Geological Aspects of the Design and Construction of the Reservoir Inlet and Draw-off Channels, Sugarloaf Reservoir Project

W. M. REGAN
Senior Geologist, Coffey & Partners Pty Ltd, Melbourne (formerly Project Geologist, MMBW)
J. R. L. READ
Superintending Geologist, Melbourne & Metropolitan Board of Works

SUMMARY Large excavations for the inlet and draw-off channels of Sugarloaf Reservoir were open cut in a siltstone and sandstone sequence containing bedding plane seams with very low shear strengths. The construction of permanently stable faces in rock containing these seams dictated the design of the cut slopes to suit the geological structure at each site. Careful geological monitoring enabled the design to be kept under continual review during all construction stages.

1 INTRODUCTION

The Sugarloaf Reservoir Project, located 35 km north-east of Melbourne, Victoria, includes an off-river storage reservoir filled by pumping from an adjacent river and from an existing aqueduct. After comprehensive treatment the water will be distributed in the Melbourne Metropolitan district.

The Project is sited in folded Silurian-age sedimentary rocks. The nearest major fault is the Yarra Fault located 3 km east of the Project. There is no evidence of fault movement in the area since Pleistocene time and the region is seismically quiet.

Two large excavations have been constructed within the reservoir basin for discharge of water into the reservoir from an inlet tunnel and draw-off to an outlet tunnel. This paper describes how different designs were developed to suit the geology at each excavation site and how close monitoring during the early construction stages allowed the design of the draw-off channel to be modified.

The inlet channel was excavated between February and June, 1977, and the draw-off channel was excavated between May and November, 1978.

2 PROJECT DESCRIPTION

The Project layout is shown on Figures 1 and 2. Water will be abstracted from both the Maroondah Aqueduct and the Yarra River at Yering Gorge and pumped along a 1230 m long, 2.6 m diameter tunnel through the inlet channel into the reservoir.

The water will be impounded by an 85 m high, 1000 m long main dam and two saddle dams to form a reservoir having a live storage of 95 000 ML, covering 455 Ha. The main dam and the 28 m high, 520 m long saddle dam No. 1 are concrete-decked zoned rock fill structures. Saddle dam No. 2, which is 6 m high and 170 m long, is an earth and rock fill structure.

Water will be drawn from the reservoir through the draw-off channel and a 400 m long, 2.6 m diameter tunnel under the left abutment of the main dam. It will then be pumped up to a treatment plant, treated and stored in a 200 ML capacity clear-water reservoir before being discharged through a 2.1 m diameter gravity main to Melbourne.

3 GEOLOGICAL SETTING

The Project area geology is illustrated in Figure 2.

Broad, flat-topped ridges up to 90 m high are separated by creeks with side-slopes ranging between 10° and 30°.

The rock types comprise siltstone interlaminated with and grading into fine grained sandstone. Distinct beds of medium grained sandstone ranging from 100 up to 1000 mm thickness also occur. These rocks are locally intruded by igneous dykes.

The main regional structure is a broad syncline which plunges gently north (Fig. 2). The rock mass also contains many minor folds either as local anticlines and synclines with axes roughly parallel to the main synclinal axis or as monoclinal folds, many of which have axes oblique to the main axis.

The principal defects in the rock mass are joints parallel to bedding with other joints mostly grouped in two orthogonal sets normal to bedding. Sheared and crushed seams formed by Devonian and Tertiary-age folding and faulting typically occur almost parallel to the bedding, although some are parallel to the other joint sets. These crushed seams are generally less than 20 mm thick and in fresh rock contain mixtures of rock fragments,

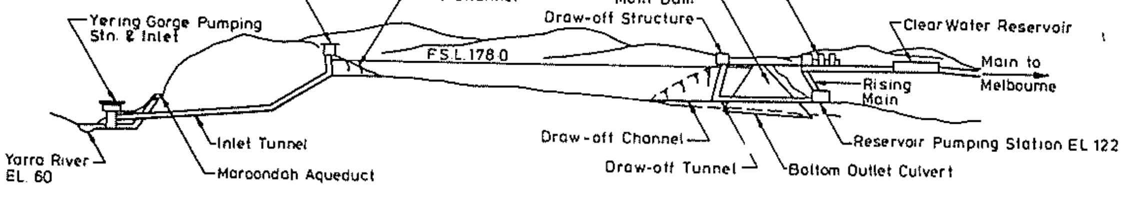

Figure 1 Schematic representation of project

silt and clay, and have soil properties (GP, GM and GC).

