ACRES

Design Calculation Cover Sheet

PROJECT No. P5700.06

FILE No. P5700.14.06.09

SERIAL No. 0032

PROJECT TITLE SUSITNA HYDROELECTRIC PROJECT

ALASKA POWER AUTHORITY

DEPARTMENT GEOTECHNICAL - BUFFALO

CALCULATIONS FOR: WATANA DIVERSION TUNNELS.
ROCK MECHANICS

ORIGINAL BY N.A.J. BOND DATE 1 OCT / 1981.

CHECKED BY DATE / /

REV No.	BY	DATE	CHECKED	DATE
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SUPPORT CALCS
ROCK MECHANICS ASPECTS
CLOSE-OUT REPORT (Ref).

APPROVED BY

PROFESSIONAL ENGINEERS SEAL

ORIGINAL COPIES PLACED IN MAIN FILE ON CLOSURE OF PROJECT

BY

DATE 2/20/83

TOTAL No. OF SHEETS 46 W/COVER

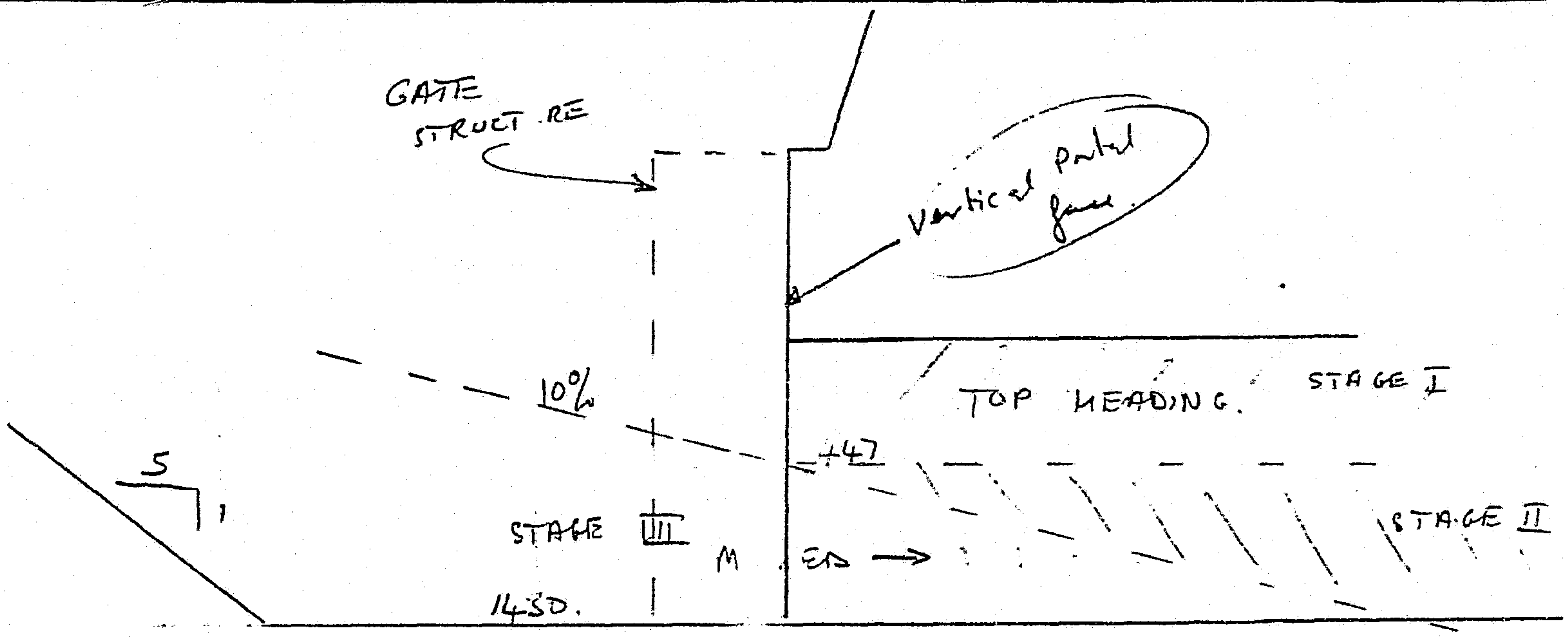
60.04.01 Form 146 A

ACRES

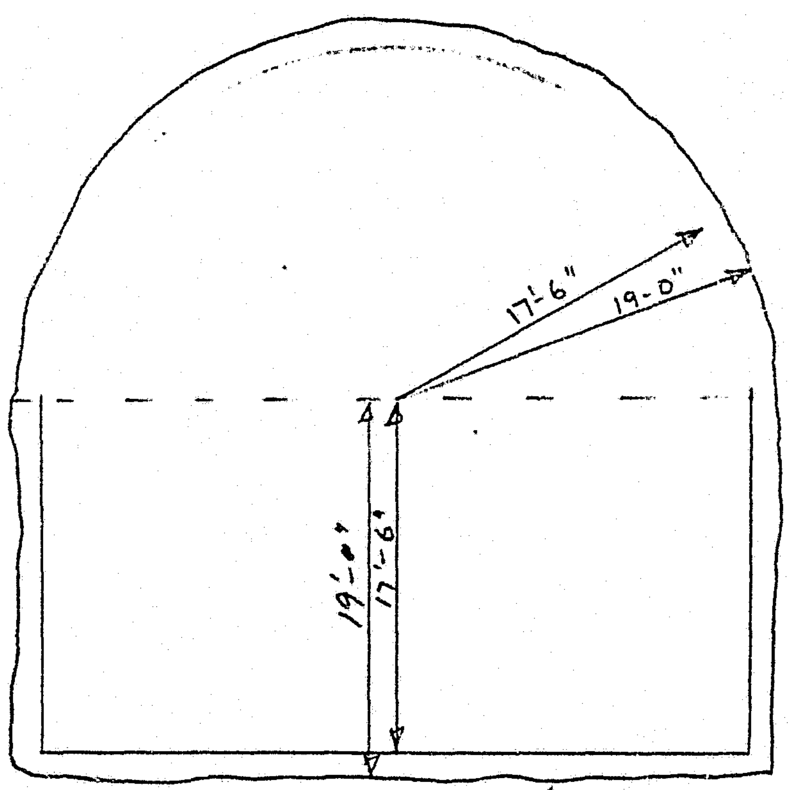
Calculations

SUBJECT: SUSITNA - WATANA
DIVERSION TUNNELS

JOB NUMBER P5700. 06.
FILE NUMBER
SHEET 1 OF 5
BY NAOB. DATE OCT 1981
APP DATE



RAMP AND MUCKING SEQUENCE U/S PORTAL.



DIVERSION TUNNEL
X SECTION.

EXCAVATION AREA
TOP 63 yd²
BENCH 80 yd²

DRILL LENGTH 12'-0"
EFFECTIVE PULL 11'-0"

DRILL 3 hours.
CHARGE 1 hour
CLEAN ½ hour
MUCK 3 hrs.

1 ROUND PER SHIFT
MAXIMUM
(ASSUME 3 SHIFT SYSTEM)

FORM NO. 152 REV. 1

Class I RQD 95-100%. 32% length of tunnel

$J_n = 6$ Assume one joint set drops out with depth.

$J_r = 1.5$

$J_a = 0.75$

$J_w = 1.0$

$SRF = 1.0$

$\sigma_c = 20{,}000$ psi

$\sigma_1 = \frac{550 \times 2.7 \times 62.4}{144}$

$= 643$ psi

$\frac{\sigma_c}{\sigma_1} =$

$$Q = \frac{97.5}{6} \times \frac{1.5}{0.75} \times \frac{1.0}{1.0}$$

$$= 32.5.$$

Span = 35' ESR = 1.6

Equiv Dimension = 21.8' = 6.6 m

No support required.

Class II. RQD = 90-95%

$J_n = 6$

$J_r = 1.5$

$J_a = 0.75$

$J_w = 1.0$

$SRF = 1.0$

$$Q = \frac{92.5}{6} \times \frac{1.5}{0.75} \times \frac{1.0}{1.0}$$

$$Q = 30.8$$

No support required.

