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Maroon Dam Embankment and Foundation

by

K. D. NUTT, B.E., M.I.E.Aust.

Senior Engineer, Irrigation and Water Supply Commission, Queensland

1 INTRODUCTION

Maroon Dam is an earth and rockfill embankment on Burnett Creek, a tributary of the Logan River, about 100 km south-west of Brisbane, storing 39 000 megalitres of water at full supply level, 34 m above stream bed level. The wide unlined spillway 10.4 m above full supply level is expected to discharge only once in 50 years, lesser floods from the 105 km² catchment being absorbed in the 43 000 Ml flood storage. Outlet valves capable of evacuating stored flood water at 30 m³/s are located in a concrete conduit near bed level which initially served for diversion during construction.

Burnett Creek near the dam has cut into the mainly sedimentary rocks of the Walloon Coal Measures, which were shown by drilling and shaft excavation before and during the design of the embankment to consist at the dam site of almost horizontally bedded clay shales, siltstone, and fine argillaceous sandstone, all of only low to moderate strength when fresh. Basalt sills varying in thickness from about 0.5 m to 2.5 m have intruded along some bedding planes. In the stream bed, sedimentary rocks and basalt are only slightly weathered, but on the abutments are generally very weathered, especially near the surface. The bed and most of the abutments are covered by alluvial or colluvial material, generally shallow, but up to 9 m deep where they fill an old stream channel. Overlying the sedimentary rock from a few metres above full supply level on both sides of the creek is a jointed cap of slightly to very weathered porphyry, in which the spillway has been cut.

To suit site conditions the embankment was designed with a thick core placed wet of optimum to ensure flexibility to follow foundation settlements, the minimum rockfill zones needed to give adequate safety against failure within the embankment, and modest weighting zones, cheaply constructed from material stripped from spillway or borrow areas, to prevent failure through the rather low-strength foundation.

2 THE FOUNDATION PROBLEM

At an early stage of construction several slips occurred on the bank side of the excavation in clay shales for the diversion conduit. Movement was along well defined lines above conduit foundation level which were shown by exposures in this trench and in a number of short exploratory trenches in the bank to be in thin extensive layers of soft very weak clay which were almost horizontal, in a dark clay seam overlying a weathered basalt sill. Flattening the slope to 1 on 3 removed disturbed material and prevented further movement, and conduit construction proceeded.

The discovery of these seams showed how incomplete our knowledge of the foundation was, and when a brief investigation revealed other seams the situation was reviewed. As the low strength seams would have a major effect on dam stability, the nature, extent and properties of the seams would have to be investigated and any other foundation defects found, and the embankment design would need to be modified. Construction should proceed if at all possible while investigations and design modifications were being made.

To assist in investigations, to provide the geological expertise which the Commission at that time lacked, and to give engineering advice, a consulting firm experienced in foundations was engaged. Throughout the protracted investigation, information and opinions on interpretation and design significance were exchanged between consultant's and Commission staff. In the few areas where differences in opinion were not resolved, this paper presents the views finally adopted and acted upon by the designers.

3 FIELD INVESTIGATIONS

Preliminary investigations suggested, and later ones confirmed, that foundation strength is controlled by surfaces of very low shear strength occurring within some clay seams, and most work was concentrated on establishing the location and extent of these surfaces and finding the shearing strength along them.

The most important information was obtained by studying and obtaining undisturbed samples from the surfaces where these were exposed on the abutments by foundation preparation, and in backhoe trenches. Numerous soft clay seams were found in the clay shales, and within some were surfaces of great extent parallel to the stream, showing clear evidence of considerable movement in the cross-stream direction - probably sufficient to reduce strength to residual. For the distance into the bank which could be studied the slip surfaces showed no irregularities which would increase effective shear strength in the cross-stream direction, but in the direction parallel to the stream irregularities existed which were estimated to add at least 2^o to the effective strength. The slip surfaces were very inconspicuous until prepared for close inspection, but were often found in clay strata distinctive in appearance, often associated with dark carbonaceous shales, and often close to basalt sills. This provided a useful, though not infallible, guide in finding slip surfaces.