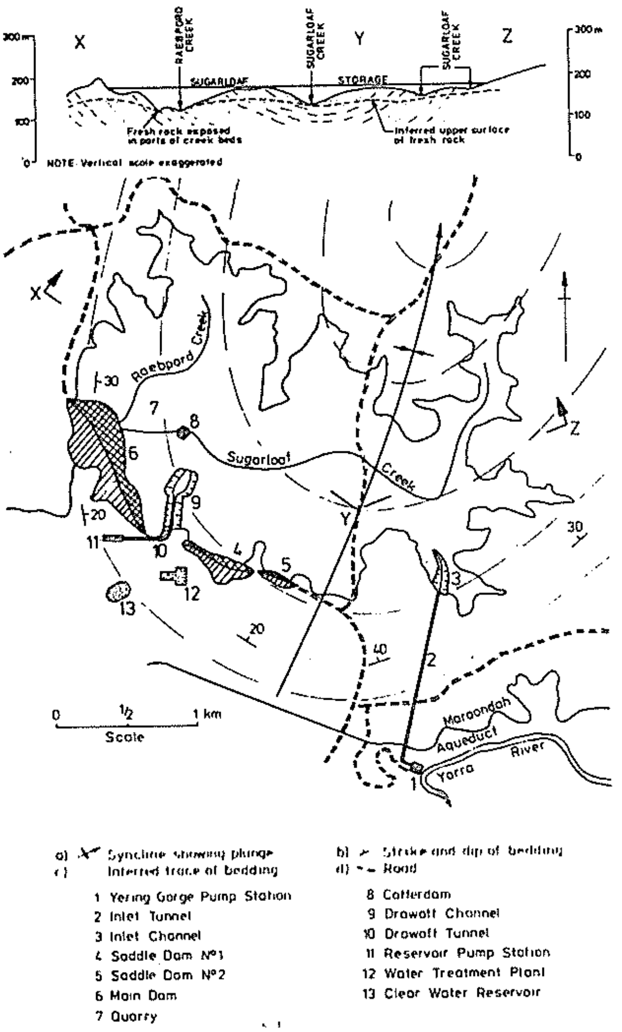

Figure 2 Project layout and geology

The rock mass has been subjected to the effects of weathering and erosion almost continuously since the end of the Devonian Period. As a consequence fresh rock is exposed only in the more deeply incised creeks and weathered zones of up to 70 m depth occur elsewhere. Above the fresh rock there is usually a gradational increase in the effect of weathering and, on the highest ridges, the joints and weathered rock substance are frequently limonite cemented.

Near-surface clay and gravelly infill seams occur where steeply dipping joints in the rock mass have opened due to mechanical weathering and downslope movement. In the weathered rock extremely weathered seams occur along the bedding and other joints forming 10 to 30 mm thick seams of silty clay (CL). Some of these seams disappear within a few metres of the surface but many of them, particularly the extremely weathered seams along the bedding, persist and with depth grade into the crushed seams which are nearly parallel to the bedding (Fig. 3).

3.1 Strength of Extremely Weathered Seams

Laboratory direct shear tests were carried out on 15 to 20 mm thick samples of extremely weathered seam material taken from bedding planes at depths of 2 m to 10 m. The shallower samples were taken where the presence of open joints and infill seams indicated that downslope movement had taken place. Attempts were made to orientate the samples with respect to the direction of inferred past downslope movement. Some were sheared through the seam material and others along the soil-rock contact.