ACRES **Calculations** SUBJECT:	JOB NUMBER P5700.06 FILE NUMBER SHEET 3 OF 5 BY NAOB DATE OCT 1981 APP DATE

Class III RQD = 75-90%. Ave 82.5%.

J_n = 9 3 joint sets.
J_r = 1.5
J_a = 2.0
J_w = 1.0
SRF = 2.5

$$Q = \frac{82.5}{9} \times \frac{1.5}{2.0} \times \frac{1.0}{2.5}$$

$$= 2.7$$

Zones 21 and 22.

Rock bolts 1m centres 2-3 cm shotcrete.

Class IV RQD = 50% - 75% Ave 62.5%

J_n = 9
J_r = 1.5
J_a = 2.0
J_w = 0.66
SRF = 2.5

$$Q = \frac{62.5}{9} \times \frac{1.5}{2.0} \times \frac{0.66}{2.5}$$

$$= 1.37$$

Support zones. 23/22.

Bolts 1m centres plus 50mm shotcrete.

FORM NO 52 REV. 1

ACRES

Calculations

SUBJECT:

JOB NUMBER	P5700.06
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Class V RQD 25% - 50% Ave 37.5%.

$J_n = 9$

$J_r = 1.5$

$J_a = 5.0$

$J_w = 0.66$

$SRF = 2.5.$

$$Q = \frac{37.5}{9} \times \frac{1.5}{5.0} \times \frac{0.66}{2.5}$$

$= 0.32$

Zone 31 Steel Sets with insitu Concrete arch

Class VI RQD = 0% to 25% Ave 12.5,

$J_n = 9$

$J_r = 1.0.$

$J_a = 8.$

$J_w = 0.66$

$SRF = 5.0.$

$$Q = \frac{12.5}{9} \times \frac{1.0}{8} \times \frac{0.66}{5.0}.$$

$Q \approx 0.02$

Zone 34 and 35. Steel sets with insitu concrete arch.

FORM NO. 152 REV. 1

ACRES

Calculations

SUBJECT:

JOB NUMBER P5700.06

FILE NUMBER

SHEET 5 OF 5

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APP DATE

Rock bolts Length $L = 2 + 0{\cdot}15 \times \frac{10{\cdot}6}{1{\cdot}6}$ in metres.

$= 3{\cdot}0$ m.

$= \underline{\underline{10'}}$

FORM NO. 152 REV. 1

SUSITNA HYDROELECTRIC PROJECT

WATANA - ROCK MECHANICS ASPECTS OF DIVERSION TUNNELS

1 - GENERAL

The layout assumed for these tunnels was determined from drawings No:

SK 5700 C6 218 A	October 14, 1981
SK 5700 C6 226	October 13, 1981
SK 5700 C6 227	October 13, 1981

The plan position shown on drawing SK 5700 C6 218 A is not consistent with the upstream portal location. It is understood that the drawing SK 5700 C6 226 is the preferred portal location and the plan location of the tunnel at the upstream end will be moved approximately 140 feet south.

The tunnel is assumed to be "Dee" shaped with a span of 35 feet and height of 35 feet (see attached sketch). The tunnels are 4,200 feet long. It is understood that the tunnels are required to be lined with insitu concrete for hydraulic consideration. It is assumed that the lining would be at least 1.5 feet thick, thereby increasing the excavated span to 38 feet.

The left tunnel falls at 0.95% downstream from invert elevation 1,490 feet to 1,450 feet. The right tunnel falls at 0.24% downstream invert elevation from 1,430 feet to 1,420 feet. The tunnels are spaced 2.5 D when D is the span of the tunnel center to center, i.e. 57 feet of rock pillar horizontally between tunnels.

The tunnel alignment is 255° (W.C.B.) which is approximately parallel to the river for about 80% of its length. The maximum vertical cover to the tunnel crown is 550 feet. The distance from the tunnel to the river varies from zero to 75 ft over lengths 800 feet

The tunnel alignment passes through 6 surface mapped fracture zones. The fracture zones intersect the tunnel alignment at 50° to 60°. The width of fracture zones, as mapped on the surface, total 220 feet, i.e. 5% of the tunnel length but it is expected that there will be some shears not mapped which will occur at depth and also there will be some improvements of the shear zones with depth. It will be assumed that these two factors will tend to cancel each other out.

This is reflected in the results from the exploratory drillings.

BOREHOLE ROCK QUALITY DISTRIBUTION
RQD

BOREHOLE NO.	0-25%	25-50%	50-75%	75-90%	90-95%	95-100%
2	19	15	22	17	13	14
6	5	2	13	22	15	43
8	4	7	18	16	18	38
	28	24	53	55	46	95
% Borehole Length	9%	8%	18%	18%	15%	32%

Although the boreholes drilled from the surface vertically or near vertically is not directly analogous to a horizontal tunnel, there ~~are~~ is sufficient similarit~~ies~~y to give a good indication of expected rock quality.

2 - ORIENTATION OF JOINTING

The tunnel alignment parallels the major joint set II and intersects major joint set I at an angle of 35° to 75°. The major joint sets are steeply dipping. See "Geology of Diversion Tunnels".

3 - UPSTREAM PORTAL

The portal is located just to the south of the "Fins". Minor shearing associated with the "Fins" intersects the portal area at approximately 30° to 40° with the line of the tunnel. The extent of this shearing is not yet defined. It should be assumed that the cut slopes will be at 1H:4V with 10 feet wide berms at 40 foot vertical intervals.

The portal arrangement, ~~as shown on drawing SK 5700 GS 226,~~ is governed mainly by the access required along the river bank. It is anticipated suggested that the access ramp start within the tunnel at 1,430 feet elevation. This will shorten the ramp and allow some saving in excavation of the cut for the access road. The ramp and remaining excavation down to 1,430 feet elevation within the portal area could be excavated after the tunnel breakthrough and mucked out through the tunnel. ~~Using this method of working, the portal can be moved 50 feet towards~~

~~the river~~. If the rock surface adjacent to the river is lower than at presently indicated, the level of the river retaining rock dyke will have to be raised by a small embankment. Because of the strong river current at the outside of the bend, a concrete gravity ~~gravel~~ wall may be preferable.

It should also be considered that the minor shearing associated with the "Fins" will pass through the rock dyke and considerable rock support may be required. The permeability of the dyke may need to be reduced by grouting or dumping impervious material on the upstream face. In view of the fast flow of the river at this point, grouting should be allowed for at this stage.

Considering the probable shearing parallel to the "Fins", extensive bolting of the whole portal face should be allowed for a 25 foot long tensioned rock bolt at 5 foot centers and two closely spaced rows of rock bolts around the periphery of the tunnel. 50% of the area of the cut slopes will require to be shotcreted to a thickness of 3 inches, 25% of the area reinforced with mesh. The sides of the portal excavation will require 50% of the area rock bolted with 25 foot long bolts.

4 - TUNNELS

For hydraulic considerations the diversion tunnels will be lined with insitu concrete. Sections of the tunnel will require temporary rock support for the time the rock is exposed between excavation and concrete lining. 17% of the tunnel length will require concrete lining from support considerations. ~~See attached support criteria~~.

Temporary support should be installed soon after excavation and light support may be used to prevent excessive overbreak thereby saving on concrete to fill the overbreak.

The orientation of the tunnel alignment relative to joint set II is unfavorable but due to the restriction on the location of the portal, the alignment of the tunnel cannot be changed to have any significant effect on the support required.

It is expected that the excavation will be in two stages. The semi-circular top section followed by the lower bench to final invert level.

In good rock with minimum support, advance rates of 160 feet per week for the top heading could be achieved. An average of 100 feet per week overall is expected. The bench excavation should average 300 feet per week. Enlargement of the upper tunnel in two locations is required for energy dissipation structures and gates. This will be mainly an enlargement vertically and, therefore, will not increase the tunnel span. Extra supports in the form of rock bolts and shotcrete will probably be required. Slight adjustment of the gate structures may be made to locate the enlargements in good rock away from shear zones. At the location of these structures and the plug in the lower tunnel, the concrete should be well ~~beyond~~ keyed into the rock. The normal irregular overbreak associated with drill and blast excavation should be sufficient; but if the quality of rock and blasting is such that a very smooth profile is being obtained, then some trimming or adjustment of peripheral holes will be required.

