# PROPOSED EXTENSIONS

# FOR

# NO. 2 TAILINGS DAM

# AT

# CLEVELAND MINE

# LUINA, TASMANIA

JULY 1978

PREPARED BY : ENGINEERING DEPARTMENT.

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# REPORT

TO

ABMINCO NL

ON

PROPOSED CONSTRUCTION WORK

FOR

FUTURE TAILINGS DISPOSAL

IN

NO. 2 AREA

AT

CLEVELAND MINE

LUINA, TASMANIA

JUNE 1978

77657

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## 1. INTRODUCTION

Tailings disposal at Abminco's Cleveland mine at Luina, Tasmania, is currently into the No. 2 area. To continue in this area certain additional construction work is required including a new embankment and a new decant system. This report describes these proposals with a view to obtaining Mines Department approval.

## 2. SCOPE OF THE WORK

The general site layout is shown on Figure 1. The No. 2 area is in a short steep sided gully with a limited catchment and as such has been close to ideal for tailings disposal since a minimal amount of embankment building has provided a large amount of storage volume. The tailings embankment is located at the southern end of the gully and has been raised by the upstream method with the coarser tailings from the gravity plant being pumped to cyclones on the dam crest. The much finer slimes tailings from the cassiterite float plant have been gravitated to the dam through a cutting between the No. 1 and No. 2 areas discharging on the western side of the pond about 80m upstream from the crest.

Water recovery has been through decant towers on four pipelines running beneath the tailings dam with outlets direct to the Whyte River immediately downstream of the embankment.

At the present time the embankment crest is at an average height of RL 342m and the pond is at RL 337.7m. These levels can be expected to rise by about 3m over a twelve month period. The cutting between the No. 1 and No. 2 areas has a floor level between RL 338.2 - 338.8m, and this therefore sets a limit in the near future on the life of the area in its current configuration. In order to continue to use the No. 2 area an embankment would be required, initially blocking off just the cutting itself, and later extending between the higher ground on either side of the cutting.

The ultimate level to which the tailings can be taken is limited by the small hill on the western side of the area and a final pond level of RL 356m would seem to be reasonable. On the basis of past performance, the main tailings embankment would need to be taken to RL 360m to keep such a pond far enough back from the crest and it is proposed that the new embankment at the northern end of the area should also be ultimately taken to this level.

To help spread expenditure it is proposed to extend the life of the area in two stages. Stage 1 would have embankment crest levels at RL 354m and a pond level at RL 350m and Stage 2 would have crest levels at RL 360m and a final pond level at RL 356m. Stage 1 would provide additional storage volume for approximately 1,400,000 tonne of tailings and Stage 2 for a further 900,000 tonne.

The additional construction works that are required are as follows:-

- new decant tower and pipelines (Stage 1),
- embankment across the cutting between the No. 1 and No. 2 areas - hereafter referred to as the secondary embankment,
- sealing the existing decant lines through the main tailings embankment (Stage 1),
- rock-fill zone against the downstream face of the main tailings embankment.

Each of these items will be described in turn.

## 3. NEW DECANT TOWER

The new decant tower will be located at the upstream toe of the secondary embankment, as shown in Figure 2. Twin 450mm diameter pipelines will run underneath the embankment with the discharge being ultimately channelled to the Whyte River.

The tower and the pipelines have been designed by Enpro Services Pty. Ltd. from Melbourne and the details are shown on Figure 3.

The criteria used for the design were as follows:-

- Tailings and mine water inflow, 300 kl/hr.

- Rainfall inflow at a peak rate of 1300 kl/hr and totalling 55 Ml over a 4 day period. These figures represent a 1 in 100 year event on the basis of rainfall records at Luina and data from the "First Report of the Stormwater Standards Committee of the Institution of Engineers, Australia" for the closest station to the mine, namely Queenstown. The total catchment area is approximately 180,000 $m^2$ of which about 50,000 $m^2$ is the tailings pond itself. The rainfall was estimated at 375mm over the 4 days. It was assumed that the first 25mm falling on the catchment above the dam would contribute no run-off and that a coefficient of 0.7 would apply thereafter. A co-efficient of 1.0 was naturally used for the actual tailings pond.

- Wind loading of 1.22 kPa over the full tower height.

The final design consists of a 1.7m x 1.7m reinforced concrete tower on a 3.5m x 3.5m base. The base will be founded on highly to moderately weathered rock and will impose a maximum edge bearing pressure of 250 kPa. Timber weir boards will be employed and access to the tower will be by a floating walkway. The tower will not be stage constructed but completed to its full height (RL 357m) as part of the Stage 1 work.

Twin reinforced concrete pipelines will run from the base of the tower. The pipes will be class Z with rubber ring spigot and socket joints. They will be laid in a trench to a fall of 1 in 200 and fully encased in concrete.

Under normal operating conditions there will be approximately 90mm head over the weir boards and the pipes will be flowing at only 15% of their full capacity. With the maximum flood inflow rate added to the operating inflow the head over the weir boards would increase temporarily to of the order of 350mm. Both pipes would be running almost full under these conditions. Even if one pipe was inoperative for any reason the remaining pipe would cope with the flow once the level had backed up about 2m in the tower.

## 4. SECONDARY EMBANKMENT

A preliminary layout and cross-sections for the secondary embankment are shown on Figure 2. Investigation drilling of the site and testing of borrow materials are required before detailed design can be completed.

The site geology is shown on the figure and it can be seen that the embankment is located over volcanics and serpentinite. The serpentinite visible in the cutting between the No. 1 and No. 2 areas appears to be closely jointed but tight and varying from highly to moderately weathered. Above the cutting there is evidence that both rock types are overlain by 1-2m of gravelly clay, presumably the product of in situ weathering.

It is expected that such gravelly clays would be suitable for embankment construction but since the quantity from any nearby borrow area is likely to be limited a zoned cross-section has been proposed. This assumes that gradually less and less weathered material would be recovered from the borrow area which could be used in progressively more free draining zones towards the downstream shoulder. It may even be more economical to use waste rock from the mine or crushed reject from the heavy media plant in the downstream shoulder.

Notwithstanding the above, it is envisaged that the Stage 1 embankment will consist mainly of material classified as gravelly clay. A cut-off trench will be incorporated in the foundations and the fill will be compacted in layers to normal embankment standards. The need for seepage control in the foundation rock will be investigated during site drilling.

The final embankment will be designed for a minimum factor of safety of 1.3 for both Stages 1 and 2 against a slip circle or wedge type failure of the downstream face under steady state conditions. The face slopes shown on Figure 2 are those typically found to be suitable for the materials envisaged but the analyses will reveal whether or not they need to be altered. There will be no provision for drawdown of the water in the pond and slimes will gradually accumulate against the upstream face. However, the upstream face will be designed for a minimum factor of safety of 1.00 under rapid drawdown conditions to guard against an emergency situation.

A study of the seismic risk at the site has been undertaken and it has been shown to be low. A ground acceleration of 0.05g has been estimated to be representative of a 1 in 160 year event and the stability of the downstream face will also be checked under these circumstances. A factor of safety of 1.1 will be required.

The maximum operating water level at each stage will allow for a free board of 4m. From the data given in section 3 above, the design flood would raise the pond level by 1.1m, should the decant system be inoperative at the time for any reason, but this would still leave a more than adequate margin against overtopping.

## 5. SEALING EXISTING DECANT LINES

Once the new decant system is commissioned the four existing lines through the tailings embankment will be sealed off.

The proposed method for sealing off is indicated diagrammatically on Figure 5 and is based on pumping a sand/cement grout up the line from the outlet end. Since full reservoir head must be applied to force the grout up the line, the injection must be done through a packer located well up the pipe to avoid bursting the rubber ring joints and damaging the embankment at the toe. Once injection through the packer is completed, low pressure grout can be used to backfill the line downstream of the packer.

## 6. ROCK-FILL ZONE AGAINST MAIN TAILINGS EMBANKMENT

Earlier detailed investigations and analysis have indicated the need for a rock-fill zone against the downstream face of the main tailings embankment if minimum safety factors of 1.3 and 1.1 are to be maintained under static and earthquake loads respectively at the future crest levels envisaged. Information on the phreatic surface within the tailings bank is available from the monitoring of a series of standpipes installed in 1974, and strength data on the tailings were obtained from in situ and laboratory testing.

The proposed layout for the rock-fill zone is shown on Figure 4. The tailings dam has been raised behind a starter dam constructed of run-of-mine rock waste. Crushed rock reject from the heavy media plant has been spread downstream of the starter dam and to the level of the starter dam crest to improve access but without prior stripping of the area. The current proposals will involve:-

Stage 1 . Excavation of the existing crushed rock, downstream of the starter dam.

. Excavation of the underlying clay soils to rock and cleaning of the exposed foundation.

. Placement of a 1m wide sand/gravel filter layer against the downstream face of the starter dam.

. Replacement and compaction of the crushed rock back to starter dam crest level.

The foregoing work will be undertaken on a hit-and-miss basis so that the starter dam will not be exposed over its full length at any one time.

. Continued placement of the filter layer and the rock-fill zone to reach RL 340m by the time the tailings embankment crest is at RL 354m.