The range of peak and residual shear strengths obtained are shown in Figure 4. One of the tests showed no peak strength, i.e. the sample was already at residual strength, indicating that the sample had

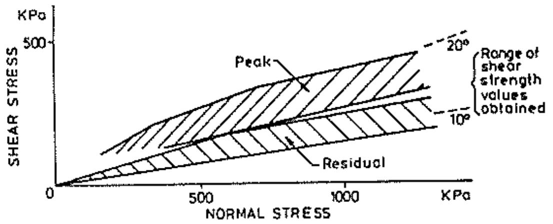

Figure 4 Shear strength of bedding plane seams

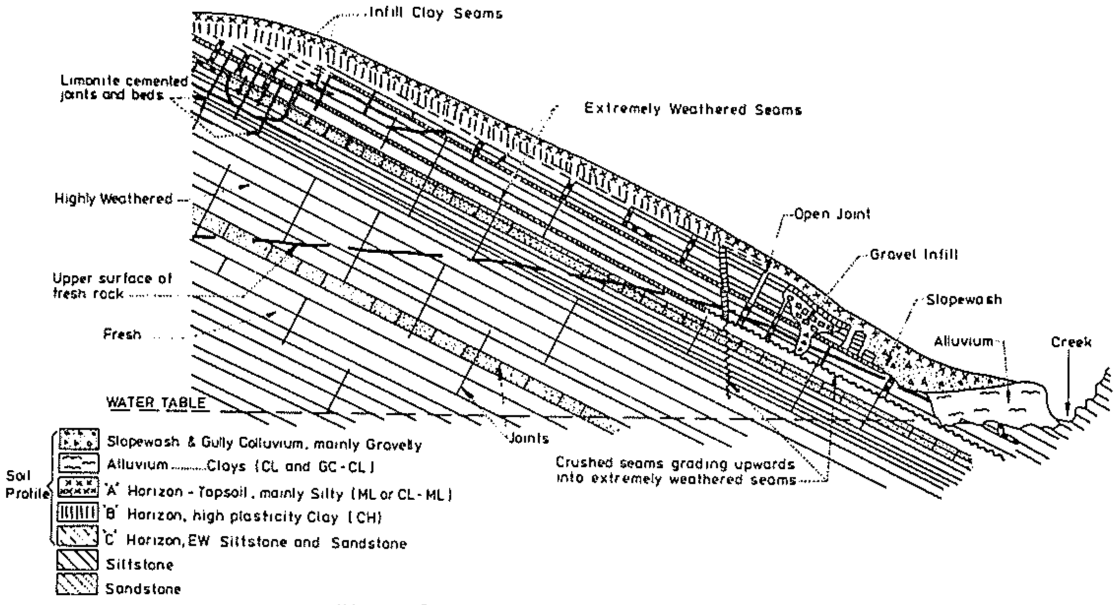

Figure 3 Soil and rock mass profile

been correctly orientated and sheared parallel to a pre-existing shear plane.

3.2 Groundwater

The rock mass permeabilities measured in the investigation boreholes were mostly in the low to very low ranges of less than 10 lugeons ($<10^{-6}$m/sec.). The groundwater table lies close to the bed of Sugarloaf Creek and the gradients beneath the higher ground are generally flat. Both the inlet and the draw-off channels are above the natural water table.

4 INLET CHANNEL

A control gate structure is inclined against and supported by the end wall of the inlet channel. This wall is 35 m high and 10 m wide at the channel invert (Fig. 5).

Site investigations, consisting of a bulldozer trench around the walls of the channel and a borehole in the deepest part of the channel, confirmed the site geology of interbedded siltstone and sandstone dipping 23° to 30° to the west. These rocks graded from highly weathered at the surface to slightly weathered in the deepest part of the channel and contained extremely weathered seams up to 50 mm thick along the bedding direction. Most joints were steeply dipping between 50° and 90°.

The channel was aligned with the end wall normal to the strike of bedding and an asymmetric shape was adopted (Fig. 5).

