb/ft^3. Both in-situ standpipe and borehole permeability tests were carried out giving results ranging from 10^{-1} to 10^{-3} cm/s.

5.3. Rock-fill

The amount of testing for design data on rock-fill was very restricted until construction work was well under way and quantities of representative excavated rock were being produced. Most early tests were done on rock-fill being placed in the downstream berm in layers about 4 ft thick and receiving nominal compaction by the spreading dozer. Density tests showed that relative densities were generally below 65%. Later density tests taken after vibrating rollers were used on 3 ft layers gave relative densities of over 70%. All-in dry densities averaged 121 lb/ft^3; some of the coarser graded materials were as low as 114 lb/ft^3, even when they had a high relative density.

Most permeability investigations for rock-fill were done as standpipe tests on rock-fill placed in the field. Values ranged between 3×10^{-3} and 3×10^{-1} cm/s.

6. SHEAR STRENGTH

6.1. Core Material

The shear strength of the core materials was determined from triaxial compression tests on $-\frac{3}{4}$ in fractions of selected samples. Test specimens were 8 in long by 4 in diameter. Test procedures were made to simulate as closely as practicable conditions in the dam.

For determining strength during the construction stage, unconsolidated, unsaturated, undrained tests were carried out on samples compacted under various placement conditions. Test data were assembled, as described by Skempton and Bishop (5), showing values of the ratio of pore pressure to maximum principal stress. The highest value of this ratio attained in triaxial tests was 0.85 for a sample compacted at optimum + 0.3% moisture content.

For conditions of steady state seepage with constant external loading, consolidated, saturated, undrained tests were carried out. Typical effective strength parameters were $c' = 1.2$ lb/in^2, $\phi' = 39.5°$.

Lake drawdown conditions were simulated by generating a pore pressure within a saturated specimen subjected to selected deviator and confining loads then shearing it by maintaining the deviator load while reducing the confining pressure. Effective strength parameters were $c' = 0$, $\phi' = 43.4°$.

For analysis of earthquake loading a special method of "semi-total-stress" analysis was developed. During the rapid changes of load in the structure, changes in the steady-state pore pressures will be induced. It is therefore not valid to appy the steady-state pore pressures to the stability analysis using effective stress parameters of shear strength.

Triaxial cell specimens were tested in a sequence which attempted to simulate the steady seepage pressure state followed by the application of additional load with no further drainage allowed. The specimen was confined and saturated, and a selected pore pressure (u_1) was imposed upon it, then drainage was closed off and the deviator load applied to shear the specimen. Tests were carried out for a number of selected values of u_1 and σ_3. The shear strengths were plotted on the Mohr diagram in terms of total normal stress on the shear plane less a steady-state pore pressure, u_2. The value of u_2 was obtained from the triaxial test observations at the point when the deviator stress had reached half its final maximum (i.e. when the factor of safety was approximately 2).*

A Mohr envelope was plotted for each series having a constant value of u_1/σ_3 (σ_3 = confining pressure), i.e. representing a particular degree of consolidation. With the exception of $u_1/\sigma_3 = 0$, where it was suspected that these test specimens

*An alternative method would have been to select u_2 (instead of u_1), impose this pore pressure on the specimen when the deviator stress had reached half its estimated maximum and then close off drainage for the remainder of shearing.

were not properly saturated, different values of u_1/σ_3 gave very similar Mohr envelopes. The parameters so obtained were: $'c'=7$ lb/in², $'\phi'=17°$, where $'c'$ and $'\phi'$ are "semi-total-stress" parameters of shear strength (see Fig. 5) which could then be applied to the stability analysis for earthquake loading, using the steady-state pore pressure values. The loads applied in the laboratory test to simulate earthqake loading were applied at ordinary rates of strain (0.1% per minute) and were therefore "rapid" only in relation to dissipation of pore pressure. The strength parameters so obtained did not take into account any increase in shear strength which may be mobilised by the "viscous" resistance of the soil under very rapid application of load. Some authorities provide for doubling the "cohesion" to take this effect into account. In the absence of suitable test equipment to check this "viscous" effect no allowance was made for this strength increase for Benmore dam.