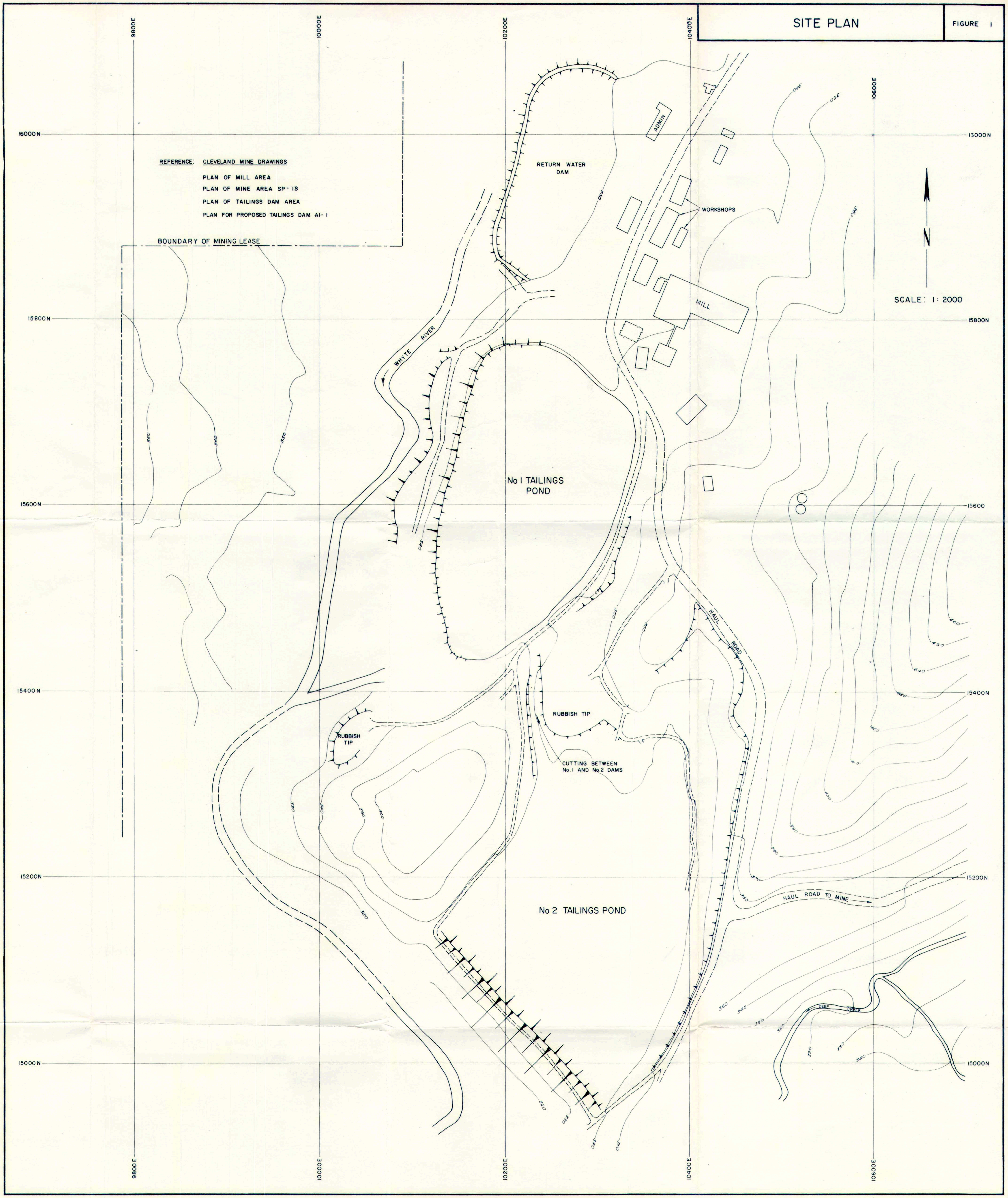
Stage 2 . Widening of the rock-fill zone by 10m. This will involve some diversion of the Whyte River as shown on Figure 5 and the placement of rip rap protection at the toe of the slope. The widened rock-fill zone will reach RL 340m by the time the tailings embankment is at RL 360m.

GOLDER ASSOCIATES PTY. LTD.
per:

I. K. Walker

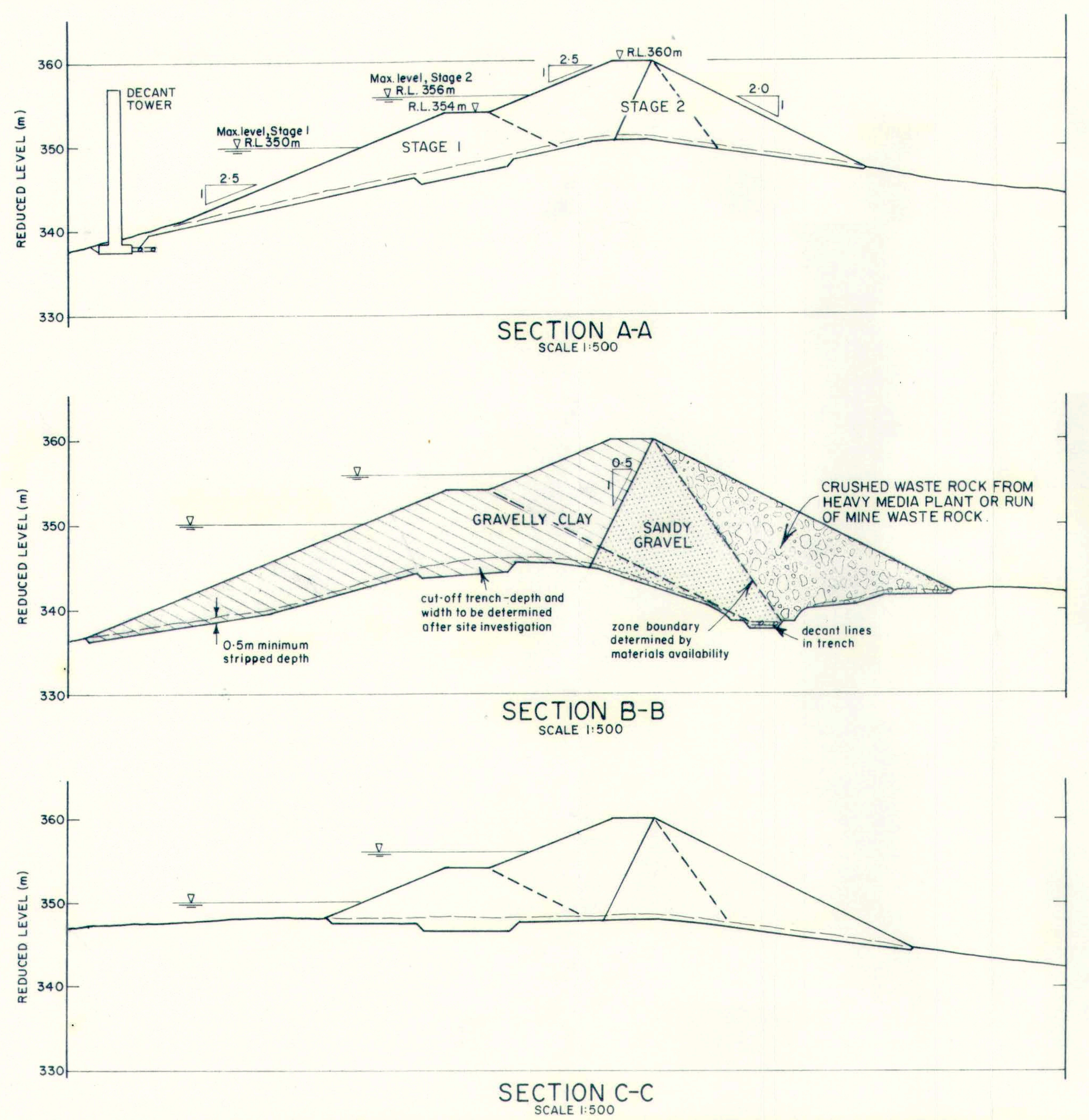
for M.P.A. Williams

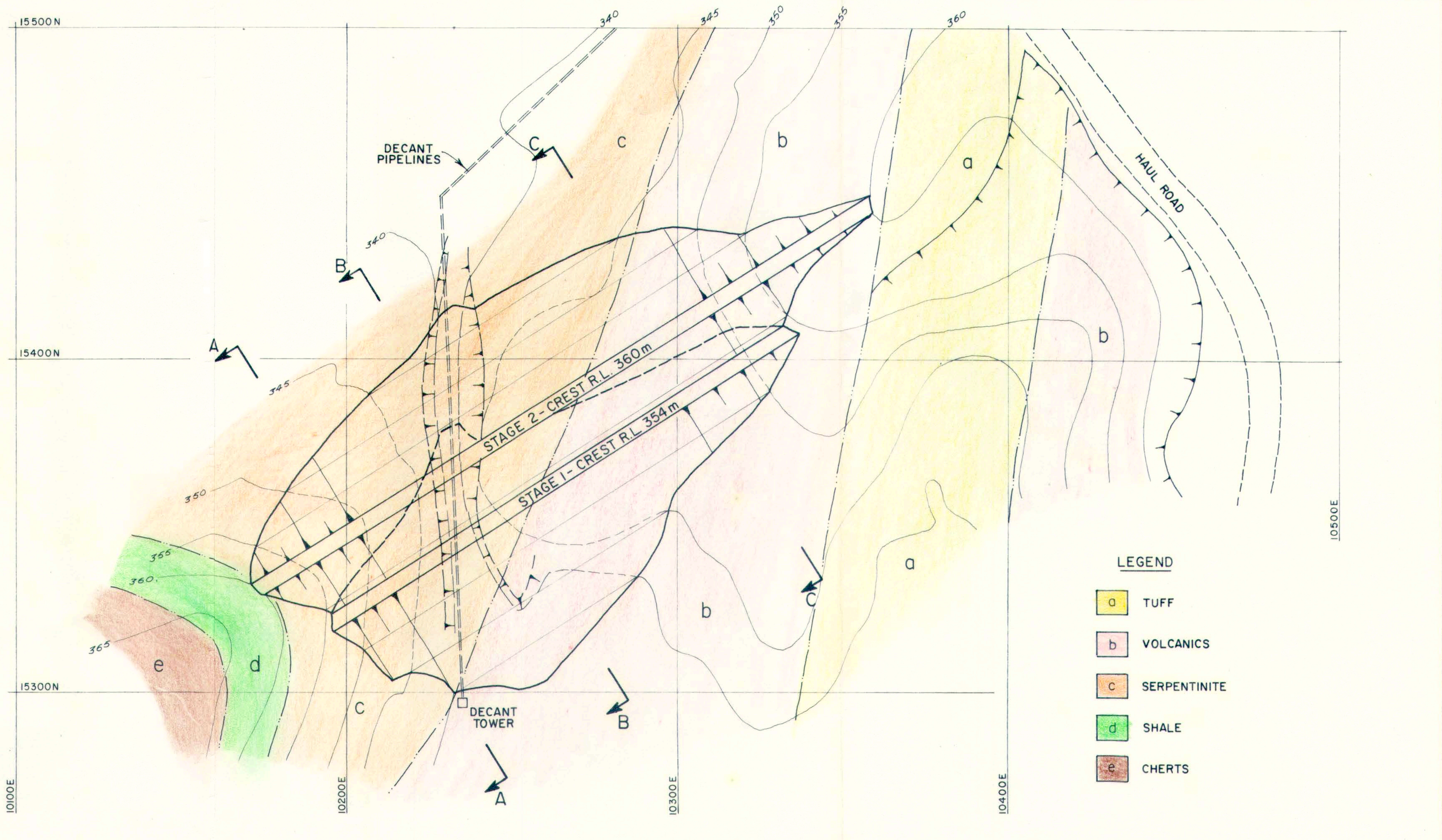
MPAW/blh


SITE PLAN
FIGURE 1
REFERENCE: CLEVELAND MINE DRAWINGS
PLAN OF MILL AREA
PLAN OF MINE AREA SP-15
PLAN OF TAILINGS DAM AREA
PLAN FOR PROPOSED TAILINGS DAM AI-1
BOUNDARY OF MINING LEASE
RETURN WATER DAM
ADMIN
WORKSHOPS
MILL
WHYTE RIVER
No 1 TAILINGS POND
HAUL ROAD
RUBBISH TIP
RUBBISH TIP
CUTTING BETWEEN No.1 AND No.2 DAMS
No 2 TAILINGS POND
HAUL ROAD TO MINE
DEEP CREEK
SCALE: 1:2000
N
9800E
10000E
10200E
10400E
10600E
16000N
15800N
15600N
15400N
15200N
15000N