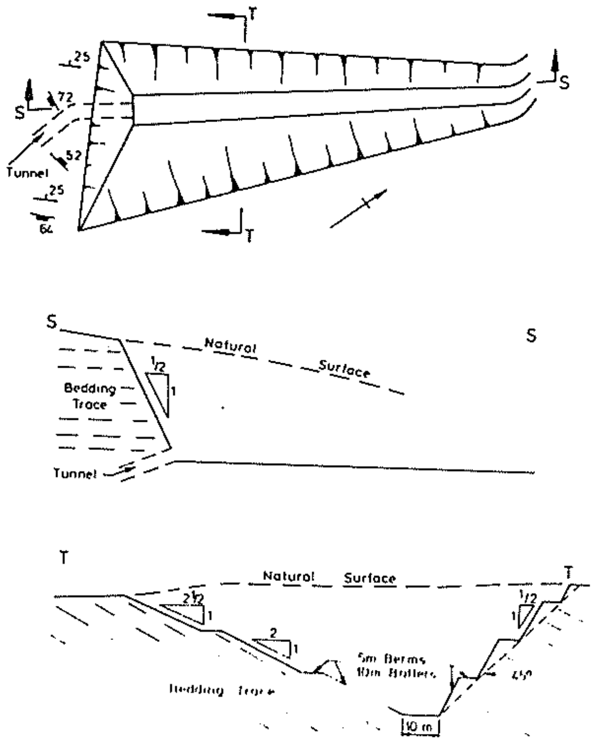

Figure 5 Inlet channel design details

4.1 End Wall

The end wall was cut at a slope of ½H : IV(63°) so that it would be stable with respect to the extremely weathered bedding plane seams and would not undercut the majority of the steeply dipping joints.

In addition the design allowed for the wall to be reinforced by 32 mm diameter mild steel deformed bars 5 m long, installed on a 2.5 m grid and inclined at a downward slope of 15° into the rock mass and grouted over their length.

The wall was first pre-split and then bulk excavation was carried out by ripping and dozing in 3 m vertical lifts. As each level was exposed the face was washed and geologically mapped to identify any seams or joints dipping out of the face which could require additional support or modification of the designed cut slope. Apart from some minor discontinuous seams and joints, which required the addition of local support, no such major defects were identified.

As the face was mapped the reinforcing bars were installed. Steel mesh was then pinned to the face and a 100 mm minimum thickness of a wet-mix shotcrete applied before the next lift was excavated. The shotcrete is intended to protect the weathered rock from fretting during the life of the reservoir. Drainage holes 2 m deep were drilled through the shotcrete to prevent hydrostatic forces from dislodging the shotcrete.

4.2 Dipslope Wall

The eastern wall of the channel was excavated parallel to the bedding planes (Fig. 5, Section TT). This was more economical than cutting a steeper face which would have required support to prevent sliding along the extremely weathered bedding plane seams. The design incorporated two 5 m wide berms and the face was geologically mapped as bulk excavation proceeded to ensure that the dip of the beds exposed was within the designed batter slopes of 2H : IV and 2½H : IV.

Except for a 25 m long meshed and shotcreted zone on the lowest batter, adjacent to the inlet structure, the dipslope wall was left unprotected.

4.3 Western Wall

For the western wall an overall slope of 45°, flatter than the majority of joints, was formed by cutting 10 m high batters inclined at ½H : IV, separated by 5 m wide berms (Fig. 5, Section TT).

The batters were pre-split and excavated with the end wall in 3 m vertical lifts. A 25 m long zone adjacent to the end wall was reinforced, meshed and shotcreted as for the end wall, but the remainder of the wall was left unprotected and unsupported. Geological mapping did not identify any areas requiring additional support.

5 DRAW-OFF CHANNEL

A control gate and trash rack structure incorporating shutters to permit water to be drawn off at any level is inclined against and supported by the end wall of the draw-off channel.

In the initial design the channel was to be located near the left abutment of the main dam in a gully to reduce the volume of excavation. However, this meant that one side wall would undercut a dipslope requiring either an excavation cut along the bedding, as in the inlet channel, or an extensive permanent support system. Neither option was economical so the possibility was examined of re-locating the channel along the axis of a nearby anticline which could result in a stable bedding orientation on both sides of the cutting.