At the downstream end of the lower diversion tunnel, a junction will be formed with the tailrace tunnel. This junction should be formed at the same time as the diversion tunnel is constructed to avoid blasting close to the concrete lining of the diversion tunnel. A stub heading about 40 feet long on the line of the tailrace tunnel should be excavated and left unlined until the tailrace tunnel is completed.

The junction of the access tunnels with the energy dissipating gate structures should also be formed prior to concreting of the upper diversion tunnel.

5 - DOWNSTREAM PORTAL

The location of this portal is severely restricted due to the close proximity of the downstream cofferdam, the chute spillway and service spillway discharge valves and tailrace tunnel. The geology of the portal area is controlled by the major shears trending at 45° to the tunnel axis. A zone of alteration with minor shears and fractures has been mapped on the surface crossing the tunnel alignment 100 feet to 200 feet from the portal.

As at the upstream portal, a rock dyke will give protection to the portal area during construction. It is anticipated that the excavation slopes will be at 1H:4V. The portal face will be extensively bolted in the same manner as the upstream portal.

Drawing SK 5700 C6 218 shows an upstand between diversion tunnel portals; but with the tunnels at 30 feet difference in elevation with 57 feet spacing, this upstand would probably require extensive support. It may be more economical to remove this rock upstand. Generally, a rock cover of 1.5 D (where D is the tunnel span) is required. However, since the tunnels will be concrete lined, it may be possible to reduce this requirement to 1.0 D cover. Since the portal is in a zone of major shearing and alteration, a cover of 1.5 D should be assumed until more detailed information is available on the portal geology.

6 - EXCAVATION SLOPE DRAINAGE

Drainage channels should be provided at the top of all slopes to channel surface run off away from the cut slopes. Pressure relief holes may be required. For estimating, assume 25% of the cut slope area will require drain holes 10 feet X 10 feet grid ~~20~~ 15 feet in depth for upstream and downstream portals.

Subtask 6.21 - Watana Diversion Scheme

Closeout Report

VERY ROUGH DRAFT JWH 1/7/82

1 - INTRODUCTION

The objective of this closeout report is to present the results of the Diversion Scheme studies and present information on the selected scheme.

Basically a diversion scheme study is a simple economic optimization of the tunnel diameter vs. cofferdam height. In optimizing the tunnel diameter certain limiting criteria have to be adhered to. These would include but are not limited to geologic conditions which would determine structural support conditions for the tunneling, foundation conditions which influence the type of cofferdam and treatment, physical geometry of the site which would limit the size or type of cofferdam, flows to be handled by the scheme, and construction sequence and scheduling.

An additional consideration that had to be investigated and evaluated for the Watana diversion scheme is the incorporation of the low level outlet into the diversion tunnel. The requirement and parameters for the low level outlet are presented in Closeout Report 6.17, Watana Spillways, Preliminary Design.

2 - SCOPE

The scope of the study was to conceptualize a diversion scheme or schemes, determine and evaluate the flows and water levels the scheme would be required to handle, determine the various heights of cofferdams required for the various tunnel sizes, estimate capital costs for the various schemes, select the tunnel size and corresponding cofferdam height. Upon completion of the selection of the optimum tunnel diameter, the preliminary design of the tunnel and cofferdam used for the capital costing aspect are refined and confirmed.

Operations during construction were considered and incorporated into the preliminary design of the tunnels. The operation of tunnels during final closure and the final closure scheme or sequence itself was determined.

As part of the hydraulic studies incorporated in the diversion studies was the development of downstream tailwater elevations considering ice buildup in the downstream reach of the river. This directly affected the water surface elevations which directly determined the upstream cofferdam height.

The incorporation of a permanent low level outlet into one of the diversion tunnels was considered. This involved preliminary design comparison of a separate low level outlet versus incorporation of a low level outlet.

3 - DESIGN CONSIDERATIONS

3.1 - General

The first parameter to be established in any diversion study is the flow or recurrance period flood that has to be handled by the diversion scheme. This is determined by an economic risk analysis in which the cost of the diversion scheme is compared against the damage that would result from a flood beyond the capabilities of the diversion scheme, and the risks involved in exceeding the capabilities of the scheme.

This ~~would~~ will be carried out during the detailed design phase of the project however general criteria by others such as the U.S.B.R. and the U.S. Army Corps of Engineers have established criteria and experience which is acceptable for feasibility studies.

The general criteria adopted by ______________ and recognized by international insurance companies is a 10% risk per annum is acceptable. This translates to a diversion design for a 1 in 10 year exceedance flow or flood per year of construction. Preliminary estimates show that a critical construction period of 5 years is required ~~to construct the main dam to an elevation where~~ where damage from ~~cofferdam~~ a flood overtopping the cofferdam would ~~not~~ be unacceptably detrimental to the main dam and ~~opportunities.~~ appurtenances. Thus a 10 year exceedance risk per year on a cumulative basis for a 5 year construction period equates to a 50 year recurrance period flow or a 10% risk of exceedance for the entire 5 year period. This flow has been established as 83,000 cfs. (See Subtask 6.09, Design Criteria).

Once the flow or flood hydrograph is selected several diversion schemes were devised and layed out to conform to the site geometry and characteristics. Additional design considerations are ~~Other additional criteria is~~ outlined in the following subsections.

3.2 - Reservoirs

In the tunnel/cofferdam optimization studies the height of the cofferdam, and thus the size of the reservoir, is optimized by economic comparison against the tunnel diameter.

The Watana Reservoir resulting from the cofferdam was analized for possible flood storage, and routing of the design flood. The reservoir was too small to provide any storage and the routing effects were very small.

3.3 - Cofferdams

The cofferdams will be earth and rockfill structures with the height to be determined from the optimization studies. Consideration will be given to foundation treatment due to the alluvium present in the Susitna riverbed.

(a) Foundation Treatment

Foundation treatment will consist of a slurry wall through the alluvium material to bedrock excavation to sound rock in the abutment areas.

The depth of alluvium material in the river bed area ranges from non existant to a maximum of 100 feet. The alluvium material is a silty sandy gravel with numerous cobbles. The soil/bentonite slurry wall will be constructed through the closure dam and alluvium material to bedrock and will minimize the amount of seepage into the maindam excavation. The abutment areas will be cleared and grubbed with excavation of all material to sound rock prior to placement of any cofferdam material.

(b) Upstream Cofferdam

The upstream cofferdam will be a zoned embankment founded on the closure dam. See Figure/Drawing _____. The closure dam will be constructed to elevation 1475 based on a low water level of elevation

1470. It will consist of coarse material on the upstream side grading to finer material on the downstream side. When the closure dam is completed the soil/bentonite slurry wall can be constructed to minimize seepage into the main dam foundation excavation. If the slurry wall is not effective in preventing seepage into the excavation a dewatering system can be established in the main dam excavation.

The cofferdam from elevation 1475 to 1545 will be a zoned embankment consisting of a central core, fine and coarse filters, and rock and/or gravel shells with rip rap on the upstream face. The core material will come from Borrow Area "D" and will be the same material to be used in the main dam. The filter materials will be obtained from Borrow Area "E" and will also be the same as the materials to be used in the main dam. The rock for the cofferdam shells will come from either Quarry L or A. Gravel material will come from Borrow Area "E". The choice of rock or gravel for the shells

(c) Downstream Cofferdam

The downstream cofferdam will be a closure dam constructed from elevation 1440 to 1472. See Figure/Drawing _____. It will consist of coarse material on the downstream side grading to finer material on the upstream side. When the closure dam is completed the soil/bentonite slurry wall can be constructed in the finer material to minimize seepage into the main dam foundation excavation. A dewatering system can be installed in the main dam excavation if required. The downstream slope will be protected by a rip rap layer.

The width of the crest of the cofferdam can be varied if clearance of the outlet structures are required.