Only one soft clay seam was found below the stream bed, and this was studied and sampled where exposed by the slight dip of the strata, and in two

shafts. Though the seam was found to be very extensive, no evidence was detected to suggest any shearing movement.

Diamond drill cores from both old and freshly drilled holes were used to help determine extent, strike and dip of seams detected at the surface. It was rarely possible to detect actual slip surfaces in cores, even in the few cases when the position could be accurately determined from nearby surface indications. However, the stratum in which a slip surface was detected at the surface could usually be traced with certainty in diamond drill cores. As no cases were found in surface exposures of slip surfaces moving from one stratum to another it was assumed, though it could not be positively established, that the slip surfaces extended in from the surface within the clay seam unchanged in properties, at least as far as was important for stability calculations.

The difficulty in finding slip surfaces, even on stripped surfaces and in cores where they were known to exist, explains why they were not known and allowed for in the original design. However, available information on regional geology, if found at that time and read carefully, would probably have alerted the designers to the possibilities at the correct time.

4 GEOLOGY OF THE FOUNDATION

The geology of the immediate dam area is shown in a somewhat idealised and simplified way in

Drainage of the clay seams will influence foundation pore pressures. The basalt is jointed, and the shales and sandstones fissured, joints and fissures being sufficiently open to provide drainage. Some settlement of the foundation was expected under dam loading, mostly during construction, with a possible reduction in permeability of the sedimentary rocks.

5 TESTING FOR SHEAR STRENGTH

Laboratory testing consisted mainly of direct shear tests on 51 mm samples, at rates slow enough to avoid pore pressure generation. Tests on samples from the abutments were usually run in the direction of slip shown by striations, and indicated a residual strength of 8° . At commencement of testing the slip surfaces were already at or very close to residual strength. Limited testing across the striations confirmed the same residual strength of 8° , which was adopted for analysis.

Testing of the seam below the creek bed gave peak and residual strengths of 13° and 9° respectively, and showed that very substantial shearing movement was needed to reach residual strength. No pre-existing slip planes had been detected in the seam, so a value of 13° was adopted for design.

6 EMBANKMENT DESIGN

(a) Design Procedure.

To facilitate continuing with construction of

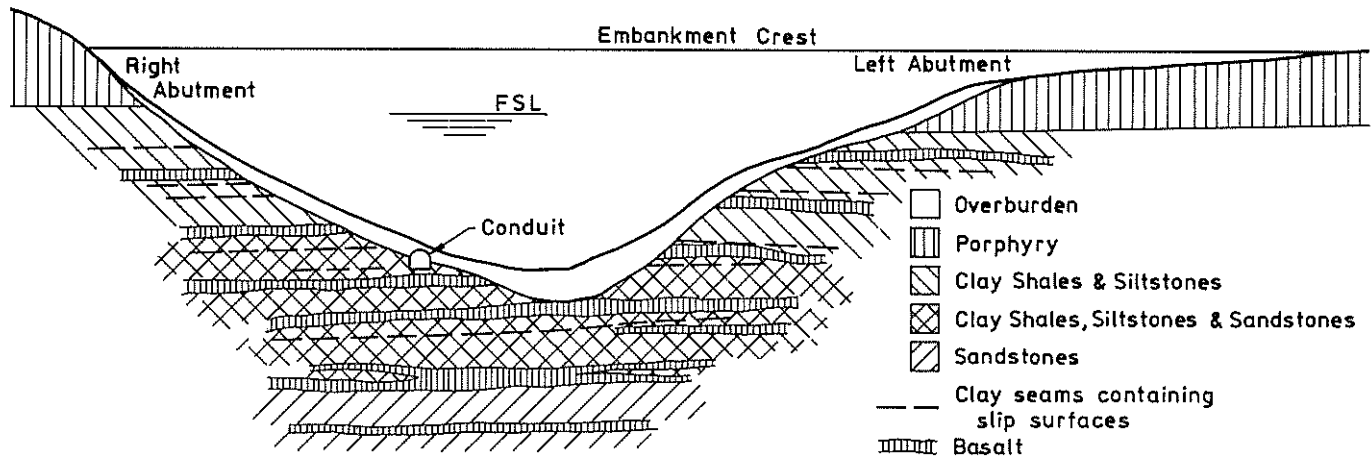


Fig. 1 Cross-section of foundations

the cross-section shown in Fig. 1.