6.2. Shoulder Material

Assessment of the shear strength of the river gravels and rock-fill was made difficult by the coarseness of these materials. Triaxial tests were carried out in a large testing machine which took specimens 22.5 in long by 9 in diameter, allowing $-1\frac{1}{2}$ in material to be tested. In addition to these laboratory tests, angle of repose measurements were made on site on high stockpiles of the materials to provide an indication of likely shear strengths of unconfined uncompacted materials.

6.2.1. *River Gravels*

Triaxial tests carried out were mainly consolidated, saturated, drained series. Varying degrees of initial compaction gave the strength parameters shown in Table 1.

Special undrained tests were not carried out for earthquake strength data. Dilatancy occurred throughout all tests, so it is unlikely that positive pore pressure changes will occur if additional shear strain occurs under earthquake loading.

Angle of repose measurements for end-tipped river gravels averaged 35° on batters up to 30 ft high. High faces of a stockpile of river gravels when excavated from the toe stood temporarily at 45° but they gradually flattened to 35°-37° over a period of time.

TABLE 1

Strength Parameters—River Gravels

Average relative density (%)	*c'* (lb/in²)	*ϕ'*
55	3.3	42°
67	1	44.4°
83	2	46°

6.2.2. *Rock-fill*

Laboratory triaxial tests on the $-1\frac{1}{2}$ in rock-fill were particularly difficult with membranes being frequently punctured by the sharp particles. Consolidated, saturated, undrained tests were carried out at two different degrees of compaction:

Average relative density = 52%, c' = 3.4 lb/in², ϕ' = 38.8°.
Average relative density = 75%, c' = 4 lb/in², ϕ' = 39°.

Pore pressure measurements taken during the tests indicated contraction of specimens during straining. A semi-total-stress plot was therefore constructed for application to shear strength during earthquake. For 75% relative density the result was $'c'$ = 8 lb/in², $'\phi'$ = 33°.

Angle of repose measurements on 20 ft high faces during early placement of rock-fill in the downstream berm of the dam gave the following:

End-tipped banks: Coarse grading, 39.5°; finer grading, 37°.
Batters of an embankment excavated from the toe: 42° to 44½° (stable).

7. FLOW NETS

Pore pressures that were dependent on seepage and independent of loading conditions were assessed by the construction of flow nets. To allow for the effect of stratification due to earth placing procedures the ratio of horizontal to vertical permeability was assumed to be 9:1. This approximate ratio was derived from a study of steady-state flow nets plotted from piezometer records from the Cobb dam, which is constructed with materials and methods similar to Benmore's.

For steady-state seepage the flow net was constructed by a graphical trial-and-error method. The resulting equipotential lines are shown in Fig. 6.

To check the infrequent condition of partial drawdown of the lake a flow net was constructed to represent seepage pressures in the dam when the lake is drawn down 100 ft in 14 days. The fall of the phreatic line in the upstream shoulder and core

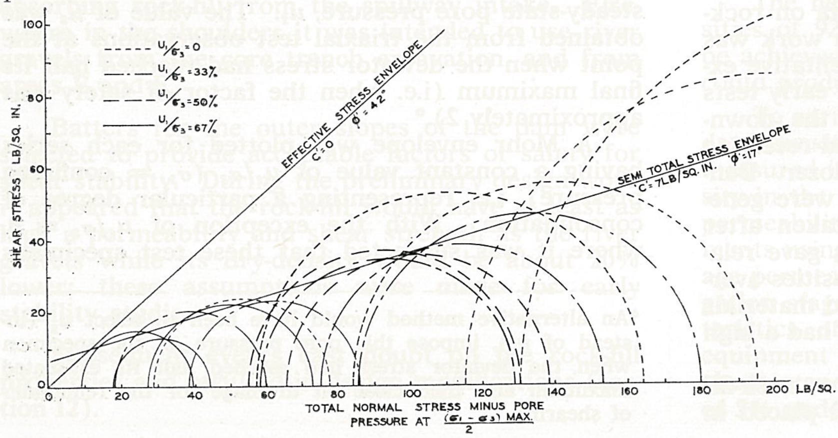


Fig. 5: Shear strength envelope for semi-total-stress analysis for core material.