After further investigation of the geometry of the anticline by bulldozer trenching and two boreholes, a 400 m long symmetrically shaped channel with a 65 m high end wall cut at ¼H : IV and side walls cut across bedding was adopted (Fig. 6). This arrangement gave a stable side wall situation with respect to the bedding planes. The joint pattern determined from the trenches showed that the principal joint sets were dipping at angles greater than 60^o or, in the case of two sets (H and G, Fig. 6), in the range of 40^o to 60^o. Accordingly, the side wall slopes were designed at 45^o overall. 10 m high, ½H : IV, batters separated by 5 m wide berms were originally selected but, after consideration of the height of cuts, 10 m high vertical batters separated by 10 m wide berms were adopted as a safety feature. Provision was made for fully grouted rock anchor support in case local areas of batter instability were encountered.

As the end wall undercut the gently plunging beds (Fig. 6, Section WW), additional measures were proposed to improve the stability of the face. The width of the channel was reduced within 75 m of the end wall by steepening the overall slope of the side wall from 45^o to 57^o in order to gain the maximum possible strength from arching effects and to reduce the size of any potential block or wedge failures. Presplitting followed by excavation in 3 m vertical lifts, patterned reinforcement with fully grouted 5 m long bars, and then meshing and shotcreting for protection were also proposed. In addition the reinforcing bars, mesh and shotcrete were extended to cover the steepened sections of the side walls.

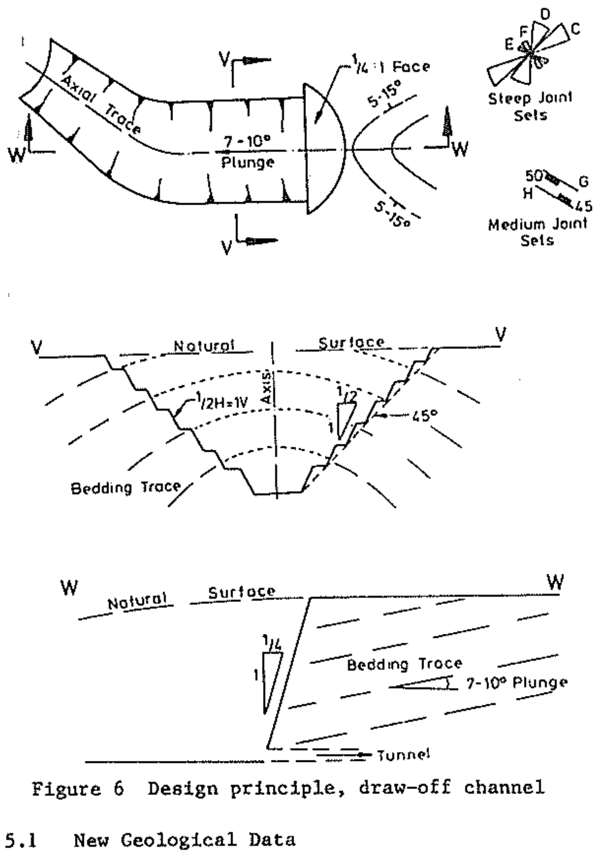

Figure 6 Design principle, draw-off channel

5.1 New Geological Data

During the construction of the main dam the Contractor exercised an option to widen and develop the northern end of the channel as a rockfill quarry. This action, together with the early completion of the draw-off tunnel to within 15 m of the channel portal, provided an opportunity to upgrade the geological model established during the investigation phase.

Geological mapping of the new exposures confirmed the overall geometry of the anticline and joint sets but revealed the moderately dipping joints of Set H (Fig. 6) to be more abundant and the steeper joint sets to be more continuous than envisaged. During quarry blasting it was also noted that slabs of rock were lifted. This new information was therefore used to re-assess the stability of the channel walls.