The slip surfaces shown are diagrammatic only to indicate their known and assumed characteristics and their importance in considering dam stability. For the purposes of design, it was assumed that undetected slip surfaces could occur at any level, but that they would be parallel to those known, and would have a strength not lower than those tested.

The orientation of slip surfaces is of great importance to design. From careful matching of strata in surface exposures and cores, and from the dip and strike seen over limited areas at the surface, it was concluded that all strata are bedded almost horizontally, striking parallel to the dam axis and dipping upstream at approximately three degrees.

the dam during investigation and re-design, an early decision was made to retain the original cross-section of the main zoned embankment and achieve stability by increasing the relatively inexpensive weighting zones. The method of analysis was an adaptation of the normal limit equilibrium methods in which total shear strength under ultimate conditions along selected possible failure surfaces is compared with shearing forces actually developed along such surfaces, with the low strength slip surfaces having a dominant role in determining the failure surfaces to be analysed, as shown in Figure 2.

The method of analysis developed applies the method of slices, and allows for the effect of interslice shears. The shears used affect the calculated position of the "pressure line" of

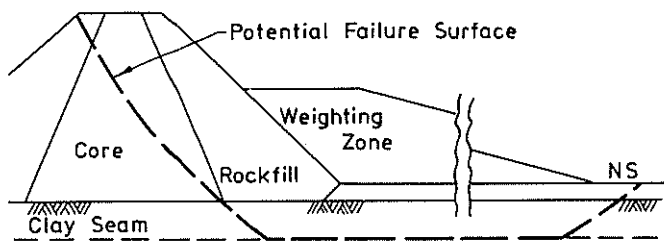


Fig. 2 Typical surface for analysis

normal forces across interslice faces, and are adjusted to obtain a satisfactory pressure line. The factor of safety is dependent on the interslice shears adopted, but is not sensitive over the range of values which give reasonable pressure lines.

The method is basically similar to the now well-known Morgenstern and Price method, and comparative tests conducted after that method became available to us gave similar results.

(b) Foundation Shear Strength

For analysing the deepest section 13° was adopted for the effective friction angle ϕ' , this being determined from tests on the clay seam under the creek bed. For slip surfaces in the abutments the residual strength of 8° was increased to 10° to allow for the effect of irregularities, as described in Section 3. However, Fig. 3 shows that a failure would necessitate shearing through rock between adjacent slip surfaces. As a three-dimensional analysis would be impossibly complicated, allowing for this effect by modifying ϕ' for use in the two-dimensional analysis is a realistic alternative.

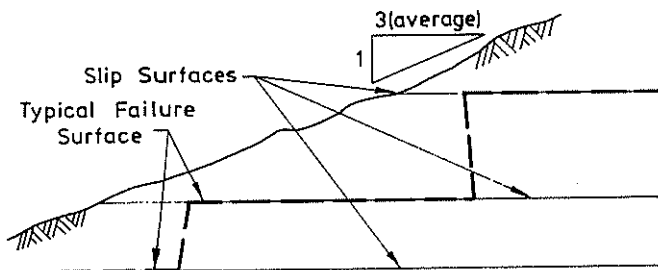


Fig. 3 Failure surface on abutment

Fig. 3 shows that the minimum ratio of length of failure surface through the rock to that through the seam is 0.33. Adopting a ratio of horizontal stress to vertical stress within the foundation of 0.6, and a ϕ' for the rock of 20°, it is seen that for a vertical force of P on the slip surface the normal force on the vertical plane in rock is $0.6 \times 0.33 P$ or $0.2 P$, and the total shear resistance is $P \tan 10^\circ + 0.2 P \tan 20^\circ = 0.25 P$. The value of ϕ' required to give a shear resistance of 0.25 times the normal force is 14°, but 13° was adopted for the stability analysis.