Conditions analysed

SS Steady state seepage
Lake full
Earthquake factor 0·1

CS Construction stage
Construction pore pressure $\bar{B}$ = ·85
Lake empty for upstream slips
Lake full for downstream slips

DD Drawdown of lake in 14 days

Zone	Material	Moist Density lb/cu.ft	Saturated Density lb/cu.ft.	C' lb/sq.in	tan ϕ'	Undrained Earthquake 'C' lb/sq.in.	Undrained Earthquake tan 'ϕ'
1	Area A (Core)	150	152	0	0·8	7	0·3
2 C/D F	Gravels	144	150	0	0·9	0	0·9
3 H	Gravels	150	—	0	0·95	0	0·95
3	Rock fill	134	—	0	0·81	0	0·81
3S	Rock fill	—	145	0	0·81	8	0·65
2/O	Rockfill (Coarse)	116	135	0	0·78	0	0·78
3 H/O	Rockfill (Coarse)	116	—	0	0·78	0	0·78
3/O	Rock fill	116	—	0	0·725	0	0·725

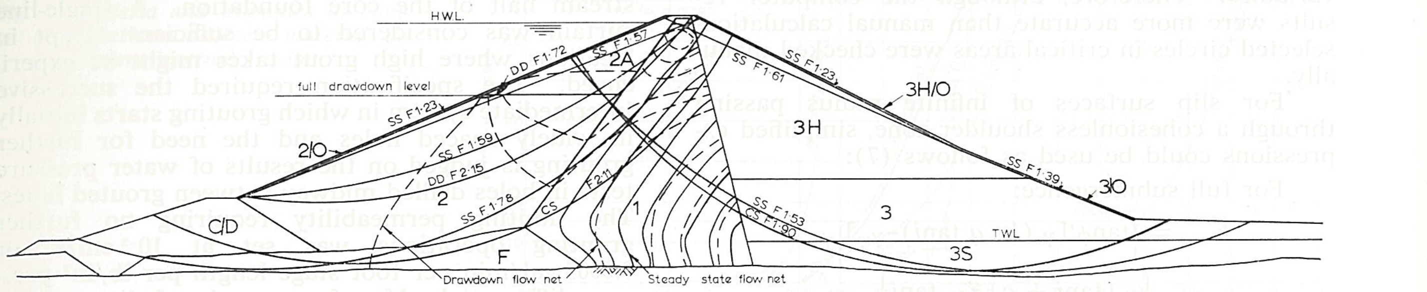


Fig. 6: Stability analysis: data and results.

zones was computed from data given by Reinius (6). Values of (k/nV_w) were taken as 3.3 for the upstream shoulder and 0.01 for the core (where k = permeability, n = porosity, V_w = rate of fall of water surface on the upstream face of the zone). From interpolation of Reinius's data these ratios correspond to a rate of fall of phreatic line in the upstream shoulder of 65% of lake drawdown rate and a negligible rate of fall in the core zone. Using this boundary data a new flow net was constructed to show the modification to the steady state net caused by drawdown (see Fig. 6).

8. STABILITY ANALYSES

8.1. Method

For stability calculations the dam was divided into zones according to their weight and shear strength properties. Figure 6 shows the subdivision of these zones, the properties used, and selected results obtained for the final stability checks. These were carried out to determine the factor of safety against shear failure on numerous potential slip surfaces of constant radius (including infinite radius). The general equation applied for factor of safety including earthquake was:

$$F = \Sigma[c'l + (\sigma - u)\tan\phi']/[\Sigma W\sin\alpha + \Sigma qW_s\cos\alpha]$$

This method of analysis relies on the assumption that during an earthquake the dam is oscillated by its rigid foundation and that the momentary horizontal acceleration at all points along a potential slip surface is $q \times$ gravitational acceleration. The inertia of the portion of the dam above the slip surface thus provides an additional horizontal disturbing force acting in opposite sense to the earthquake acceleration. The assumption that the value of q at any instant is the same throughout the length of the slip surface ignores the dynamic effects that modify the accelerations in the body of the dam. Until further research can solve the dynamic response of earth structures the simplified approach is justified since it is likely to be conservative through taking no account of damping.