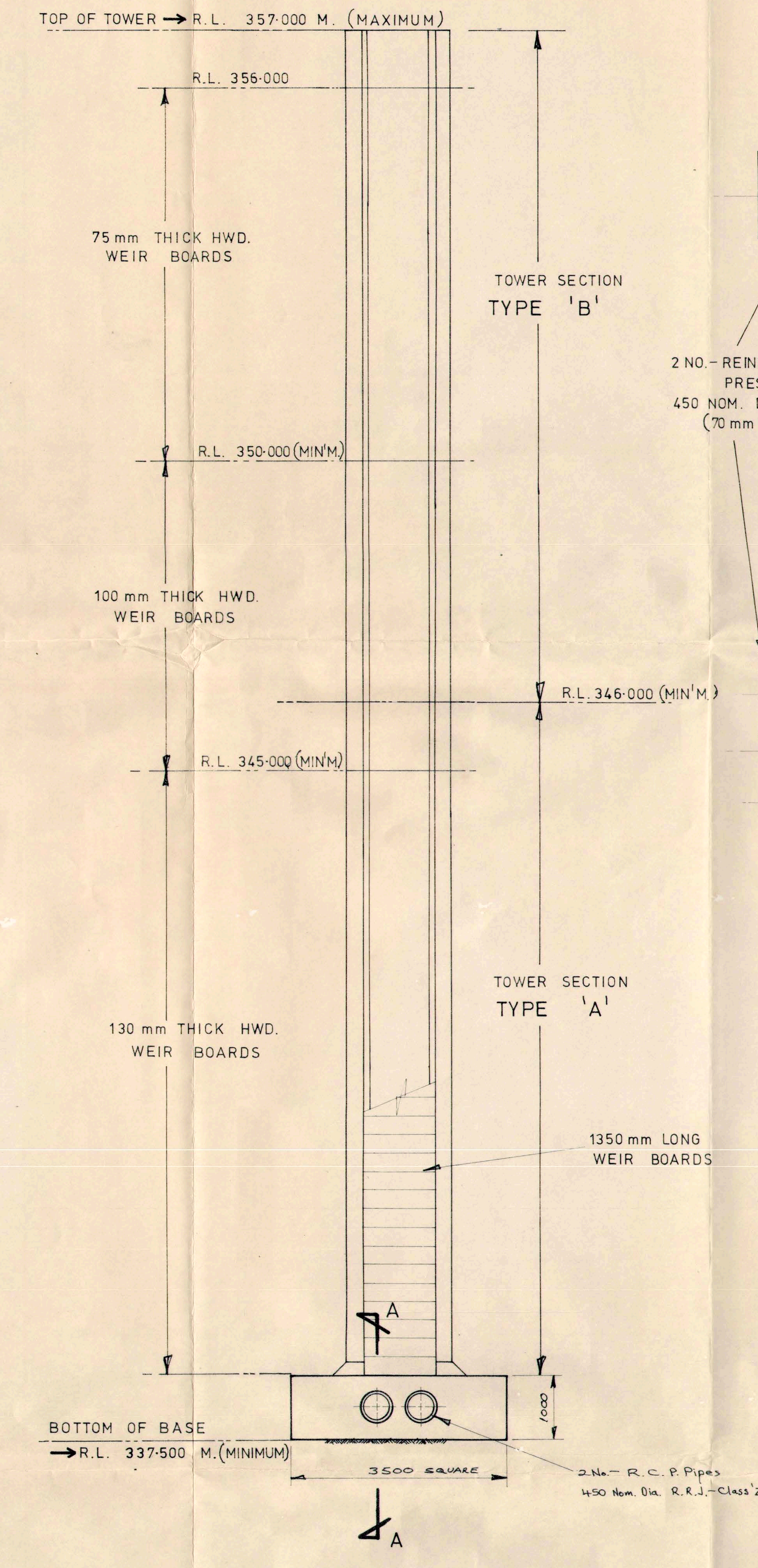
# SECONDARY EMBANKMENT

FIGURE 2

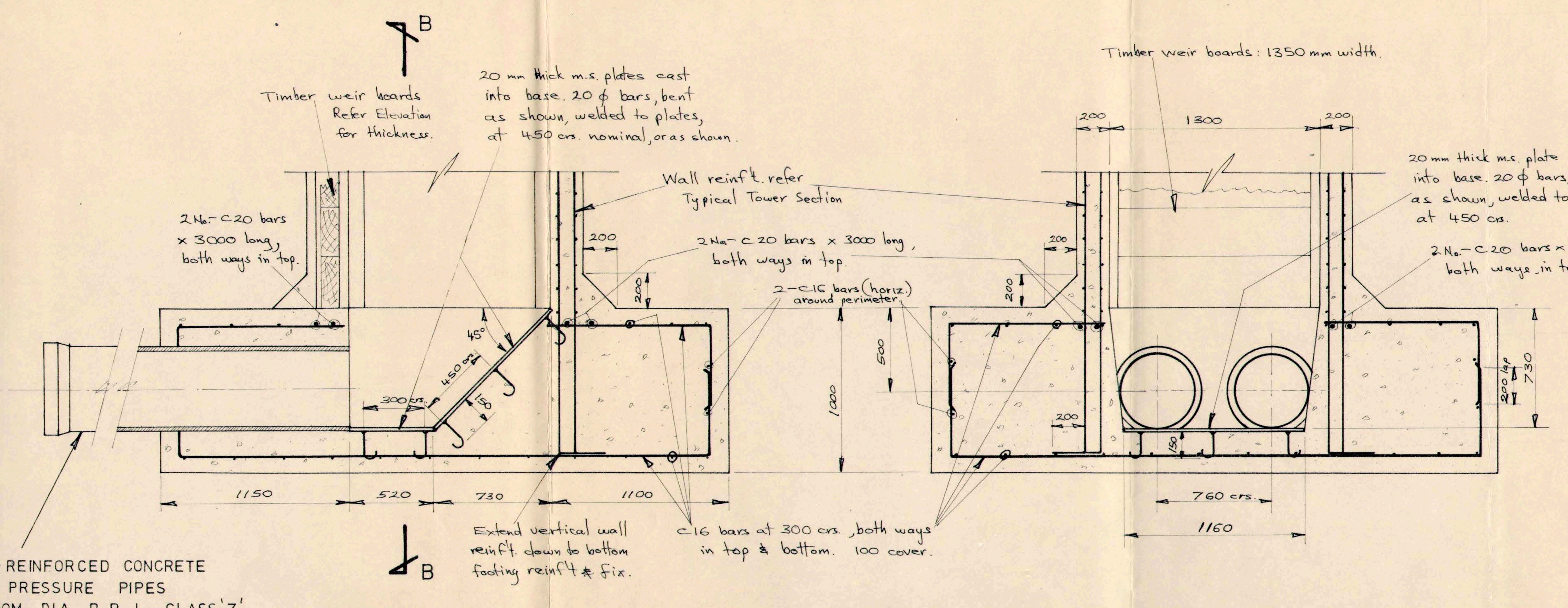



GOLDER ASSOCIATES

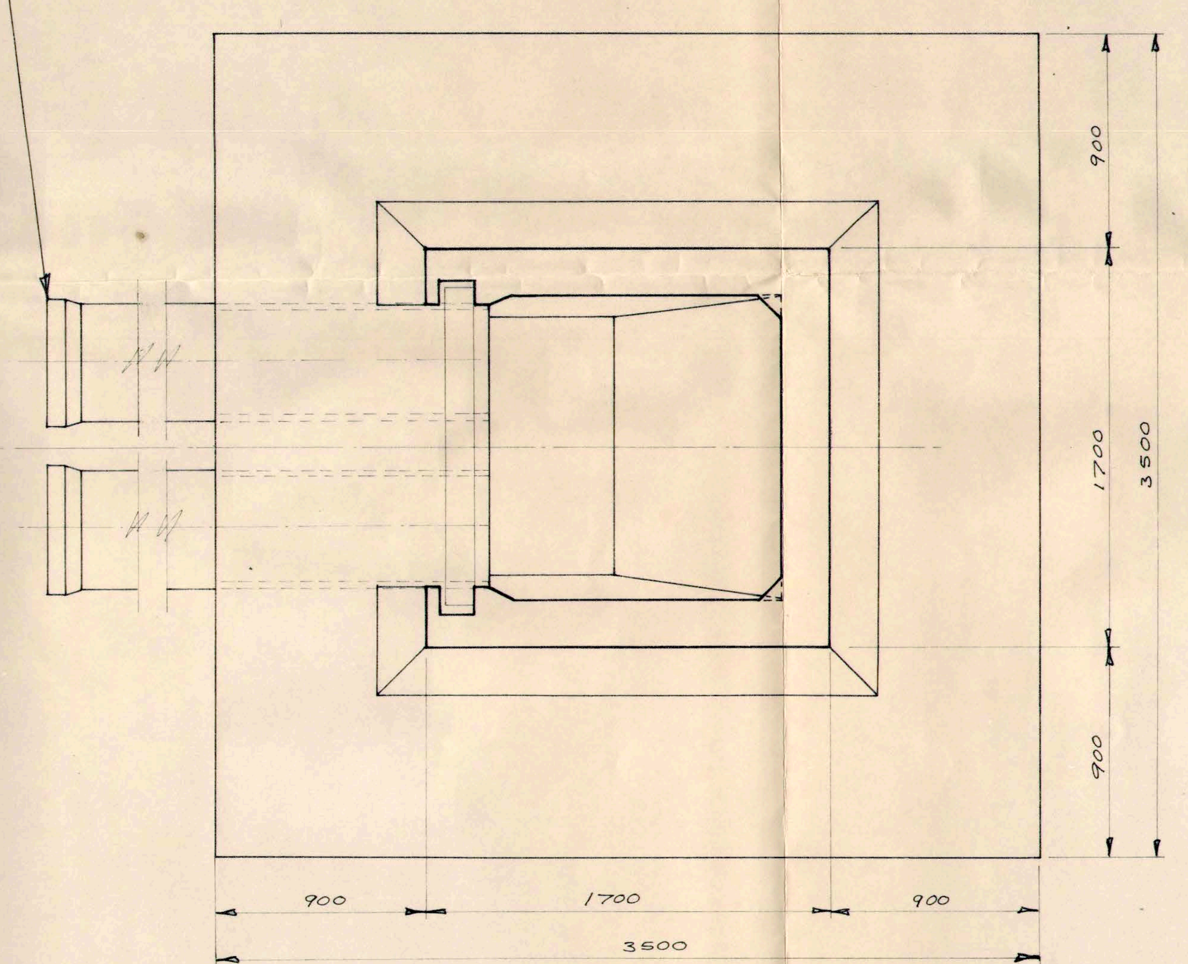



ELEVATION

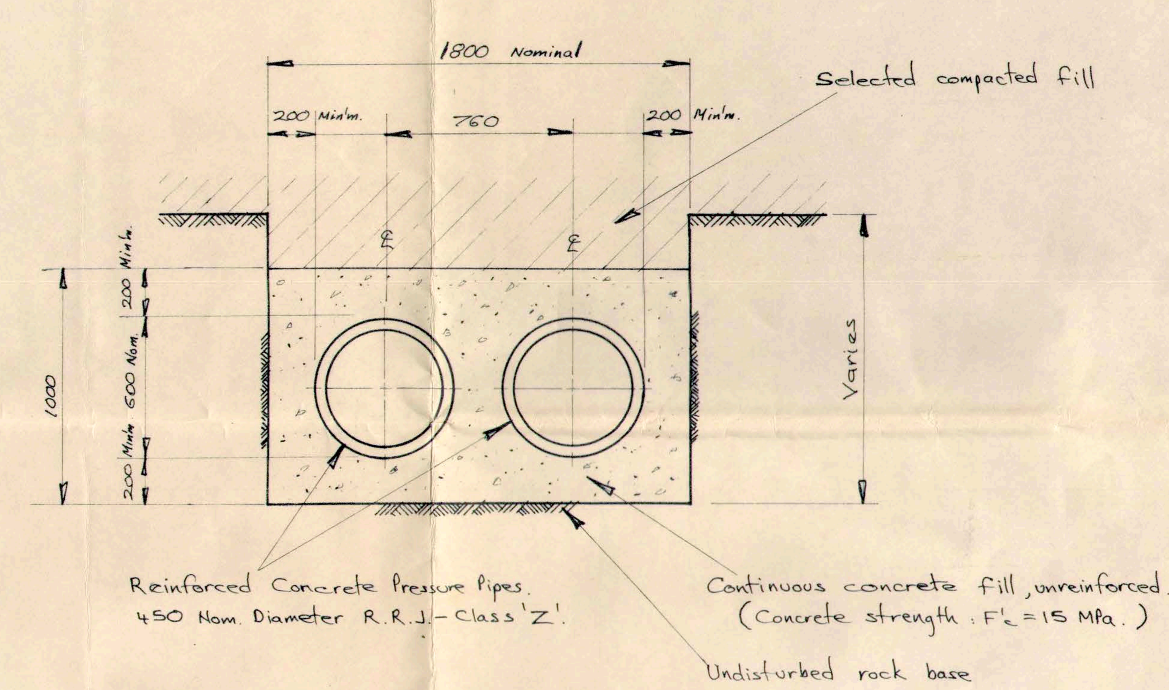



SECTION A—A

SECTION B—B



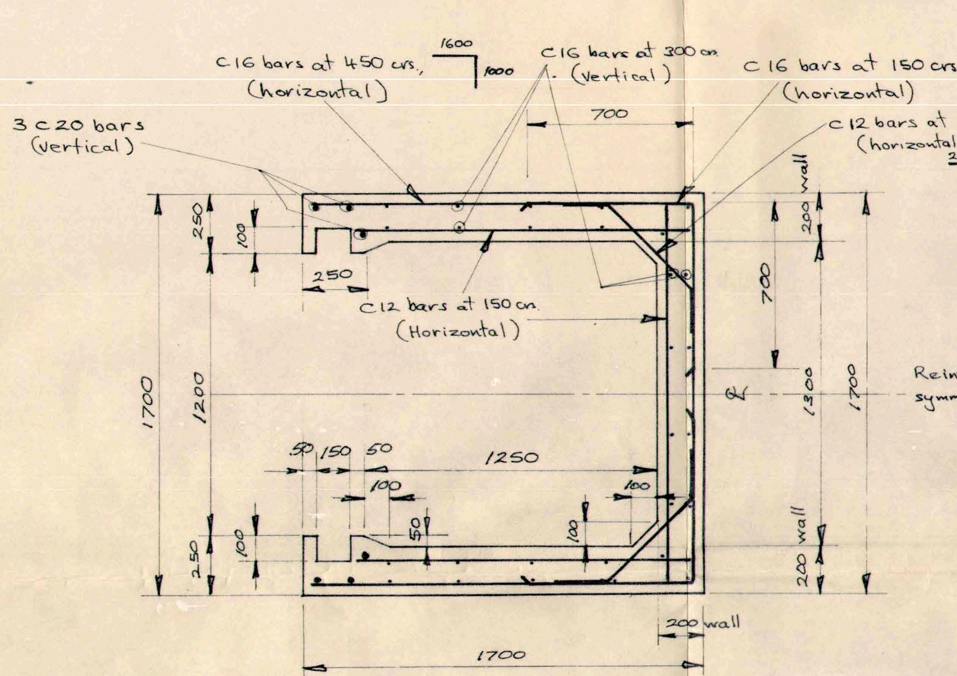

PLAN



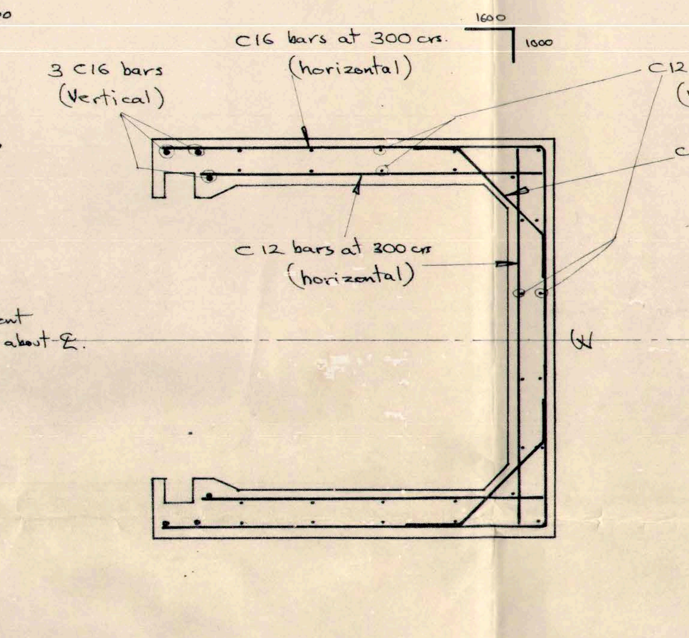

TYPICAL PIPE BEDDING

## GENERAL NOTES

1. ALL CONCRETE CONSTRUCTION AND MATERIALS SHALL CONFORM TO A.S. 1480 1974 – 'CONCRETE STRUCTURES CODE'.
2. ALL CONCRETE SHALL HAVE STANDARD 28 DAY COMPRESSIVE STRENGTH (F'c) OF: 20 MPa FOR PAD FOOTING
   25 MPa FOR TOWER WALLS
3. CONCRETE SHALL BE MOIST CURED FOR AT LEAST 7 DAYS AFTER COMPLETION OF CONCRETE PLACEMENT.
4. CONCRETE COVER (CLEAR) TO REINFORCEMENT SHALL BE:
   WALLS .......... 50 mm
   PAD FOOTING .... 75 mm (U.O.N.)
5. MAXIMUM CONCRETE SLUMP SHALL BE 75 mm ± 12 mm OR 100 mm WHEN A PUMP IS USED, UNLESS OTHERWISE SPECIFIED.
6. ALL REINFORCEMENT SHALL CONFORM TO THE RELEVANT AUSTRALIAN STANDARD AND SHALL BE:
   COLD TWISTED DEFORMED BARS – SHOWN: C16.
7. ALL REINFORCEMENT SHALL BE HELD RIGIDLY IN POSITION WITHIN THE SPECIFIED TOLERANCES AT ALL TIMES.