(c) Pore Pressures

Pore pressures in the core and the slip surfaces are particularly important in the stability analysis. For steady seepage and drawdown analyses, core pore pressures were conventionally

determined. For the steady seepage case, it was assumed that drainage provided by the rockfill zone and drilled foundation drains would prevent seepage from the reservoir from affecting pore pressure in the downstream slip surface, taken as being hydrostatic pressure with the water table at the natural surface. Effective connection to the reservoir was assumed for the upstream slip surface, and pore pressure was taken as hydrostatic pressure due to the reservoir.

Weighting zone configurations designed to satisfy steady seepage and drawdown requirements were checked for end of construction factor of safety. Core pore pressure was determined by assuming a B value of 0.6 in a core placed wet of optimum, and even to achieve a factor of safety of only 1.10, pore pressure on the slip surface had to be very low, corresponding to a B of only 0.04 for the section founded at El. 166.2, and about 0.10 for that founded at El. 173.8. Because it was considered that drainage to the jointed basalts and shales could keep pore pressures low, construction was allowed to proceed, but pore pressure measurements were taken regularly in the core and seams as discussed more fully later, to provide a basis for predicting ultimate pore pressure and for stability checks.

(d) Adopted Design

The cross-section shown in Fig. 4 is that for the maximum section. The weighting zone berm widths and batters were maintained constant over the width of the dam, but the berms sloped upwards as foundation levels rose up the abutment, finally rising to El. 207.26 to join the natural surface. Fig. 4 shows also some of the drainage provisions intended to ensure that the foundation pore pressures would not exceed the design values.

Factors of safety determined during design are shown in Table 1.

TABLE I
FACTORS OF SAFETY

Type of Failure	Factors of safety for foundation levels		
	El. 166.2	El. 173.8	El. 182.9
Steady seepage	1.44	1.42	1.42
Rapid drawdown	1.20	1.19	1.19
End of construction	1.10*	1.10*	--

* See section 6 (c) for pore pressure conditions.

They are lower than are commonly accepted, but are regarded as acceptable because:

- (i) the foundations were ultimately investigated more thoroughly than is common, removing much of the usual uncertainty about the actual strength and conditions;
- (ii) adopted shear strengths are regarded as being slightly conservative;
- (iii) the instrumentation discussed in Section 7 is sufficiently comprehensive to allow dam performance to be monitored closely, both during and after construction;

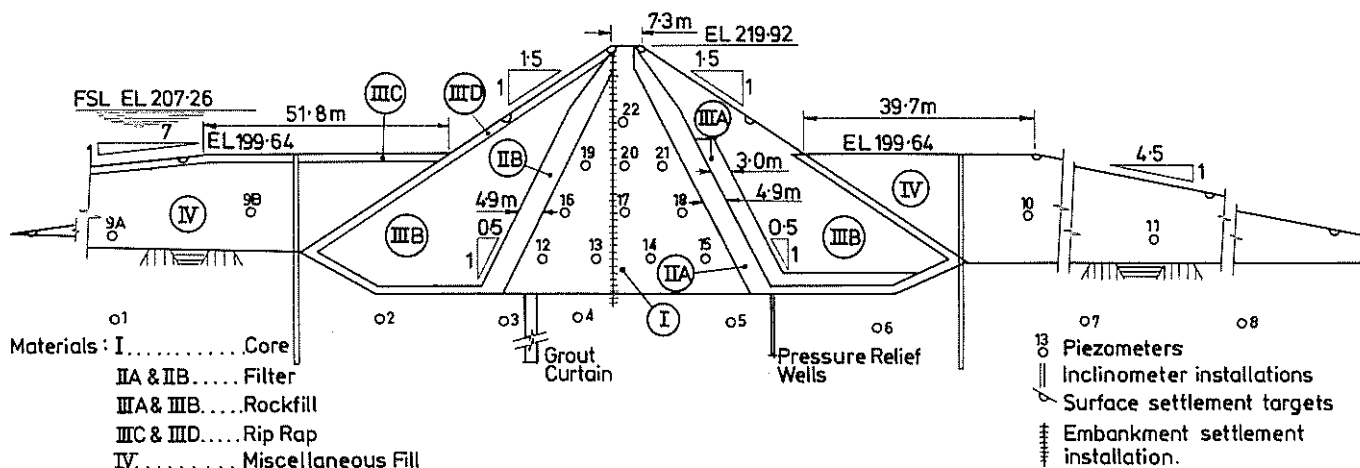


Fig. 4 Typical cross-section and instrument layout

(iv) large capacity outlets will allow rapid lowering of the reservoir if desired.