5.2 Side Wall Re-assessment

Two potential slip mechanisms were identified in the side walls of the channel. In the western wall blocks bounded by Sets H and D joints could fail and, in the eastern wall, blocks bounded by Sets G and D joints could fail (Fig. 7, Plan).

The potential size of the blocks depends on the overall side wall slope. Where the slope is 45^o only partial berm failure would be likely (Fig. 7, Section XX), but where the overall slope is 57^o complete slots could be formed for the full height of the channel (Fig. 7, Section YY).

Even though failures were possible in the steeper side wall sections adjacent to the end wall, it was decided not to flatten the overall slope in this area because :

- The probability of a failure occurring near the end wall was estimated to be only 10%.
- If necessary the potentially unstable blocks could be supported with rock anchors.
- Flattening the overall slope would increase the channel width thus reducing the end wall strength gained from arching effects and increasing the size of any potential block or wedge failure in the end wall.

5.3 End Wall Re-assessment

Two potential slip mechanisms were recognised in the end wall. They were (Fig. 7, Plan & Section ZZ).

- A thin, steep slab which could slide down along a Set G joint dipping 50^o, pulling away from a Set E vertical joint and striking laterally against a Set D vertical joint. This is shown as the GD wedge.
- A series of blocks resting on bedding planes, triangular in plan. The triangular shape assumes failure back to joints of the two persistent near-vertical Sets C and E (or D). The size of the uppermost (wedge-shaped) block (1) is controlled by the outcrop of a bedding plane extending from the cutting face back to point P on the level surface. The other blocks (2) to (5) are triangular prisms and their sizes are limited by the levels at which their basal surfaces daylight in the cutting face.

5.4 Support Measures Adopted

5.4.1 Side walls

Systematic temporary or permanent support was judged to be unnecessary for side walls with an overall slope of 45^o. However, the batters were geologically mapped as excavation proceeded. Any potentially unstable areas identified were either supported with fully grouted anchor bars or removed.