8. REINFORCEMENT SPLICES SHALL BE LAP SPLICES AS REQUIRED BY THE CURRENT CONCRETE CODE UNLESS NOTED ON DRAWINGS.
9. ALL FOOTINGS SHALL BE BASED ON FIRM UNDISTURBED MATERIAL WITH A SAFE BEARING CAPACITY OF AT LEAST 250 kPa AND SHALL BE BASED AT LEAST 750 mm BELOW FINISHED OR NATURAL GROUND LEVEL, WHICHEVER IS DEEPER.
10. ALL TIMBER SHALL CONFORM TO A.S. 081 1966 "ENGINEERING TIMBERS FROM E. AND S.E. AUSTRALIAN HARDWOODS", STANDARD GRADE, STRENGTH GROUP – 'S3' (MINIMUM).




TYPICAL TOWER SECTION — TYPE 'A'




TYPICAL TOWER SECTION — TYPE 'B'

| REF | BY | DATE | REVISION | APP'D | DATE |
|---|---|---|---|---|---|
| E | | | | | |
| D | | | | | |
| C | | | | | |
| B | | | | | |
| A | | | | | |

**Enpro Services**

Design, Contracting and Project Management Engineers



DESIGNED
DRAWN 4
CHECKED
APPROVED
DATE

CONCRETE DECANT TOWER — No. 2 TAILINGS DAM

SCALE 1:50; 1:200;
DRAWING No.

MAIN EMBANKMENT

FIGURE 4


GOLDER ASSOCIATES

# SEALING EXISTING DECANT LINES

FIGURE 5

PROCEDURE:

(A) (i) LOCATE PACKER BENEATH DAM CREST AND INFLATE

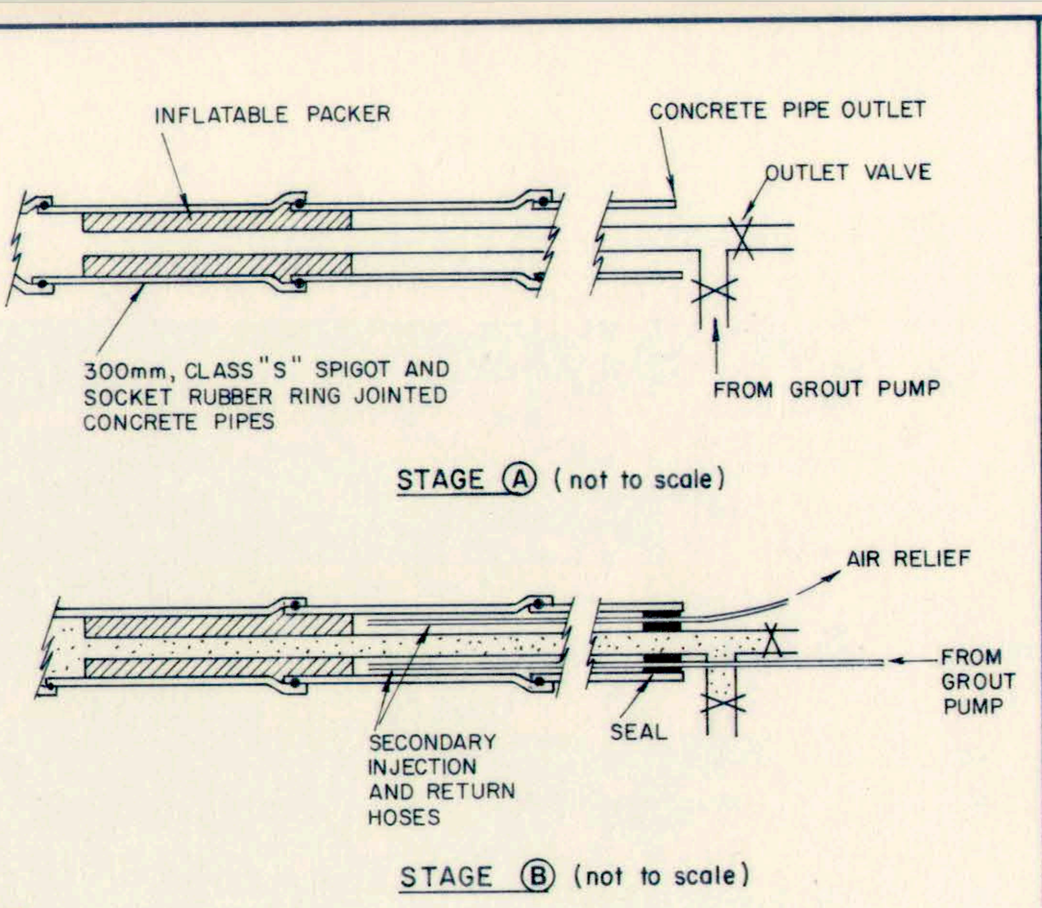
(ii) CLOSE OUTLET VALVE, OPEN GROUTING VALVE AND PUMP GROUT TO FILL 75 m OF PIPE