3.4 - Inlet and Outlet Structures

The inlet and outlet structures are reinforced concrete structures. The intake structure will support and house vertical lift fixed wheel gates for control and final closure.

The intake and gates will be designed to operate under the following conditions.

(a) Gates open, reservoir partially full
(b) Gates open, reservoir full
(c) Gates partially open, reservoir partially full
(d) Gates partially open, reservoir full
(e) Gates closed, reservoir partially full
(f) Gate closed, reservoir full.

The intake will have a rounded corner (bell mouth) entrance to reduce energy losses and prevent cavitation. The outlet structures will have slots for provisions of stoplog installation.

3.5 - Tunnels

The tunnels were designed to accomodate the structural geology of the site. The major joints sets to be avoided ran ______________________.

Thus the range of unacceptable orientation is ____.

The range of preferred orientation is ____.

The location of the dam and the layouts indicated the tunnel alignment of ____________________________________ is acceptable.

The tunnels are designed to handle the flow stated in Section 3.1 5.1 thus the resulting velocities are approximately 50 ft/sec and are presented in Table _____. This necessitated the concrete lining of the tunnels be of a thickness sufficient to prevent scour.

Geotechnical data and evaluations indicated structural support of the tunnels will be required. For the study purposes a percentage of the tunnel length is estimated to require rock bolting and steel sets for support. This is presented in Table _____. The ~~where~~ cost estimates include these portions of support.

Geotechnical considerations for the sizes of tunnels investigated favored a smaller diameter two tunnel scheme over one large diameter single tunnel. It was also decided for security and risk considerations, two tunnels were superior to a single tunnel.

4 - TUNNEL SCHEMES

4.1 - General Description

The general schemes considered during the study consisted of a pressure tunnel scheme and a free flow tunnel scheme. These pressure tunnel schemes were further subdivided into a pressure tunnel with a free outlet and a pressure tunnel with a submerged outlet. The pressure tunnels flow full and are designed for an internal pressure. The pressure tunnel with a submerged outlet has the crown of the outlet portal always submerged under all flow conditions. This is the most common and widely used type of diversion scheme. The other pressure tunnel with a free outlet has the crown of the outlet portal never submerged under all flow conditions. The free flow tunnel scheme is a scheme where the tunnel flows free and is not designed for internal pressure.

The various tunnel schemes were optimized for optimum tunnel diameter. This necessitated several diameter tunnel sizes be evaluated for costs, associated cofferdam height, and physical layout arrangement.

Layout studies of the project site located the diversion tunnels on the North bank of the Susitna River. The North bank was selected due to ease of access and less length of tunnel required than the South bank. The geotechnical considerations favored the North bank. The South bank would have required the tunnel to pass through a major shear zone. The North bank has ~~the "fins"~~ "The Fins" located near the upstream portal of the tunnel and a shear zone located near the downstream portal of the tunnel. The diversion tunnel can be located between these two features however there is virtually no tolerance or room for adjustment.

4.2 - Hydraulics

Hydraulic studies were carried out to determine the 50 year recurrence period flow. This was established as 83,000 cfs. The initial step estimated the routing effects to reduce the outflow of the diversion tunnels to 76,000 cfs. Hydraulic calculations were then carried out to determine the Headwater/Discharge relationship for the various types and diameters of the tunnels. This is presented as Figure 1, 2, 3, and 4. The tunnel diameters presented in the graph are for a modified horshoe shaped tunnel. The area for this shape of tunnel is 13.7% greater than for a circular tunnel with the same diameter. The tunnel sizes investigated established the maximum velocity under the flood flow in the 50 ft/s to 60 ft/s range. This dictated the requirement of concrete lining. An "n" value of 0.014 was selected for the concrete lined tunnel.

4.3 - Capital Costs

Capital costs were developed for the optimization studies.

(a) Cofferdams

An earth and rockfill cofferdam with a 30 ft crest width, 2H:1V side slopes, and a cross section that has 70% rock or gravel, 10% filter, and 20% impervious material was used for quantity take offs. Several dam heights were selected and layed out. Quantity take offs were carried out and unit costs applied. A graph showing the Dam Elevation/Capital Cost realtionship was prepared and is shown as Figure 5.

(b) Tunnels

Major quantities for the tunnels were calculated for the various diameter tunnels. These included rock excavation, concrete linar, rock bolts, and support steel sets. Unit costs were applied and the capital cost of the tunnels were developed. Portal quantities for the upstream and downstream portals were calculated and consisted of rock excavation and rock bolts for support. These were included in the capital costs for the various tunnel diameters. The capital costs for the various tunnel diameters are presented in Figure 6. The difference in costs for the same diameter tunnel for the pressure vs. free flow tunnel are due to the portal costs. The submerged tunnels are at a lower elevation than the free flow tunnels and require a larger portal, thus a larger cost. The total costs do not include the costs for the intake structures or gates. It is estimated these costs will vary directly with the tunnel diameter and therefore will not effect the optimization analysis.

4.4 - Optimization

The optimization consisted of developing the capital cost of the dam required for the various tunnel diameters. This was accomplished by first selecting a 15 ft freeboard requirement. 5 ft is for settlement and wave run up. 10 ft is for ice. The 10 ft for ice is a provision for possible ice jamming downstream, thus raising the tailwater elevation and subsequent headwater elevation. Using Figure 4, the 15 ft freeboard, requirement and Figure 5, the capital cost of the dam/tunnel diameter relationship was produced and is presented in Figure 6. The total capital costs of the various tunnel diameters were then produced and are presented in Figure 7. This is a composit of the individual costs presented in Figure 6. From Figure 7 it can be seen that 30 ft is the optimum diameter for the ~~free flow~~ pressure tunnel. while 35 ft is the optimum diameter for the free flow tunnel scheme.

4.5 - Cofferdam Closure

An important consideration at this point is the cofferdam closure. For the pressure tunnel scheme this is no problem for the invert of the tunnel is below the riverbed elevation. Once the tunnel is complete the riverflow will divert to the tunnel with only a 10 ft high closure dam. The free flow tunnel scheme however has the tunnel invert some 30 ft above the riverbed. This will necessitate an end dumped closure dam some 50 ft high. There are design problems involved in dumped fill of these heights however it can be done. A more serious problem is the size of particle (boulder) of the dumped fill. As the closure dam crosses the river the water surface increases along with the velocity of the river to be closed. This necessitates large boulders of sufficient weight be and dumped to resist the velocity of the river water. The size of boulders expected to resist these velocities are large (greater than 3 ft diameter) and availability of them is small. Therefore it is impractical to assume closure can be accomplished without major problems for the free flow scheme.
These costs for cofferdam closure have not been included in the costs presented in Fig 5 through Fig 7.