7 LAYOUT AND PERFORMANCE OF INSTRUMENTS IN DAM

Compared with the instrumentation provided for other Commission dams of a comparable size, that for Maroon is unusually complete, and it played an important role during the design and construction of the dam. Fig. 4 shows the typical location of instruments.

The importance of pore pressures to dam stability has been mentioned. To measure them, 55 hydraulic and 10 electric piezometers were installed, 17 in the core, with most of the remainder in the foundations. The piezometers in the foundations are intended to measure pore pressure on the slip planes. They were installed in NX diamond drill holes which had been cored to locate the slip planes, or more usually the seams containing them. Each piezometer tip is installed in a sand envelope occupying about 300 mm of the hole, not only intersecting the slip surface and all or part of the clay seam, but often extending into the adjoining fissured and therefore better drained shale. The pressure read is therefore probably that in the adjacent material. The assumption made that the pressure on the slip surface is the same is reasonable in view of the small thickness of the clay seam and of the slow rate of loading and possible movement, but it is not proved. Some uncertainty therefore exists in a factor which has considerable influence on stability. In operation, the hydraulic piezometers have generally performed reliably despite some de-airing problems, but electric piezometers were generally less satisfactory.

Calculated factors of safety are subject to error through uncertainty in assumed or measured pore pressures, soil properties, and the method of calculation. Because the calculated factor of safety is low and its accuracy uncertain, inclinometer installations were provided as a further independent check on dam performance. Within each of the six casings extending into the foundations below the expected limit of movement, regular readings are taken at 0.5 m intervals with an inclinometer s

supplied by Soil Instruments Ltd., allowing horizontal movements to be found at all levels in the dam and foundation, and excessive movements to be detected. The embankment analysis did not provide information about expected movements, but the ability to measure movement along slip surfaces or detect any abnormal extent or rate of movement at any point is useful, and the absence of any such abnormalities would be reassuring.

Inclinometer performance has been disappointing, failures in the equipment causing the loss of most readings during the important period late in the construction and for some time after completion, in spite of co-operation by the manufacturer in attempting to rectify the problems. At other times the instrument served its purpose in indicating movements with sufficient accuracy.

Electric settlement gauges installed at 3 locations have functioned satisfactorily, allowing settlements to be measured at 1½ m vertical intervals and consolidation calculated. Surface settlement points at 47 locations on the finished embankment allow surface movements, both vertical and horizontal, to be determined periodically by accurate surveying, and supplement the inclinometers as a means of detecting dam movement. Two points were moved by accident during construction, but otherwise the only limitation on performance is the accuracy of the surveys.

Finally, provision exists for measuring leakage into the drilled foundation drains by piping flows through 6 pipe outlets to V-notch weirs at the downstream face.

8 INSTRUMENT RESULTS AND THEIR APPLICATION

In Section 6 the importance of pore pressure to dam stability was emphasised, and piezometers have provided information for checking stability during construction and since completion. Regular readings during construction provided the current pressures, and from the response to superimposed load end of construction values were predicted, a simplified pattern of pore pressures adopted, and stability calculated. Table 2 shows predicted values for the completed embankment for some core and foundation piezometers, and actual measured values since completion in June, 1973.