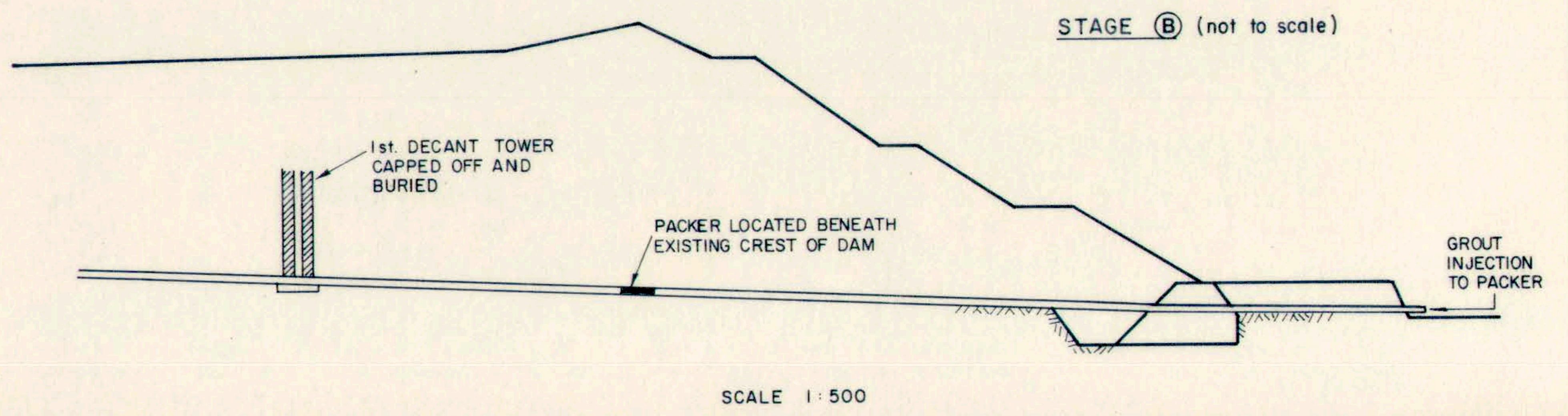
(iii) CLOSE GROUTING VALVE

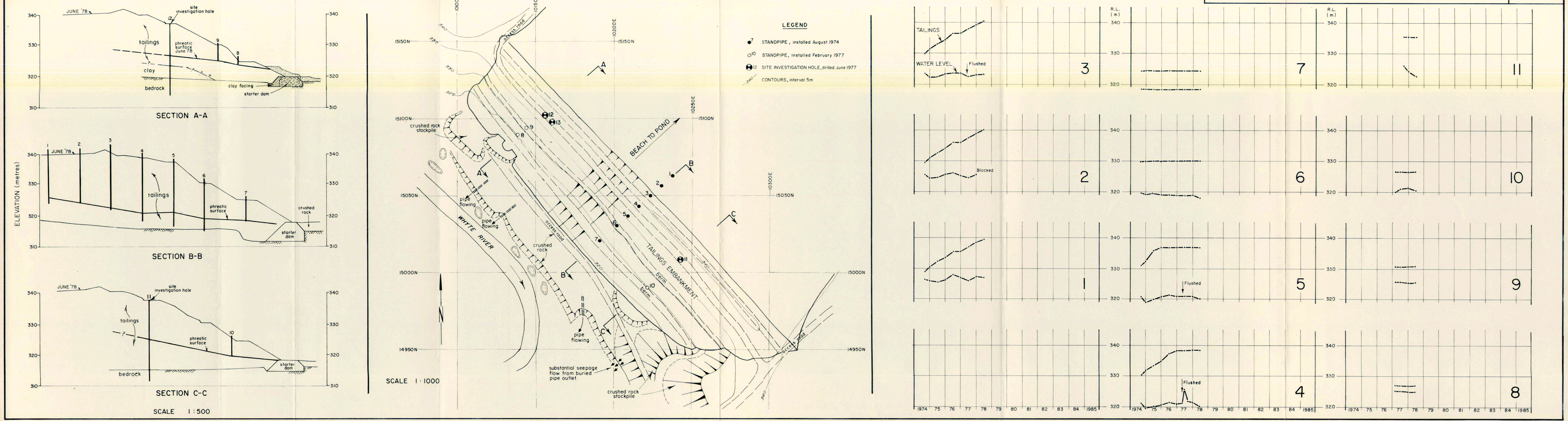
(B) (i) PLACE SECONDARY INJECTION AND RETURN HOSES TO DOWNSTREAM END OF PACKER AND SEAL PIPE AT OUTLET

(ii) INJECT GROUT UNTIL IT RETURNS THROUGH RETURN HOSE

(iii) CLOSE OFF BOTH HOSES






2 TAILINGS EMBANKMENT — PHREATIC SURFACE MONITORING

Fig. 1

# CALCULATIONS

# FOR

# DECANT TOWER

# NO. 2 TAILINGS DAM

PREPARED BY ENPRO SERVICES.
JUNE 1978

ABMINCO N.L. - Luina Tasmania.

Calculations for Decant Tower - No. 2 Tailings Dam. June 1978.

## CONCRETE DATA

$F_c'$ : 25 MPa

$F_s$ : 210 MPa

$K_{val}$ = 1·576 MPa, $a = 188 \times 10^{-6}$

## LOADING — HORIZONTAL

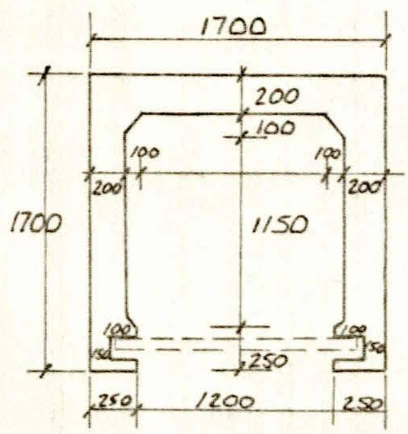
Water pressure = 1000 kg/m³ (Density)
Tailings = 1760 kg/m³
Assume angle of Repose = 15°
∴ Coeff. = 0·590

∴ Tailings pressure = 0·59 × 1760
= 1038 Kg/m³

∴ Critical pressure = $1038 \times \frac{9 \cdot 8}{1000}$
= 10·17 KN/m³

Max'm height of Tower = 22m.

∴ Consider 1m. wide strip at 21m. average height :
— Dam full (up to 22m depth max'm)
— Weir boards up to top of tower.
— Tower empty internally.




∴ Max'm. pressure on walls
= 10·17 × 21 = 213·57 KN/m³
= 214 KPa.

## SIDE WALLS

Assume concrete wing walls acting as propped Cantilevers:

$\therefore$ Pos. B. Mom. $= 214 \times (1{\cdot}15)^2 \times \frac{9}{128}$

$= 19{\cdot}90$ KN.m

$\therefore$ Neg. B. Mom. $= 214 \times (1{\cdot}15)^2 \times \frac{1}{8}$

$= 35{\cdot}38$ KN.m

For 200 concrete wall:

Cover = 50 mm, $\therefore$ depth, $d = 150$ mm.

$M_{RC} = 1{\cdot}576 \times (150)^2 \times 10^{-3} = 35{\cdot}46$ KN·m ——— OK.

POS. $A_s$ req'd $= \frac{19{\cdot}9 \times 10^6}{188 \times 150} = 706$ mm²/m

use C12 bars at 150 crs.
Horizontal

Neg. $A_s$ req'd $= \frac{35{\cdot}4 \times 10^6}{188 \times 150} = 1255$ mm²/m

use C16 bars at 150 crs.
Horizontal

Consider 1m wide strip at 11·0m. average height:

Max'm press $= 10{\cdot}17 \times 11 = 111{\cdot}87$ KN/m²

$= 112$ Kpa

$\therefore$ Pos. $A_s$ req'd $= \frac{112}{214} \times 706 = 370$ mm²/m

use C12 bars at 300 crs.
Horizontal

$\therefore$ Neg $A_s$ req'd $= \frac{112}{214} \times 1255 = 657$ mm²/m

use C16 bars at 300 crs.
Horizontal

## END WALLS

Pos. B. Mom. $= 214 \times (1.2)^2 \times \frac{1}{24}$

$= 12.84$ kN.m

Neg. B. Mom. $= 214 \times (1.2)^2 \times \frac{1}{12}$

$= 25.68$ kN.m

Pos. $A_s$ req'd $= \frac{12.84 \times 10^6}{188 \times 150}$

$= 455.3$ mm²/m

use C12 bars at 150 crs.
Horizontal

Neg $A_s$ req'd $= 911$ mm²/m

use C16 bars at 150 crs.
Horizontal

By inspection, use same reinf't. & layout as for Side Walls.

## LOADING — VERTICAL

WIND LOAD: Wind velocity = 41 m/sec
Mean return period = 50 years

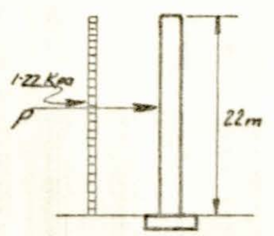
Terrain Catergory : 2

∴ Terrain Multiplier = 1·10

∴ Wind pressure $= (41 \times 1.1)^2 \times 0.6 \times 10^{-3}$

$= 1.22$ Kpa.

Critical case when dam empty, leaving full height of Tower exposed:




(Load/m $= 1.22 \times 1.7 = 2.07$ kN/m height of Tower)

Max'm. $P = 1.22 \times 22 \times 1.7$

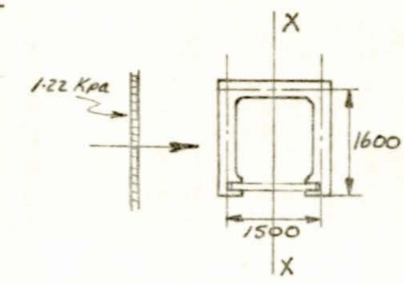
$= 45.63$ kN

. Max'm. B. Mom. about base $= 45.63 \times 11$

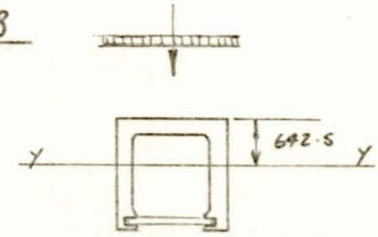
$= 502$ kN.m

TWO WIND LOAD CASES:

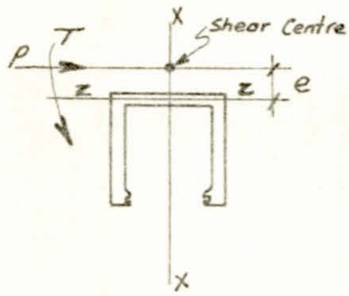
A




B




CASE A. :




Area = $2 \times 200 \times 1500 + 1700 \times 200$
$= 940 \times 10^3\ mm^2$

Distance of C. of G. from external face

$$= \frac{2 \times 200 \times 1500 \times 950 + 1700 \times 200 \times 100}{940 \times 10^3}$$

$= 642.5\ mm$

$$e = \frac{b^2 h^2 t}{4 I_{xx}}$$

$$I_{xx} = \frac{1700 \times (1700)^3}{12} - \frac{1500 \times (1300)^3}{12}$$

$= 421.4 \times 10^9\ mm^4$

$$\therefore e = \frac{(1600)^2 \times (1500)^2 \times 200}{4 \times 421.4 \times 10^9} = 683.5 mm$$

$\therefore$ Torsdon $T = 2.07 \times \left(\frac{1700}{2} + 683.5\right)$
$= 3.17$ KN.m/m height of wall.