4.6 - Low Level Outlets

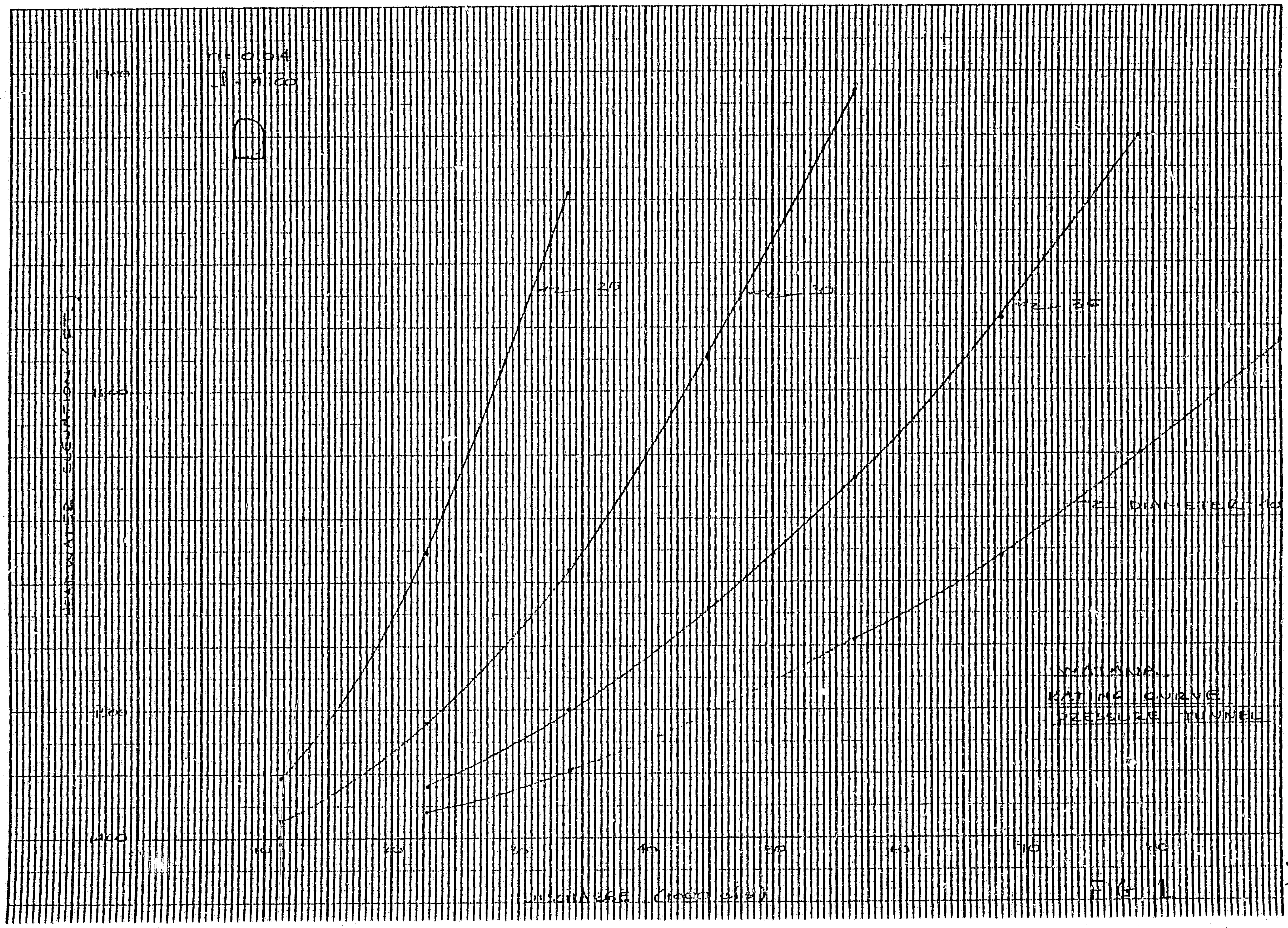
In keeping with current practice it has been decided to provide for draining of the reservoir. The provision of draining of the reservoir is for an extreme emergency case.

Preliminary studies showed the low level outlets with capacity to draw the reservoir down in 4 months (in compliance with Corps of Engineers Regulation No. 1110-2-50) was prohibitavely high.

A capacity of 30,000 cfs at full reservoir ~~pad~~ pool for the low level outlets was arrived at after evaluating the conversion of one diversion tunnel into a permanent low level outlet and in keeping consistent with the project spillways discharge capacities.

Environmental considerations required the low level outlets prevent nitrogen supersaturation of the discharged waters. This requirement necessitated the use of an energy dissipation device. Two alternatives for energy dissipation were investigated. One alternative was Howell Bunger valves and the other alternative was an expansion chamber. The expansion chamber was selected as the preferred alternative. The valve alternative would have rquired a large number and size of valves that were not compatible with the size of the diversion tunnel.

The "sudden expansion" which makes the expansion chamber dissipate energy is a proven scheme that has been used on other projects. The Mica project and the Glen Canyon Dam projects are two ~~is one~~ of the previous projects where this type of dissipation device has worked satisfactorily. One constraint of the dissipation chamber is the requirement that the chamber be located above tailwater in elevation. If the chamber is submerged, cavitation would result in the area of the discharge jet entering the expansion chamber. This constraint necessitated that the diversion tunnel to be converted to a low level outlet be a "free flow" tunnel or a "pressure tunnel with a free outlet" while it is being used as a diversion tunnel.


n = 0.014
RATING CURVE
PRESSURE TUNNEL
FIG. 1

K·E 10 X 10 TO ½ INCH • 10 X 15 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.