To further simplify the analysis the following assumptions applied:

(a) The earthquake inertia force on each vertical elemental slice was applied at the slip surface (an assumption leading to slightly conservative results (7)).

(b) During earthquake the change in external lake-water load on the dam is negligible (8).

For preliminary analyses the equation for factor of safety was solved by graphical integration of vertical slices using the "phreatic-buoyant" method (7). For final stability checks on slip circles Bishop's "rigorous" arithmetic method (9) was adopted using the equation:

$$F = \Sigma\left\{\sec\alpha[c'b + \tan\phi'(W_1 + W_2 - bu_s)]/\left[1 + (\tan\phi'\tan\alpha)/F\right]\right\}/\left\{\Sigma(W_1 + W_2)\sin\alpha + \Sigma qW_s\cos\alpha\right\}$$

In 1961 a computer programme was written to solve the Bishop equation using the I.B.M. 650 electronic computer (10).

The programme provides for specifying the external and internal geometry of the dam with 50 co-ordinated points. Soil properties of strength and weight can be specified for up to seven independent zones (including foundations). Allowance is made for earthquake forces where required. Other data specified are pore pressures (in seepage zones these are defined on a grid of up to 80 points), earthquake conditions, and the required slip circles (co-ordinates of centres and radii). The computer analysis is carried out in two passes. In phase A, on receiving punched data on the geometry of the dam, the computer distributes vertical slices over the dam section and computes a table of intercept of slices on all zone boundaries. This is produced as a punched card output which is then added to a set of input cards containing soil properties, pore pressures, slip circles, and other control data which is then fed through the computer for phase B of the computations. As the equation for the Bishop method involves successive approximation, the computation of F for each slip circle is done by iteration until the value of F is stable in the second decimal place. The print-out gives the values of factor of safety along with various identifying data.

The complicated nature of the computer programme, arising from making it applicable to a very wide range of conditions, made it difficult to prove it correct under every foreseeable combination of variables. Therefore, although the computer results were more accurate than manual calculation, selected circles in critical areas were checked manually.

For slip surfaces of infinite radius passing through a cohesionless shoulder zone, simplified expressions could be used as follows (7):

For full submergence:

$$F_{min} = \left\{\tan\phi'[\gamma_s(1 - q\ \tan i) - \gamma_w]\right\} / \left\{\gamma_s(\tan i + q) - \gamma_w \tan i\right\}$$

Where F_{min} = factor of safety on a shallow plane parallel to the outer slope.

For an unsubmerged slope:

$$F_{min} = [\tan\phi'(1 - q\tan i)]/[\tan i + q]$$

8.2. Results

Lowest factors of safety for various conditions analysed are summarised in Fig. 6; shallow slip surfaces frequently had lower factors of safety than deep-seated ones. Values of minimum allowable factors of safety varied according to the conditions. Slip surfaces that cut through the core wholly below lake level were required to have F values exceeding 1.5. For other slip surfaces a minimum of 1.25 was considered desirable. The lowest factor of safety obtained (1.23) was for very shallow slip surfaces located in the surface layers of the dam where ϕ' was assumed to be similar to the angle of repose. This type of failure during earthquake would have no dangerous or expensive consequences and for this reason most authorities would accept $F = 1$ for such a condition.

9. FOUNDATION AND ABUTMENTS

9.1. Preparation

Over the areas of core-rock contact the rock was required to be thoroughly cleaned, and a special silty grading of core material was specified for placing against the rigid base. Excavated rock batters were limited to 1:1 (when averaged over 20 ft) to facilitate compaction of the special core material into the irregular surface. Slopes of more regular surfaces such as concrete or smooth rock were allowed to be 0.25 (horizontal):1.