TABLE II
PREDICTED AND ACTUAL PIEZOMETRIC LEVELS

Piezometer Number	Piezometric Levels for Completed Embankment (m)					
	Predicted			Actual		
	Aug. '72	Oct. '72	Feb. '73	Jun. '73	Nov. '73	Oct. '74
1(2 similar)	178.0	178.5	180.5	176.0	181.5	192.5
3	179.0	178.0	177.0	176.0	178.5	187.5
4	177.5	178.0	175.5	176.0	176.5	184.0
5	180.5	177.0	174.0	174.0	175.5	176.5
7(6 and 8) (similar)	175.0	174.5	174.0	173.0	173.5	174.5
12	218.5	215.5	214.0	215.0	214.5	215.0
13(14 similar)	241.0	239.5	227.0	223.5	223.0	222.0
15	215.5	212.0	210.5	210.5	209.5	206.0
16(17 similar)	226.0	217.5	213.0	214.0	212.0	213.0
18	214.5	201.5	204.0	204.5	204.5	203.5
19	-	211.0	201.0	200.5	201.0	205.0
20(21 similar)	-	218.0	208.0	206.0	208.5	210.0
22	-	215.5	213.0	214.0	215.0	215.0
Embankment Fill Level	197.0	199.5	213.5	220.0	220.0	220.0
Storage Level	-	-	179.0	179.0	187.0	204.0

The pore pressure response in the lower part of the embankment where moisture contents higher than the specified target of optimum + 1% are suspected indicated B values up to 0.75, with 0.3 to 0.45 common at higher levels. This resulted in core pore pressures higher than designed for, and though foundation pressures remained low, factor of safety based on early predictions of end of construction conditions fell to 1.02, but rose for later predictions, and actual measurements after completion gave a value of 1.23. Since completion, foundation pore pressures upstream have responded to rising reservoir level, but downstream there has been little change. Low permeability has resulted in very slow pore pressure dissipation and change towards steady seepage conditions in the core, so the factor of safety is unacceptably low for a full storage, and full supply level has been temporarily reduced six metres, which still allows sufficient storage for present irrigation needs.

When construction was 15 m below crest level concern over pore pressures caused us to consider limiting the rate of construction, but this was made unnecessary when shortage of funds slowed construction. The slow rate of pore pressure reduction experienced since completion suggests, however, that the failure to reach predicted pore pressures was due to over-estimating the effect on vertical pressures of adding relatively small volumes of fill at the upper levels, rather than to pore pressure dissipation.

The maximum horizontal displacements indicated by the inclinometers up to November, 1974, have been 30 mm in the foundations and 80 mm in the embankment. Surface settlement points have shown no movements greater than 50 mm, except for the two moved by accident, and most have been below 25 mm. Neither method of indicating horizontal displacement suggests any abnormal behaviour, and they confirm embankment stability.

Readings from electric settlement gauges show foundation settlements of up to 200 mm in the river

bed. Settlements in the embankment are following a normal pattern, with a maximum consolidation in the core of about 5% up to the present, which is about 1% below laboratory values for the same vertical pressure.

Foundation leakage varies with reservoir level, and totalled 6 l/s in November, 1974, with the reservoir about 6 m below designed full supply level.

9 CONCLUSIONS

A number of conclusions are suggested by experience at Maroon Dam.

(a) It is necessary at the start of site investigation to seek out and study any available reports on regional geology to help with an understanding of the immediate dam area and to draw attention to aspects of the geology which could influence the dam design and construction. A study of surface geology of the area within several kilometres of the dam can supplement, or if necessary replace, this study of published information.

(b) Careful field and laboratory investigations of inconspicuous foundation defects take a long time, and are obviously better made before design rather than during construction. It is also possible that the presence of such defects will increase the cost so greatly that an early awareness of their presence and significance may influence the decision on constructing a dam at that site. Adequate investigation of foundations should commence at the earliest possible time.

(c) When unusual foundation conditions exist the design method must be adapted to make proper allowance for them. In some circumstances considerable benefit is obtained by making allowance by some means for the effect on stability of three-dimensional action, even at a site where topography would not normally suggest such considerations to be worth while.

(d) Instrumentation can be used with advantage to allow less conservative assumptions than usual to be made, especially for construction pore pressures, as actual conditions can be monitored. Means must be available to overcome any problems detected, and this could involve changing construction rate or techniques, modifying design or material specifications, or control of reservoir filling. Though appropriate provisions may be included in contract documents, the benefits are greatest if the flexibility of day labour work is available. Even in more normal circumstances than those at Maroon, if end of construction stability seems likely to influence the dam design, or foundation pore pressures can greatly affect stability, a provisional plan to improve conditions, and suitable instrumentation to show if such improvements become necessary may be cheaper and more effective than over-conservative design.

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