47 1320

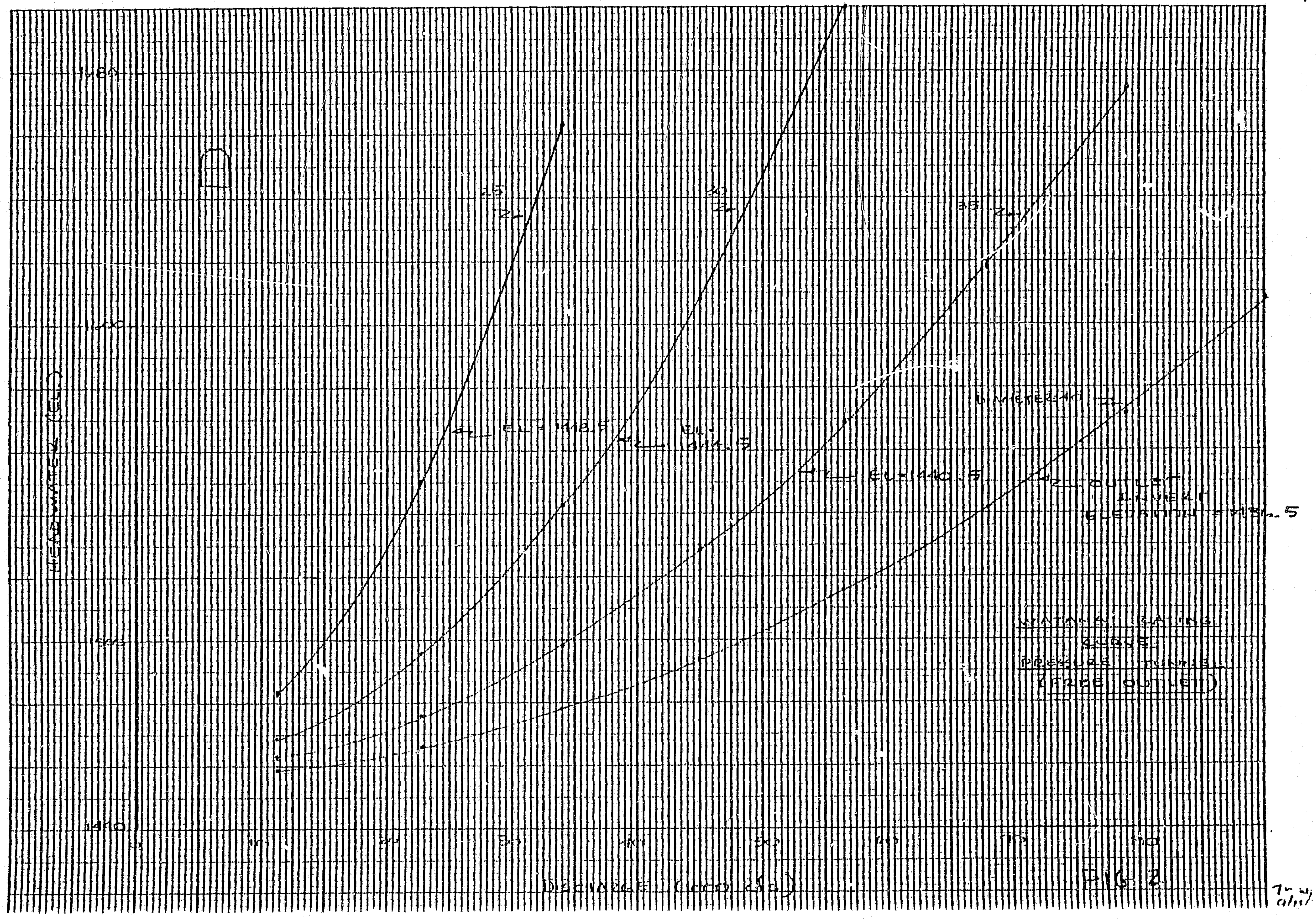


FIG. 2

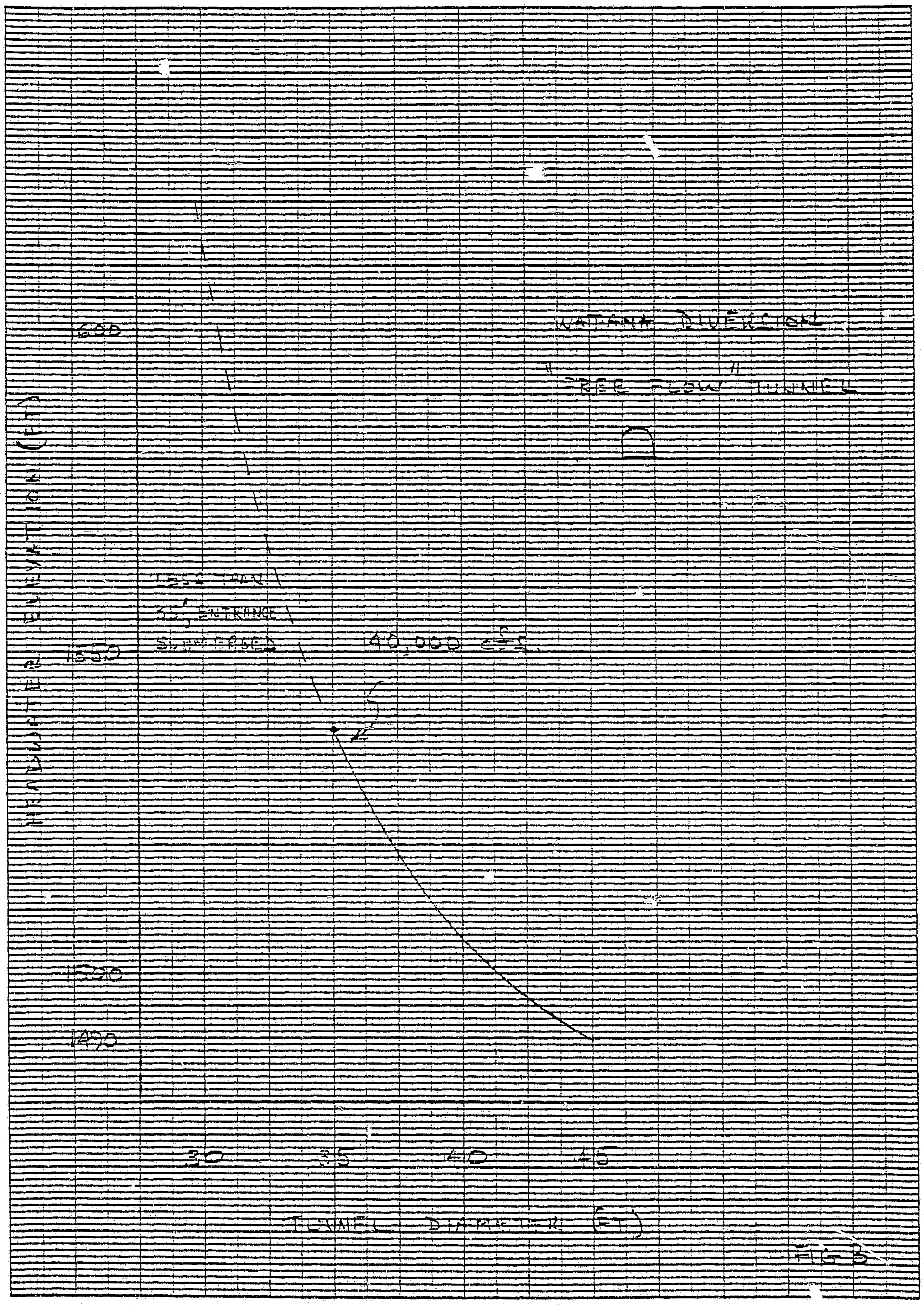


FIG 3