The foundations and abutments for the shoulders of the dam required no special treatment except: (a) the removal of all in-situ soil under the upstream shoulder in the design drawdown range; and (b) the removal of silty deposits whose permeability could approach that of the core or whose frictional strength could be low.

9.2. Grout Curtain

Preliminary borehole tests indicated that some sections of the foundation rock were considerably more permeable than the core and could possibly approach the vertical permeability of the in-situ gravels and talus forming the downstream shoulder foundation. A grout curtain was therefore provided along the length of the dam under the upstream half of the core foundation. A single-line curtain was considered to be sufficient except in locations where high grout takes might be experienced. The specification required the successive intermediate system in which grouting starts initially in widely spaced holes and the need for further grouting is judged on the results of water pressure tests in holes drilled midway between grouted holes. The limiting permeability requiring no further grouting operations was set at 10^{-5} cm/s, or 0.0003 gal/min per foot stage length per lb/in² pressure differential. After four months of site grouting work it became clear that cement grouting could not achieve the required permeability as the finely jointed rock was preventing the intrusion of cement. Trials with bentonite having finer particles showed no improvement. Rather than adopt a more expensive grouting medium it was decided to allow for a more pervious grout curtain and to prevent the development of seepage pressures under the downstream shoulder by constructing a gravel drain of high permeability (10^{-3} cm/s) along the downstream edge of the core-rock contact. Curtain grouting then continued with a revised permeability requirement of approximately 10^{-4} cm/s.

10. SURFACING

10.1. Downstream Face

Local conditions of dry climate and scarcity of topsoil restricted the choice of downstream face protection to either rock-fill or river gravels. Rock-fill was specified because of its slightly greater angle of repose and better stability against surface movement. For the 2.5:1 slope the surfacing was specified to have a grading similar to that of the river gravels and for the 2:1 slopes a coarser grading was specified having not more than 50% passing 2 in (i.e. D_{50} greater than 2 in).

10.2. Upstream Face

The degree of upstream surface protection varied at different levels according to the expected service. The 3.5:1 cofferdam face was left as river gravels except near the diversion inlet where temporary rip-rap was provided to resist transverse velocities during diversion floods.

Between the cofferdam crest and el. 1,080 ft, where only nominal wave protection was required during lake filling, a rock-fill surfacing 3 ft thick with D_{50} greater than 2 in was provided.

Between el. 1,080 and 1,170 ft, where the lake level may be held static for a period during the initial filling or during future maintenance, rip-rap was required to be quarried from a sound greywacke quarry to provide a layer 3 ft thick with D_{50} greater than 4 in.

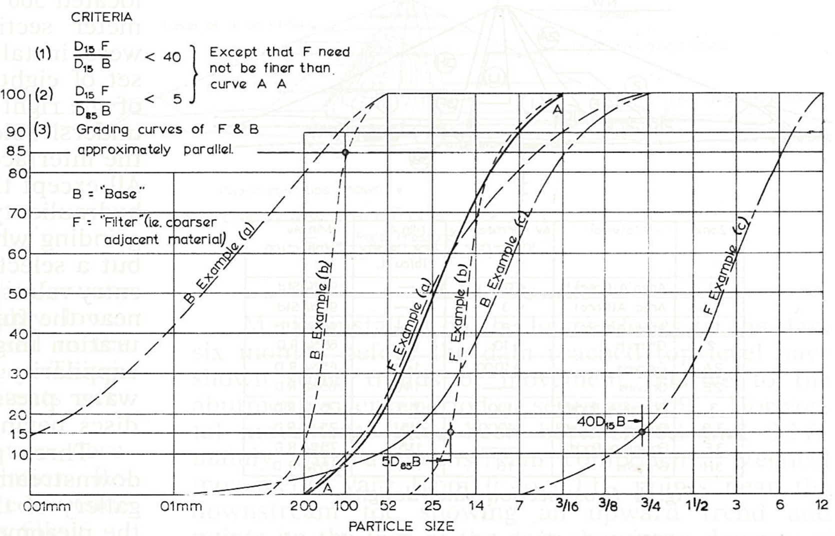


Fig. 7: Criteria and examples of compatibility between adjacent materials to prevent internal erosion.