$\therefore$ Max'm $T = 3.17 \times (22 - 1.65)$
$= 64.5$ KN.m.

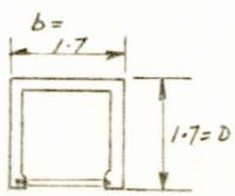
Equivalent shear;

$$V_1 = V_u + \frac{1.6\,T_U}{b}$$

$$= 1.8 \times 42.3 + \frac{1.6 \times 1.8 \times 60.5}{1.7}$$

$= 178.63$ KN.

Max'm shear
$= 1.22 \times 1.7 \times 20.45$
$= 42.3$ KN




$$v_u = \frac{178.63 \times 10^3}{16500 \times 200} = 0.54\ Mpa$$

$\rho = 0.009$

$\therefore \; v_c' = 0.85 \times 0.73 = 0.62$ Mpa, $> 0.54$ Mpa

$\therefore$ ———— o.k.

Equivolent moment;

$$M_1 = T_1 + M_u$$

$$\text{Where } T_1 = T_u \sqrt{\left(1 + \frac{2D}{b}\right)}$$

$$= 64.5 \sqrt{\left(1 + \frac{2 \times 1.7}{1.7}\right)}$$

$$= 11.72 \text{ KN.m.}$$

$$\therefore M = 11.72 + 502$$

$$= 613.72$$

SAY 614 kN.m.

$$Z_{xx} \text{ min'm.} = \frac{421.4 \times 10^9}{800}$$

$$= 526.75 \times 10^6 \text{ mm}^3$$

$$\text{Max'm. Comp. Stress} = \frac{614 \times 10^6}{526.75 \times 10^6} = 1.17 \text{ Mpa.}$$ ———— o.k.

$$\text{Tension} = \frac{614 \times 10^3}{1500} = 409 \text{ kN}$$

$$A_{st} \text{ req'd} = \frac{409 \times 10^3}{188} = 2177 \text{ mm}^2$$

$$= \frac{2177}{1.7} = 1280 \text{ mm}^2/\text{m}$$

(Min'm — 11 No.) $\therefore$ use C16 bars at 300 crs, both faces Vertically

($\phi$ Area = 1334 mm²/m)

Check at 11.0m average height:

$$T_{max} = 2.97 \times 11 = 32.67 \text{ KN.m.}$$

$$M_1 = \frac{1.22 \times 1.7 \times (11)^2}{2} + \frac{32.67}{60.5} (106.95)$$

$$= 125.48 + 57.75$$

$$\therefore M = 183.2 \text{ KN.m.}$$

$$\therefore \text{Tension } A_{st} \text{ req'd} = \frac{183.2 \times 10^6}{188 \times 1500} = \frac{650}{1.7} = 382 \text{ mm}^2/\text{m}$$

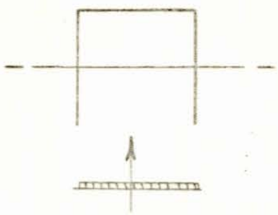
$\therefore$ USE C12 bars at 300 crs, both faces Vertically

CASE B :

i) Assume side wing walls resist all bending:

$\therefore$ Max'm. B Mom. $= \frac{502}{2}$

$= 251$ kN.m. per side wall.



$b = 200, \quad d = 1600$ mm.

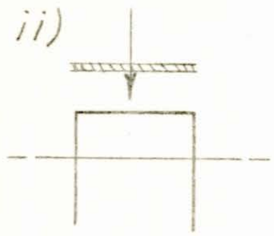
$M_{RC} = 1.576 \times (1600)^2 \times 200 \times 10^{-6}$

$= 807$ KN.m ———————— O.K.

$A_s$ req'd $= \frac{251 \times 10^6}{188 \times 1600} = 834$ mm$^2$

Use 3 - C20 bars Vertically

ii)



$A_s$ req'd $= \frac{502 \times 10^6}{188 \times 1600} = 1668$ mm$^2$

$= 1050$ mm$^2$/m

$\therefore$ Use C16 bars at 300 crs, both faces Vertically

(Min'm. — 10 No.)

## STABILITY

Dead Load :– Tower $= 0.2 \times (1.7 + 2 \times 1.5) \times 24 = 22.6$ kN/m

$\times 22$

$= 497$

SAY 500 kN.

Base $= 3 \times 3 \times 24 \times 0.60$ $= 130$ KN.

TOTAL : 630 KN.

Overturning Mom. = 609 KN.m.

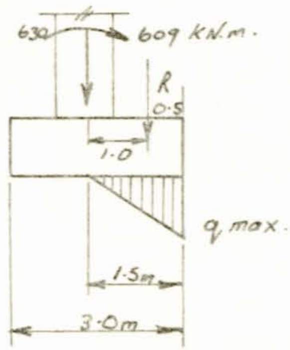
Resisting Mom. $= 630 \times 1.5 = 945$ KN.m.

Factor of Safety $= \frac{945}{609}$

$= 1.55$ ———— O.K.

## FOUNDATION BASE

Max'm. Load = 630 kN.




$e = \frac{609}{630} = 0.97$

SAY = 1.0 m.

Actual bearing press on ground $= \frac{2 \times 630}{3 \times 3 \times 0.5}$

$= 280$ Kpa, high !

∴ Increase to 3.5 m Square :

D. L. → 680 K.N.

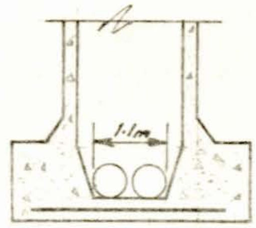
$e = \frac{609}{680} = 0.90$

∴ Actual bearing press on ground base

$= \frac{2 \times 680}{3.5 \times 3.5 \times 0.5} = 222$ Kpa. ———— O.K.

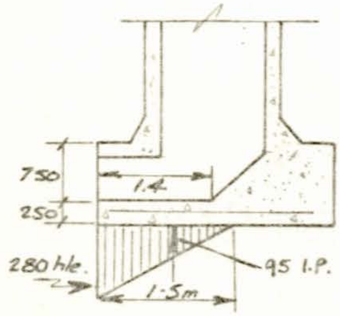
Assume foundation based on rock having minimum bearing capacity of 230 Kpa minimum.

## Footing design :




Shear : ——— o.k.

Bending : Critical location under double pipes sink into base :



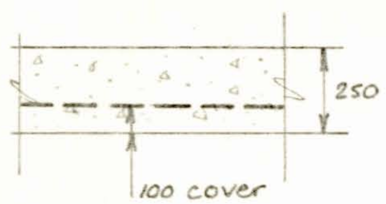

∴ Max'm pressure (average) under base $= \frac{280 + 96}{2}$

$= 188$ Kpa

∴ Max'm. B. Mom. $= 188 \times (1.1)^2 \times 0.085$

Neg. $= 19.0$ KN.m

Minimum thickness of base = 250 mm.




Cover = 100 mm, ∴ d = 150 m (for Neg B.M.)

$M_{RC} = 1.109 \times (150)^2 \times 10^{-3} = 25$ KN.m $> 19$

∴ ——— o.k.

∴ $A_s$ req'd. $= \frac{19 \times 10^6}{209 \times 150} = 606$ mm²/m

Use C16 bars at 300 crs.

Check for Pos. B. Mom. :-

Max'm. Pos. B. Mom. $= 188 \times (1.1)^2 \times 0.042$

$= 9.5$ KN.m.

$M_{RC} = 1.109 \times (100)^2 \times 10^{-3} = 11.09$ KN.m. ——— o.k.

∴ $A_s$ req'd $= \frac{9.5 \times 10^6}{209 \times 100} = 455$ mm²/m $< 606$ mm²/m

∴ ——— o.k.

∴ ADOPT 3.5 x 3.5 x 1.0 m DEEP CONC. PAD FOOTING REINF'D WITH C16 BARS AT 300 CRS, BOTH WAYS, TOP AND BOTTOM.

## WEIR BOARDS

LOADING:

Max'm. pressure $= 10{\cdot}17\ kN/m^3$

At 22 m. max'm depth

Max'm. Press. $= 10{\cdot}17 \times 22$

$= 224\ Kpa.$

span = 1200 mm.

Max'm. B. Mom. $= 224 \times (1{\cdot}2)^2 \times 0{\cdot}125$

$= 40{\cdot}32\ KN.m/\ m.\ height$

For 125 mm thick boards — Sel. HWD. ., Grade 'F14'

$$Z = \frac{1000 \times (125)^2}{6} = 2{\cdot}61 \times 10^6\ mm^3$$

$$f_b = \frac{40{\cdot}32 \times 10^6}{2{\cdot}61 \times 10^6}$$

$$= 15{\cdot}45\ Mpa$$

$$F_b = 14 \times 1{\cdot}1 = 15{\cdot}4\ Mpa.$$

Check sizes at depths:

For 100 mm thick boards:

$Z = 1{\cdot}67 \times 10^6\ m^3$

Allowable Mom. $= 1{\cdot}67 \times 15 = 25{\cdot}05\ KN{\cdot}m.$

then press $= \frac{25{\cdot}05 \times 8}{(1{\cdot}2)^2}$

$= 139{\cdot}2\ Kpa.$

$\therefore$ Height $= \frac{139{\cdot}2}{10{\cdot}17} = 13{\cdot}63\ m$

$\therefore$ use 100 thick boards at 13·0m depth.

For 75 mm thick boards:

$Z = 0{\cdot}94 \times 10^6\ mm^3$

Press $= \frac{0{\cdot}94 \times 15 \times 8}{(1{\cdot}2)^2} = 78{\cdot}3\ Kpa$

$\therefore$ Height $= \frac{78{\cdot}3}{10{\cdot}17} = 7{\cdot}7\ m$

$\therefore$ use 75 thick boards at 7·7m depth.

For 50 mm thick boards :

$$Z = 0.417 \times 10^6 \ mm^3$$

$$Press = \frac{0.417 \times 15 \times 8}{(1.2)^2}$$

$$= 34.75 \ Kpa.$$

$$Height = \frac{34.75}{10.17} = 3.42 \ m$$

$\therefore$ use 50 thick boards at 3.5m. depth.

ABMINCO N.L. - Luina Tasmania.

Calculations for Decant Tower - No. 2 Tailings Dam. June 1978.