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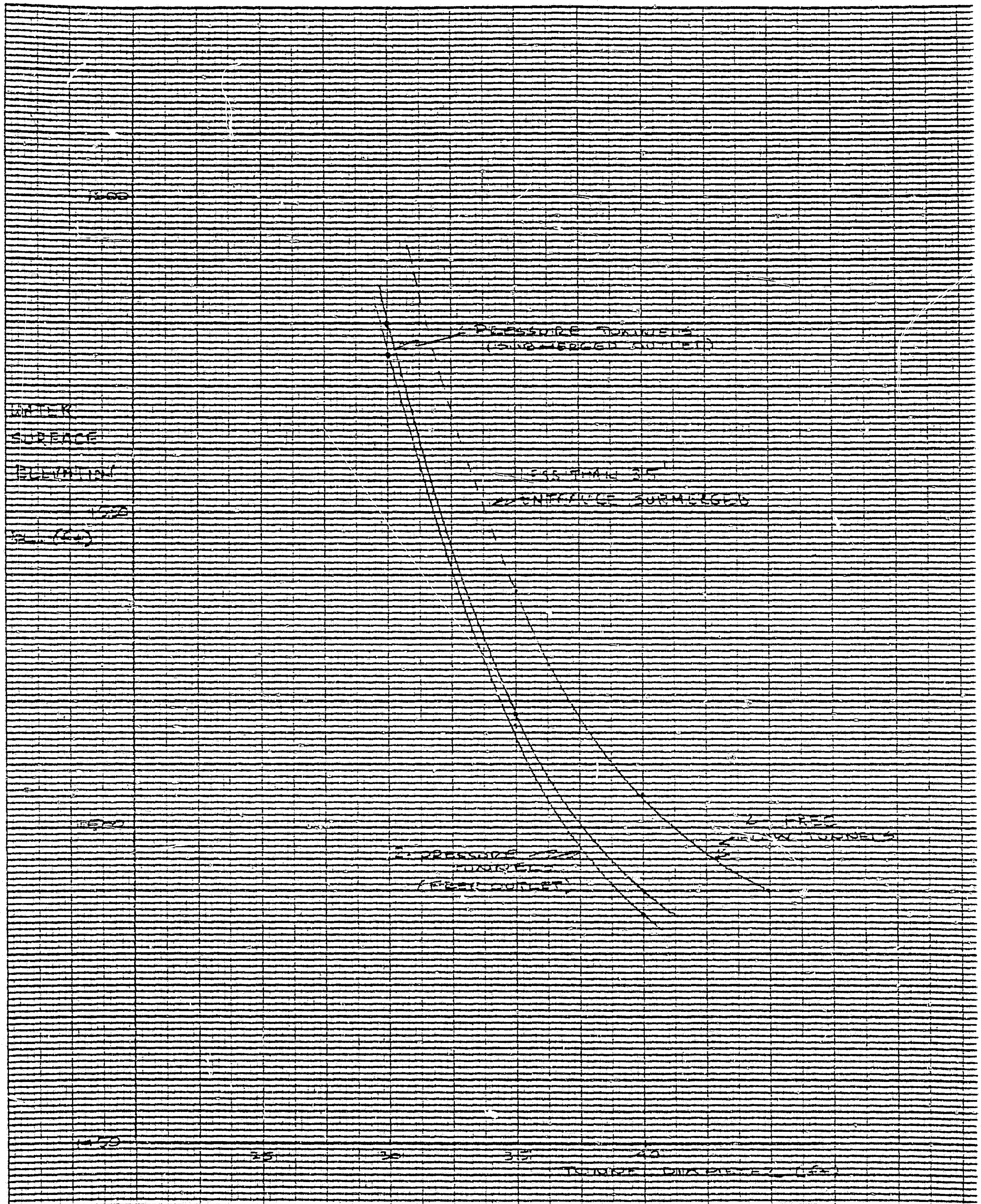
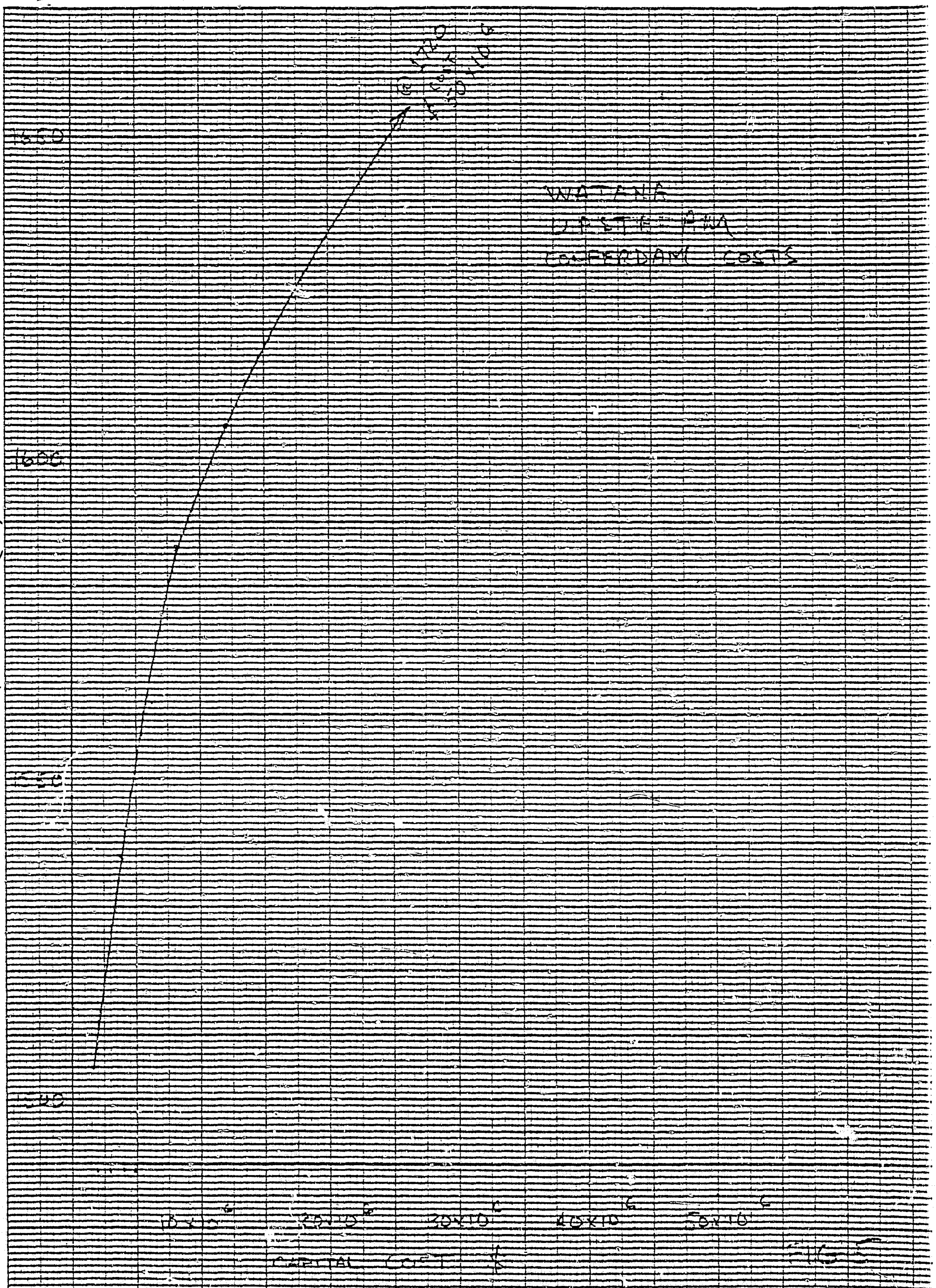