For the 10 ft above and below operating level el. 1,180 ft the rip-rap layer was specified to be 5 ft thick and having D_{50} greater than 16 in. This size requirement was checked by wave forecasting using data by Bretschneider and Putz (11) and Hudson's modification of the Iribarren formula (11) for rip-rap stability. A wave height of 4 ft was forecast for a quarter-hour wind velocity of 70 mile/h over a one mile clear fetch across the lake. The rip-rap formula used was:

$$W_r = [K'\gamma_w S h^3 \tan\phi']/[(\tan\phi' \cos i - \sin i)^3 (S-1)^3]$$

This formula when applied to the D_{50} size is considered to be fairly conservative (12).

11. CONSTRUCTION CONTROL

Where practicable the specification laid down end-result requirements that could be checked by test measurement. However, as test measurements could not cover all aspects of quality control it was necessary to specify construction procedures to some extent, and some features of the specification were:

Reasonable latitude was allowed in the grading of materials but specific controls were imposed on compatibility between materials of adjacent zones (see Fig. 7) to prevent erosion of one into the other, and specific field permeability test control was required (3). Evaluation of a group of permeability results in shoulder materials was made in terms of the median value as this gives a closer measure of effective permeability for flow across the stratification. For core material, evaluation of permeabilities was in terms of the arithmetic mean, giving a measure of effective permeability parallel to stratification.

Neither compaction plant type nor layer thickness restrictions were specified for the placement of core material, but field tests were required to include the bottom of the placed layer.

Where specified limits were close to design assumptions for material properties, frequent testing and remedial action was necessary. Specific removal or retreatment of material placed in the dam was required for zone 1 core material having substandard density or permeability and for zone 3B drainage material having substandard permeability.

Generally the specification required improvements in methods and control when test results indicate downward trends of averages rather than require later drastic action when a lower limit is not attained.

Methods of field density and permeability testing for control were similar to those done during the investigations (2).

12. DESIGN MODIFICATIONS

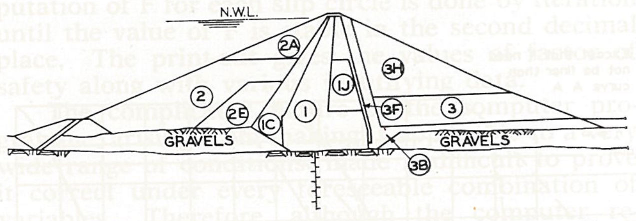
Figure 4 shows the initial design layout on which preliminary design analysis had been done. Early construction work followed this layout, but with the gain of construction experience and additional testing data various modifications appeared advisable.

12.1. Rock-fill

As rock excavation proceeded a large proportion of the resulting rock-fill was breaking down during handling to form a finer material than expected. Triaxial tests on this rock-fill indicated that it had a lower shear strength (particularly when saturated undrained) than the river gravels. It was therefore decided to make coarser grading limits for the rock-fill and exclude its use in the upstream shoulder. Relative density requirements for the remainder of the downstream shoulder were raised to partly compensate for the lower shear strength of the rock-fill. Final stability analyses (Fig. 6) were made with these modifications included.

12.2. Core Material and River Gravels

During the first season's placing, core material being hauled from the borrowpit frequently had marginal permeability, so an upstream sub-zone, 1C, of the core was introduced, and material whose quality appeared by visual inspection to be borderline was placed into this. The slightly relaxed permeability requirements for this sub-zone reduced the need to remove core material when, after de-



Zone	Material	Av. Permeability 10^{-6} cm/sec.	Min. Av. Dry Density lb/cu. ft.	Min. Av. Compaction
1	Area A (Core)	< 1	—	96% Std
1C	Area A (Core)	< 3	—	96% Std.
1J	Area A (Core)	< 3	—	96% Std.
2	Gravels	> 10	140	65% R.D.
2A	Gravels	> 1000	140	65% R.D.
2E	Gravels	—	140	65% R.D.
3	Gravels or Rockfill	> 100	115	75% R.D.
3 B	Gravels or Rockfill	> 1000	115	75% R.D.
3 F	Gravels or Rockfill	—	115	75% R.D.
3H	Gravels	> 10	140	75% R.D.