TOWER & PIPE FLOW —

Consider critical case for flooding when dam storage at approximately maximum height.

Dam surface area = 50,000 $m^2$ approx.

For 1 in 100 year storm;

max'm. rainfall – 400 mm in 4 days.
Total flow in 4 days = 55 M lit.
Peak flow rate = 1300 k. lit./hr.
(361 lit./sec.)

PIPE —

450 mm Nominal Diameter Conc. Press Pipe.
Base design on one pipe active only :
Fall of pipe :– 1 m. in 200 m.
Discharge = 361 lit./sec (1300 $m^3/hr$)
Velocity = 2·3 m./sec.
Loss of head = 8·5 m / 1000 m of pipe.

∴ For approx. 150 m of pipe length:
Hyd. Head build-up in tower = 1·3 m + Inlet losses.
SAY = 1.6 m high.

C.f. Tower height = 22 m, negligible !
∴ Pipe adequate and is not the critical criterion.

TOWER — OUTLET WEIR

Assume maximum flow occurs in the first day,
SAY 2 × average daily flow :

Rainfall = $\frac{55}{2}$ = 27·5 M. lit. / 24 hrs.
= 27500 $m^3$/24 hrs.
(1146 $m^3$/hr.)
= 318 l/sec.

Increase in water level due to rainfall.

$$= \frac{27500}{50,000} \times 10^3$$

$$= 550 \text{ mm}$$

$\therefore$ Average level $= \frac{550}{2} = 275$ mm in depth at weir boards.

Flow over weir:

$C_D = 0.6$

$L = 1.2$ m

$g = 9.8$ m/sec²

$$Q = 2/3 \, C_D L (h)^{3/2} \sqrt{2g}$$
$$= 2/3 \times 0.6 \times 1.2 \times (0.275)^{3/2} \times \sqrt{2 \times 9.8}$$
$$= 0.306 \text{ m}^3/\text{sec.}$$
$$= 306 \text{ lit./sec.}$$
$$(1102 \text{ m}^3/\text{hr.})$$

Second day:

$$\text{Rainfall} = \frac{55}{4} = 13.75 \text{ M. lit./24 hrs.}$$
$$= 159 \text{ lit./sec.}$$
$$(572 \text{ m}^3/\text{hr.})$$

$$\text{Increase in water level} = \frac{13750}{50,000} \times 10^3$$
$$= 275 \text{mm.}$$

Average = 138 mm.

Remainder from 1st day:

$$(1146 - 1102) \times 24 = 1056 \text{ m}^3$$
$$\frac{1056}{50,000} = 21 \text{mm.}$$

$\therefore$ Increase in height $= 138 + 21 = 159$ mm.

$$Q = 2.125 \times (0.159)^{3/2}$$
$$= 0.135 \text{ m}^3/\text{sec.}$$
$$= 135 \text{ lit./sec.}$$
$$(485 \text{ m}^3/\text{hr.})$$

Third day:

$$\text{Rainfall} = \frac{55}{4} \times 2/3 = 9.17 \text{ M lit./24 hrs.}$$
$$(382 \text{ m}^3/\text{hr.})$$
$$= 106 \text{ lit./sec.}$$

Increase in water level $= \frac{9170}{50,000} \times 10^3 = 1840$ mm.

Average $= 92$ mm.

Rem. $= (485 - 382) \times 24 = 2472\ m^3$

$\frac{2472}{30,000} = 49$ mm.

$\therefore$ Increase in height $= 92 + 49 = 141$ mm.

$Q = 2.125 \times (0.141)^{3/2}$
$= 0.113\ m^3/sec.$
$= 113$ lit./sec. $(405\ m^3/hr.)$

NOTE: 113 l/sec. > 106 l/sec, indicating that flow over weir equalled rainfall into dam at some time in the third day, and will continue to do so for duration of storm. i.e. Weir discharge Q = Rainfall.

$\therefore$ Tower adequate to cater for increase in dam capacity due to 1 in 100 year storm.

Operating conditions:

Deliberate Input (Tailings) = 5000 lit/min.
= 83 l/sec
$(300\ m^3/hr.)$

Increase in water level $= \frac{300}{50,000} \times 10^3$
$= 6$ mm/hr.

Weir flow, $Q = 2.125 \times (0.006)^{3/2} \times 10^3$
$= 0.99$ lit./sec.
$(3.6\ m^3/hr.)$

ie: Inflow exceeds outflow,: water level will build up until discharge over tower weir matches inflow;

For tower weir flow to match inflow rate:

$Q = 83 = 2.125 \times (x)^{3/2} \times 1000$

$\therefore$ Head, $h = 0.115$ m
$= 115$ mm.

For flow at 6 mm/hr.

$$\text{Time req'd} = \frac{115}{6} = 19 \cdot 17 \text{ hr.}$$

BUT Q inflow $= 300 \times 19$

$= 5700 \ m^3/19 \text{hrs.}$

→ Increase in water level

$$\frac{5700}{50,000} \times 10^3 = 114 \text{ mm}$$

Average inc. $= 57$ mm

→ Q (over weir) $= 2 \cdot 125 \times (0 \cdot 057)^{3/2} \times 10^3$

$= 28 \cdot 92$ l/sec.

(104 $m^3/hr$)

∴ For weir flow = inflow

$$Q = (83 - 28 \cdot 9) = 2 \cdot 125 \times (x)^{3/2} \times 10^3$$

$$\therefore h = 86 \cdot 5 \text{ mm}, \quad t = \frac{86 \cdot 5}{6} = 14 \cdot 4 \text{ hrs.}$$

∴ Water level in dam will build up to approx. 90 mm. head before flow over weir equals flow into dam. This will take approx. 15 hrs. from commencement of flow.

**ABMINCO** MEMORANDUM

Date 4th April, 1978 Ref GOB:JM

To Engineering Superintendent From G. Boyle

At Cleveland At Cleveland

Copies to Keep

Subject ROCK TYPES AND CONDITIONS IN THE AREA AFFECTED BY THE EXTENSIONS TO THE No. 2 TAILINGS DAM.

The ridge between No.1 and No.2 Tailings Dams consists of a sequence of serpentinite, medium grained volcanics (microgabbro) and tuffs, apparently vertically bedded, overlain at the south-western end of the ridge by a sequence of shale and chert dipping gently toward the Whyte River.

1. Serpentinite

This forms the core of the ridge near the current channel to the tailings dam. It is fresh, broadly jointed and has only a thin soil cover, rarely over 0.3 M in depth.

2. Volcanics

This forms the core of the ridge from the channel to the haulage road and portion of the ridge south of the channel. It is well weathered, well jointed and generally has soil cover to 0.3 M - 0.6 M deep. The rock will probably be of poor mechanical strength and decompose to clay with saturation by water. It should be readily rippable by a bull-dozer but where fresh should be strong and only broadly jointed. The depth of weathering is unknown. For optimum stability this should be ripped to hard rock.

3. Tuffs

These are exposed along the edge of the tailings dam in the area between the reclamation dump and the water. They are only lightly weathered, very closely jointed and should be readily rippable. Only 0.3 M to o.6 M solid cover. This could provide a barely adequate gravel, its tendency toward decomposition is unknown.

4. Cherts and Shales

These form a cap to the ridge at the Whyte River end. The shale underlies the chert, both contacts appear to be sharp and not marked by any weathered zones. The rocks are generally fresh although the chert is closely jointed and locally severely decomposed. Soil cover is generally 0.3 to 0.6 M. The bedding of these rocks, subhorizontal through the ridge, could cause problems of leakage.

No major shear zones of other potential leakage zones except the cherts were observed.

G Boyle

G. Boyle
Assistant Mine Geologist.

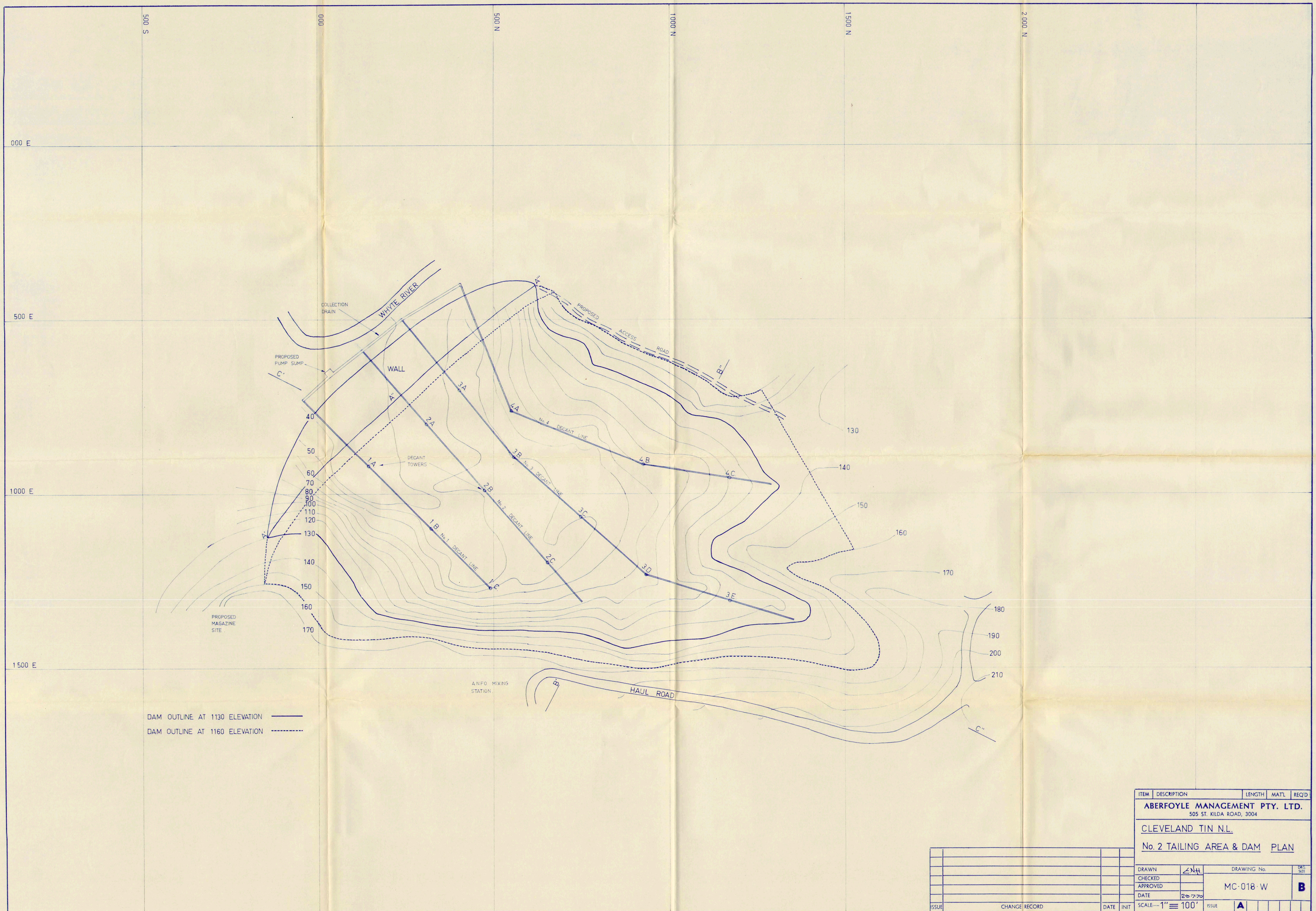
ABMINCO N.L. CLEVELAND MINE LUINA TASMANIA 7321