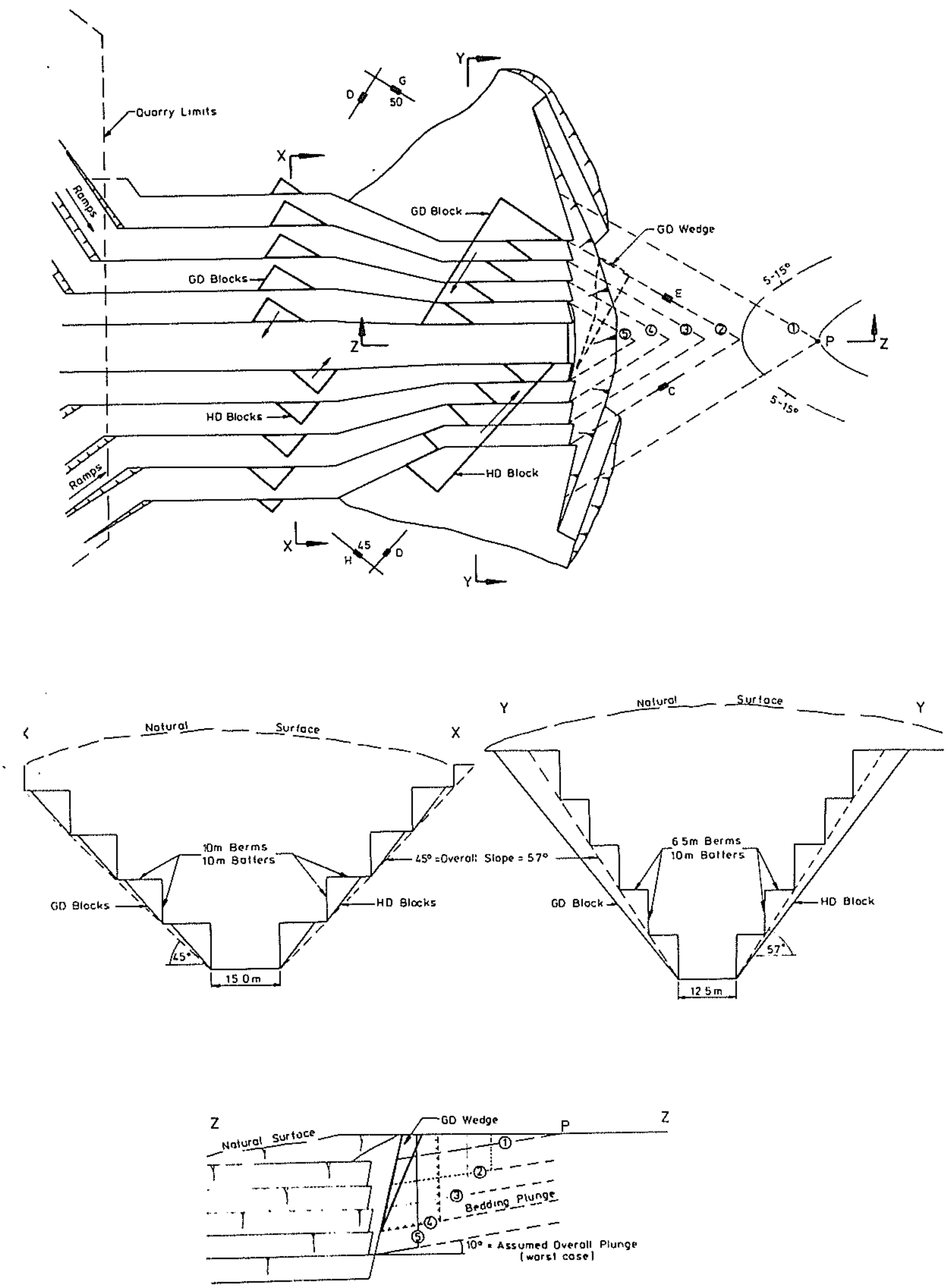

Figure 7 Draw-off channel as constructed

Within 40 m of the end wall the batters were pre-split in 10 m lifts but excavated in 3 m lifts. The walls were reinforced with 5 m long, fully grouted rock anchors installed at a downward slope of 10^{o} on a patterned 2.5 m grid. Two metre long drainage holes were then drilled on a 2.5 m grid staggered between the rock anchors before the faces were meshed and protected with a 100 mm minimum thickness of wet-mixed shotcrete. The drainage holes were protected by plastic tubes which were removed after the shotcrete had been applied.

5.4.2 End wall

A vertical row of 10 m long, fully grouted rock anchors were first installed at 1.5 m spacings 1 m behind the end wall to prevent lifting and loosening of the rock mass. The face was then pre-split in 10 m lifts prior to any burden blasting within 50 m of the face. The last 10 m next to the face was shot with zero burden and excavated in 3 m lifts. Next, the face was reinforced with 38 mm high tensile cold worked bars 14 m long, installed at a downward slope of 15^{o} on a 2.5 m horizontal by 3 m vertical grid and grouted over their length. Finally, 2 m long drainage holes, mesh and shotcrete were applied as for the adjacent side walls.

Both the end wall and the adjacent side walls were geologically mapped as they were excavated to check the geological model, and for any defects that might not be supported by the pattern of rock anchors. In the event no additional reinforcement was required in the end wall although some additional 5m anchors were required to support locally unstable areas in the side walls.

6 CONCLUSIONS

6.1

Large excavations can be successfully designed and constructed in interbedded sequences with unfavourable shear strength parameters provided :

- The geological model is carefully determined.
- An appropriate location and design recognising the limiting features of the geological model is adopted.
- All geological information obtained from construction monitoring is used to keep the geological model up to date so that the design can be kept under continual review.

6.2

The careful approach to excavation dictated by the use of vertical batters in large excavations is worth the effort. No rockfalls occurred during the construction period and a neat excavation with very little overbreak was obtained.

7 ACKNOWLEDGEMENT

The authors wish to thank the Melbourne and Metropolitan Board of Works for permission to publish this paper which is based on work executed on its behalf in connection with the design and construction of the Sugarloaf Reservoir Project.