Fig. 8: Cross-section, final design.

layed permeability testing, it was found to be substandard for zone 1. At the same time density requirements for the core zone were increased in order to obtain lower permeabilities. At a later date a similar sub-zone, IJ, was provided in the downstream third of the core zone to help further to conserve the better-quality core material.

The river gravel borrowpits also yielded quantities of material of marginal permeability, so the upstream shoulder zone was provided with an inner sub-zone, 2E, where a relaxation in permeability was permitted. In the downstream shoulder the foundation drainage sub-zone, 3B, was re-designed to extend as a 30 ft thick drain up the downstream side of the core, thus requiring selection of higher-quality drainage material in this sub-zone but in turn allowing use of lower-permeability gravels in the bulk of the downstream shoulder. A transition zone, 3F, was interposed between the core and drainage zone 3B to ensure compatibility of adjacent materials.

Finally, near the completion of construction the thickness of the core over its upper 115 ft was reduced to allow further economy in construction. This reduction still allowed a minimum ratio of core width to nominal hydraulic head of 0.9 in the upper levels, which is greater than the corresponding ratio at the base of the dam.

These later modifications made to the dam section are shown in Fig. 8. Although some of the changes slightly modified the predicted pore pressures, their effects on stability were negligible.

13. DAM PERFORMANCE

Installations were designed to measure the subsequent performance of the dam and to provide checks on its stability against shear and general deformation.

13.1. Piezometers

Fifty-six piezometers measured pore pressures during and after the construction phase. Thirty-three of the piezometer tips were located in one vertical cross-section of the dam, shown in Fig. 9. Nine tips for correlation were placed on cross-sections located 300 and 600 ft away from the principal piezometer section, and six borehole-type piezometers were installed in the foundation rock. A further set of eight tips was installed on the concrete face of the right abutment gravity block to check that no undersirable seepage pressure condition occurs at the interface of core material and the steep concrete. All except three of the piezometers are the orthodox hydraulic type with porous discs of carborundum grinding wheels, 1 in diameter × ¼ in, No. 46 grade, but a selected number having a new type (high air-entry-value discs, Doulton grade P6A) was installed near the future phreatic line where only partial saturation might exist. Under unsaturated conditions cappilarity produces air pressures in excess of pore-water pressures, which allows air to pass through discs having a low air-entry pressure value (13).

Three gauge-houses are provided: two on the downstream face of the dam (Fig. 9), and one in a gallery of the right abutment gravity block, serving the piezometers on the concrete face. In the gauge-houses the piezometer tubes are connected to a master Bourdon gauge via headers and isolating valves. To remove any air bubbles from piezometer tubes an air-trap and flushing pump are arranged with a set of regulating valves so that during closed-circuit circulation of water through the system the pressure rise on the delivery side of the pump can be adjusted to be approximately equal to pressure drop on the suction side. Three "electrical-sonic" piezometers constructed at the Ministry of Works Central Laboratory were installed in the dam to check the practicability of this system of pore pressure measurement. These use the vibrating wire principle to measure the deflection of a pressure-sensitive diaphragm. This system of electrical measurement was selected because of its stability characteristics and general robustness.

13.1.1. *Results*

Dam construction reached virtually its final level in May 1964 and up to the time of writing (July 1964) pore-pressure levels from piezometer readings have been satisfactory. In zone 1 pore pressures expressed in terms of the ratio of pore pressure to nominal vertical pressure have generally risen to a maximum in the first six months after installation after which time the ratio for a given piezometer has remained practically steady for the remainder of the construction period. Table 2 shows averages of the maximum values of this ratio attained for each piezometer layer in zone 1 and the heights of overburden placed on each corresponding layer in the first six months after installation.