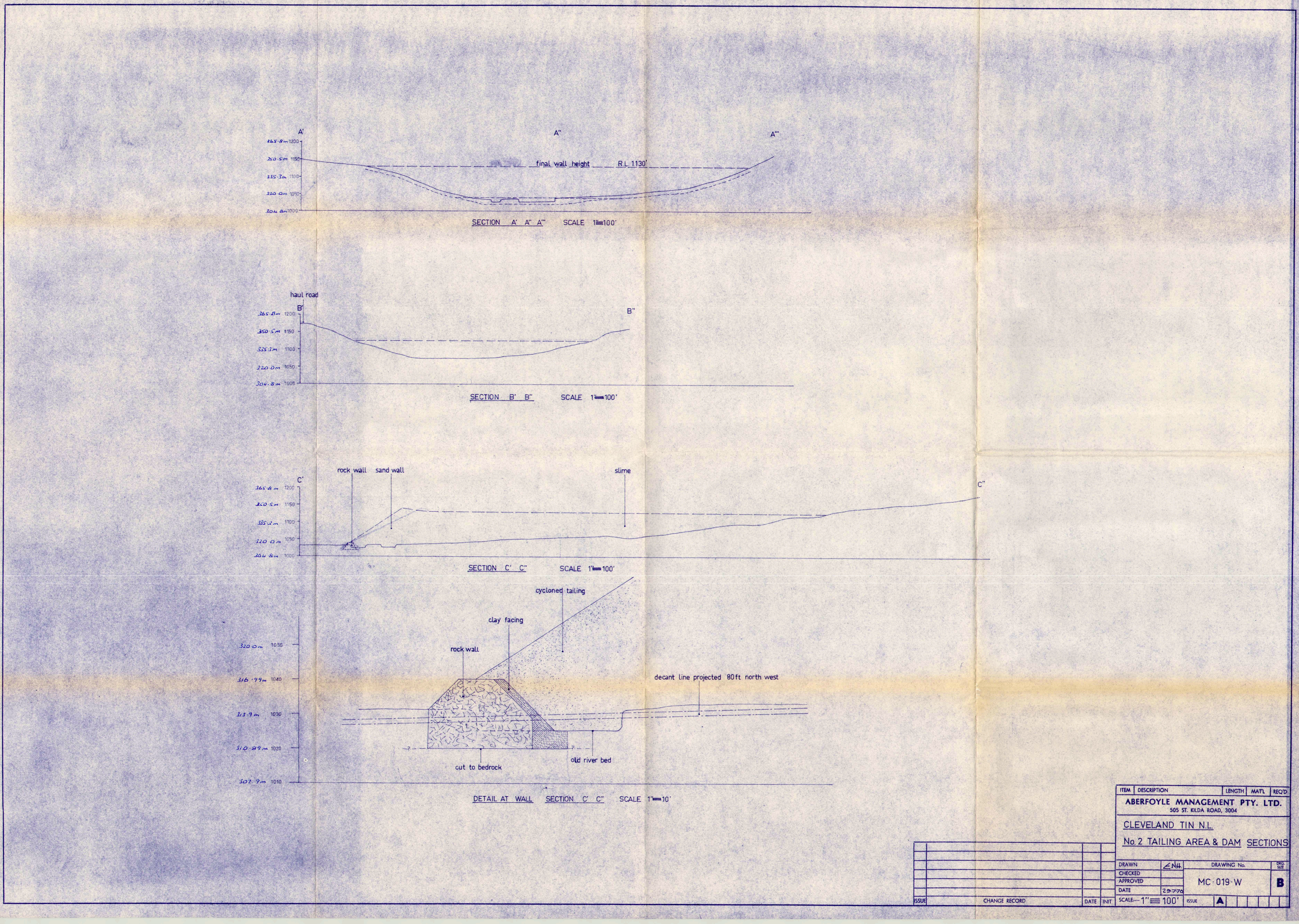
| | 1967 | 1968 | 1969 | 1970 | 1971 | 1972 | 1973 | 1974 | 1975 | 1976 | 1977 | 1978 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| JANUARY | No Records As Mine Not Operational | 46.5 | 111.0 | 89.2 | 104.1 | 53.3 | 122.4 | 9.1 | 63.1 | 59.2 | 127.4 | 69.0 |
| FEBRUARY | | 125.5 | 159.5 | 33.5 | 63.5 | 55.9 | 105.4 | 36.3 | 12.0 | 34.4 | 55.8 | 112.8 |
| MARCH | | 80.2 | 95.0 | 90.9 | 36.8 | 76.2 | 128.3 | 42.5 | Not Available | 73.1 | 163.6 | 73.9 |
| APRIL | | 230.4 | 214.9 | 155.4 | 156.2 | 161.5 | 263.1 | 186.9 | 104.2 | 135.3 | 189.0 | 167.0 |
| MAY | | 388.9 | 172.5 | 211.6 | 228.6 | 99.1 | 415.8 | 55.5 | 429.4 | 288.6 | 230.4 | 167.9 |
| JUNE | 101.1 | 282.5 | 125.0 | 193.8 | 289.6 | 174.8 | 270.8 | 311.3 | 142.8 | 190.2 | 120.9 | 56.3 |
| JULY | 187.2 | 242.8 | 280.7 | 394.2 | 177.8 | 388.9 | 148.8 | 298.0 | 347.9 | 181.6 | 227.5 | |
| AUGUST | 177.0 | 344.7 | 202.7 | 338.1 | 237.5 | 257.0 | 171.5 | 304.1 | 198.0 | 229.4 | 243.0 | |
| SEPTEMBER | 154.2 | 254.5 | 235.0 | 236.7 | 251.5 | 203.5 | 259.1 | 283.6 | 190.8 | 121.3 | 65.5 | |
| OCTOBER | 153.4 | 346.7 | 73.4 | 165.1 | 270.5 | 108.0 | 96.5 | 155.6 | 141.7 | 94.9 | 191.8 | |
| NOVEMBER | 187.2 | 384.8 | 97.5 | 53.4 | 154.7 | 95.0 | 106.4 | 60.9 | 214.6 | 115.0 | 105.2 | |
| DECEMBER | 162.1 | 40.1 | 205.5 | 165.1 | 144.8 | 102.6 | 167.4 | 230.2 | 51.2 | 338.0 | 213.5 | |
| TOTAL FOR YEAR | 1122.2 | 2767.6 | 1972.7 | 2127.0 | 2115.6 | 1775.8 | 2255.5 | 1974.0 | 1895.7 | 1856.6 | 1933.6 | |

NOTE!

ALL RECORDINGS ARE SHOWN IN "MILLIMETRES" (mm)

# RAINFALL CHART


500 S
000
500 N
1000 N
1500 N
2 000 N
000 E
500 E
1000 E
1500 E
WHYTE RIVER
COLLECTION DRAIN
PROPOSED PUMP SUMP
WALL
PROPOSED ACCESS ROAD
DECANT TOWERS
No. 1 DECANT LINE
No. 2 DECANT LINE
No. 3 DECANT LINE
No. 4 DECANT LINE
PROPOSED MAGAZINE SITE
ANFO MIXING STATION
HAUL ROAD
DAM OUTLINE AT 1130 ELEVATION
DAM OUTLINE AT 1160 ELEVATION
ITEM
DESCRIPTION
LENGTH
MAT'L
REQ'D
ABERFOYLE MANAGEMENT PTY. LTD.
505 ST. KILDA ROAD, 3004
CLEVELAND TIN N.L.
No. 2 TAILING AREA & DAM PLAN
DRAWN
CHECKED
APPROVED
DATE 29.7.70
SCALE:—1" ≡ 100'
DRAWING No.
MC·018·W
DRG. SIZE
B
ISSUE A
ISSUE
CHANGE RECORD
DATE
INIT


A'
A''
A'''
final wall height
R.L. 1130'
SECTION A' A'' A'''
SCALE 1"=100'
haul road
B'
B''
SECTION B' B''
SCALE 1"=100'
C'
rock wall
sand wall
slime
C''
SECTION C' C''
SCALE 1"=100'
cycloned tailing
clay facing
rock wall
decant line projected 80ft north west
old river bed
cut to bedrock
DETAIL AT WALL SECTION C' C''
SCALE 1"=10'
ITEM
DESCRIPTION
LENGTH
MAT'L
REQ'D
ABERFOYLE MANAGEMENT PTY. LTD.
505 ST. KILDA ROAD, 3004
CLEVELAND TIN N.L.
No. 2 TAILING AREA & DAM SECTIONS
DRAWN
CHECKED
APPROVED
DATE
29.7.70
DRAWING No.
MC·019·W
DRG. SIZE
B
ISSUE
CHANGE RECORD
DATE
INIT
SCALE:— 1"=100'
ISSUE
A


LEGEND
Present Contours
Final Height Contours
AMFO MIXING STATION
ROAD
NO. 2 TAILINGS DAM
RUBBISH TIP
WHYTE RIVER
NO. 1 TAILING DAM
ABMINCO N.L.
TASMANIA
DRAWN
TRACED
CHECKED
APP'D S/SURV
APP'D M/SURV
CONTOUR
Plan of Tailings
Dam Area
MC-052-W


Stage 1 crest RL 350 m
Stage 2 crest RL 360 m
Cherts
Volcanics
Shale
Serpentinite
Tuffs
ABMINCO N.L — TASMANIA
DRAWN D.R.J.
TRACED G.M.H. & D.R.J.
CHECKED
App'd S/Surv'
App'd M/Sup'
DATE 17 Jan 1977
Contour Plan
No. 2 Tailings Dam
Area - AREA GEOLOGY
SCALE 1:500
Sheet 1
MC-053-W


SECT. H
SECT. G
SECT. F
SECT. E
SECT. D
SECT. C
SECT. B
SECT. A
Tower
350
340
Estimated dam floor
SECT. D
345
330
315
road
MC-054-W
CLEVELAND TIN N.L.
19/12/76
9/11/77
N°2 TAILINGS DAM
PLAN & CROSS - SECTIONS
SCALE 1:500


MINE WATER DISCHARGE PIPELINE
AN-FO Mixing Station
EXISTING PIPELINE
ϕ150mm PIPE
EXISTING DISCHARGE
PROPOSED PIPELINE RELOCATION
PROPOSED DISCHARGE
Tower
Nº2 Tailings Dam
PROPOSED TOWER
2×460 NB (18") RC. DISCHARGE PIPES
360
355
350
338·2
338·8
SUMP
RECONTOUR EXISTING CHANNEL
DISCHARGE TO RECLAIMED WATER DAM
Nº1 Tailings Dam
15200 N
9700 E
15100 N
CLEVELAND TIN N.L. —— TASMANIA
Revised PWD 9·11·77
Nº2 Tailings Dam Area
MC-055-W
SCALE 1:1000