TABLE 2

Ratios of Pore Pressure to Nominal Vertical Pressure

Piezometer layer	*Average maximum ratio*	*Overburden height, first six months*
(el. in ft)	(%)	(ft)
870	29	45
955	50	30
1,050	44	30
1,145	Result not available	30

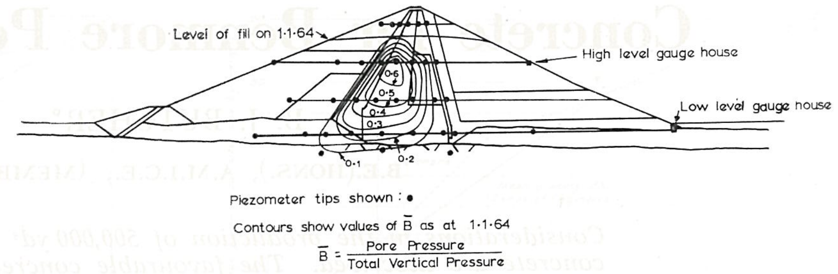


Fig. 9: Cross-section showing location of piezometers and pore pressures, January, 1964.

Figure 9 shows values of the ratio as at January 1964. The pore pressures are considerably lower than the value (average ratio = 85%) used in the stability analysis for the construction stage. This is because the average placing moisture contents were lower than the design assumption.

One of the installed sonic piezometers developed an unknown fault soon after installation. Results from the other two, which are located outside zone 1, will not be known until the lake is filled.

13.2. Internal Settlement Gauges

Two hydrostatic settlement gauges were installed at each of two levels to measure internal settlements of the dam. The operation of this equipment relies on measuring the level of a weir in a cell buried in the dam by hydrostatic transfer of the level to a manometer in the gauge house on the downstream slope of the dam. Before installation of this apparatus a full-size model, including 1,000 ft long manometer tubes, was satisfactorily tested at the Ministry of Works Central Laboratory (14).

13.2.1. *Results*

Despite the satisfactory tests in the laboratory the hydrostatic settlement gauges have given some erratic results. One of the cells shows apparent fluctuations in level of up to 0.6 ft from time to time. The most likely cause of trouble is the presence of moisture in the tube provided to transmit atmospheric pressure from the gauge-house to the cell. The method successfully developed in the laboratory for eliminating moisture from the air tube (by flushing it with methylated spirits) has so far been unsuccessful in eliminating the fluctuating readings in the Benmore installations.

Smoothing of the fluctuations in the recorded results indicates that when the dam reached full height a point in the core 50 ft above the foundation had settled 1.9 ft or 3.8%.

13.3. Surface Deflections

On the crest and downstream face of the dam 30 survey test points are installed to measure vertical and horizontal movements of the surface of the dam before, during and after lake filling. The test points are 5 ft³ concrete blocks cast below the surface and containing a target mounting. Their positions are located by intersection from two of four survey instrument pillars on the abutments close to the dam, and these in turn are related to three reference pillars set up approximately one mile from the dam using a geodimeter and triagulation.

13.3.1. *Results*

Marks installed on the lower slopes of the dam six months before the dam reached top level have shown some trends of movement relative to the abutments over a period of seven months. Horizontal movements have been between 0.1 and 0.2 ft, mainly with a downstream component. Vertical movements vary from 0 to 0.1 ft, points near the downstream toe showing an upward trend and points up the face of the dam showing a downward trend.

14. CONCLUSION

The principal lesson learned in the design of the Benmore earth dam has been the advantage of providing for flexibility in the design so that changes can be made to counter or make use of changed circumstances. Regardless of how thorough the preliminary investigations are, and they were very thorough at Benmore, there is an ever-present possibility of unexpected geological conditions being found during the construction work.

15. ACKNOWLEDGMENTS

Acknowledgment is made to M. H. White, who carried out most of the detailed design work described in this paper, and the author thanks the chief engineer (power), Ministry of Works, for permission to publish this paper.

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