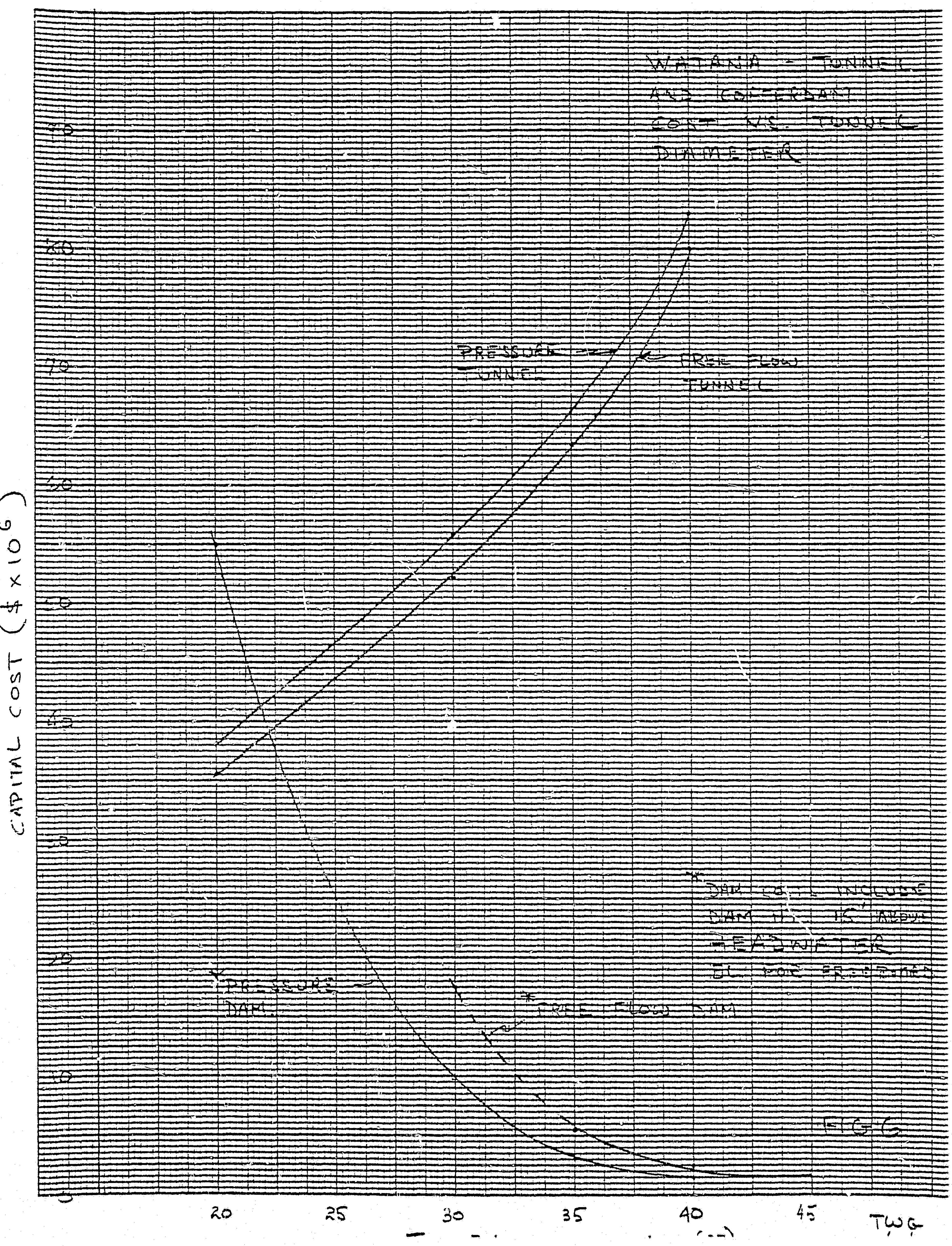
FIG 4

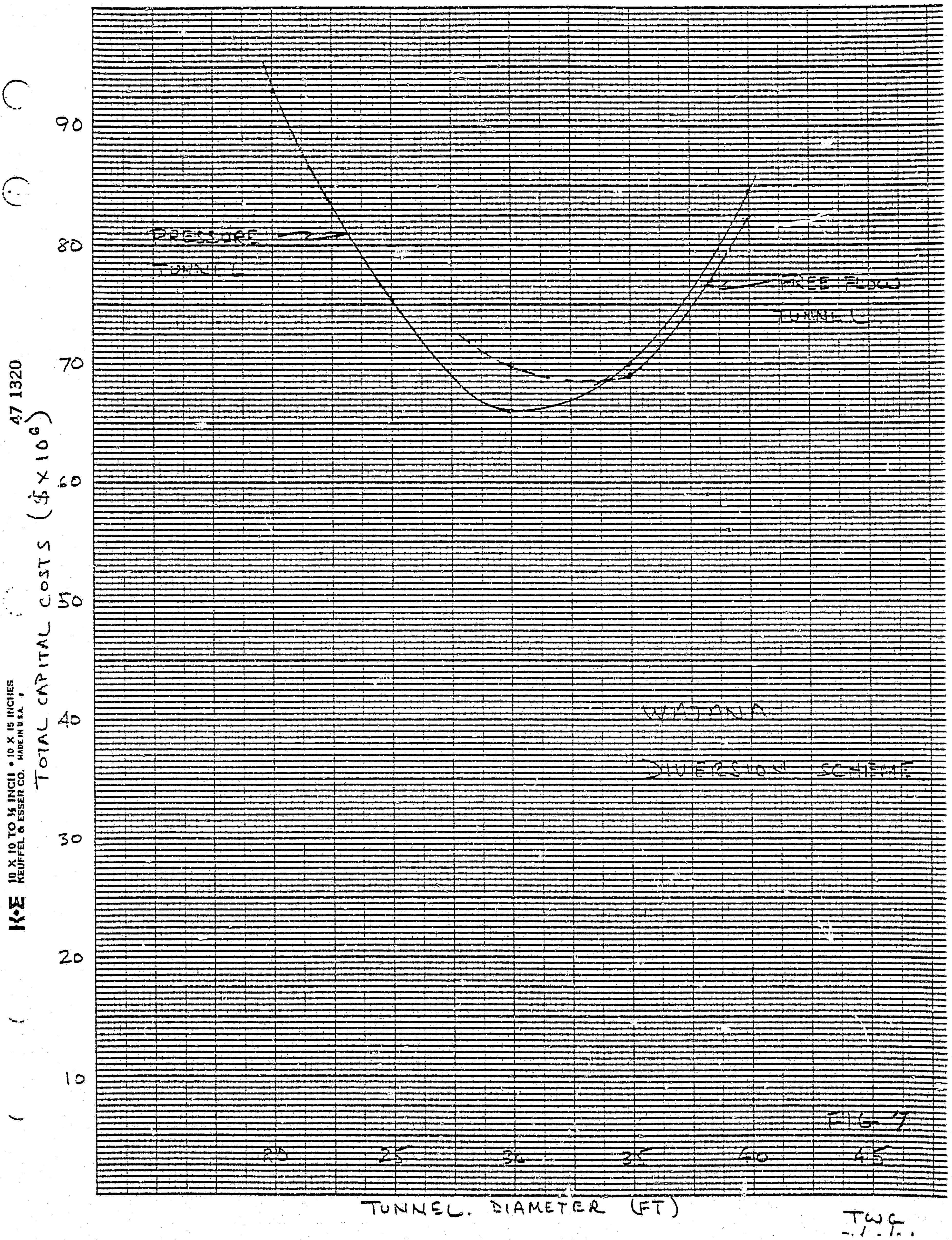
TWG
9/28/61

DAM ELEVATION (FT)



TWG 9/28/81


WATANA - TUNNEL
AND COFFERDAM
COST VS. TUNNEL
DIAMETER
CAPITAL COST ($ x 10^6)
PRESSURE
TUNNEL
FREE FLOW
TUNNEL
* DAM COST INCLUDE
DAM
HEADWATER
* PRESSURE
DAM
* FREE FLOW DAM
20
25
30
35
40
45
TWG
FIG 6



PRESSURE TUNNEL
FREE FLOW TUNNEL
WATANA
DIVERSION SCHEME
FIG 7
TOTAL CAPITAL COSTS ($ x 10^6)
90
80
70
60
50
40
30
20
10
20
25
30
35
40
45
TUNNEL DIAMETER (FT)
K+E 10 X 10 TO ½ INCH • 10 X 15 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.
47 1320

TWC

5 - SELECTION OF THE DIVERSION SCHEME

5.1 - Tunnel Scheme Selection

From Figure 7 the selection of the tunnel scheme would appear to be two - 30 ft diameter pressure tunnels. However, if the pressure tunnels are selected a separate low level release will have to be constructed to avoid the cavitation problems that would occur if one of the pressure tunnels was converted to a low level release. Taking the incorporation of a low level release into consideration, and referring to Figure 7 it would appear the selected scheme would be two - 35 ft diameter free flow tunnels. However with the free flow (or pressure tunnel with a free outlet) tunnels there are major problems with cofferdam closure explained previously in Section 4.5.

The solution, and subsequent selected scheme is a combination one pressure tunnel and one free flow (or pressure tunnel with a free outlet) tunnel. Two 30 ft diameter tunnels were selected and investigated. The cofferdam required will have a crest elevation of 1595. This is about a 150 ft high cofferdam. Layout drawings showed the 150 ft high dam would push the inlet portal upstream and into the middle at "The Fins". This is very undesirable. Subsequent layout drawings and investigations showed two - 35 ft diameter tunnels require a cofferdam with a crest elevation of 1540. This is about a 90 ft high cofferdam. The reduced size cofferdam is preferable over the higher cofferdam and the cost increase is considered insignificant. The reduced size cofferdam also allows the entrance portal of the diversion tunnels to be moved downstream to the edge of "The Fins". For the reasons presented two 35 ft diameter modified horshoe shaped tunnels were selected as the diversion scheme. One tunnel will be submerged and act as a pressure tunnel. The other tunnel will be located at a higher elevation and act as a free flow tunnel. An upper and lower tunnel scheme was used successfully previously in the Oroville Dam project.

5.2 - Operation

As stated previously during diversion one tunnel will-operate as a pressure tunnel at all times. This is the lower tunnel. The other tunnel will operate as a free flow tunnel. This is the upper tunnel. Both tunnels will have operating gates at the entrances to control flow during operation and to enable final closure to take place.

To develop the operations more accurately the discharge rating curve for the selected scheme was developed. This is presented as Figure 8. The routing of the 50 recurrance period flow is presented as a hydrograph in Figure 9.

An important consideration in the operation of the diversion is ice. The ice considerations can be broken down into two major headings. One heading is ice jamming downstream of the diversion tunnels in the open river channel. If this were to occur the tailwater elevation could rise, and thus overtop the downstream cofferdam. By raising the tailwater elevation the headwater elevation would also rise, possibly overtopping the upstream cofferdam. The other heading is ice jamming inside the tunnel, or tunnel entrance, thus blocking the outflow and possibly overtopping the upstream cofferdam.

This potential problem of ice jamming inside the tunnel is eliminated by using the upstream control gates to keep the tunnel entrance submerged during operation, thus eliminating the possibility of ice entering the tunnel. The lower tunnel (pressure tunnel) is always submerged so the gates do not need to be lowered. The upper tunnel (free flow) will not be in use except during the higher flows. It is planned to operate the diversion with the upper tunnel closed at all times. Using only the lower tunnel the reservoir upstream of the cofferdam will be allowed to rise to El 1525. This will discharge approximately 50,000 cfs. This is the _____ year recurrence period flow. For any flow above 50,000 cfs up to 76,000 cfs the upper tunnel gates will be opened, either partially or fully, to pass the required flow. At all times while the upper tunnel is in operation the entrance to the tunnel will be submerged thus preventing ice from possibly entering the tunnel. This may require operating the upstream gates only partially open.

The first heading of potential ice considerations, ice jamming downstream of the tunnels in the open river, was studied and it was found this is not a problem. The stretch of river from the downstream diversion portal to the Devil Canyon site, an approximate 500 ft drop in riverbed elevation was examined for possible ice jamming. It was found during the investigations that stretch of river had no location large enough in surface area where a large ice mass could form, thus fostering the possibility of an ice jam forming.

Another consideration regarding ice during diversion operation that was investigated was the possibility of ice in the reservoir being pushed up against, and possible overtopping the cofferdam. This ~~was~~ may cause erosion and structural damage to the cofferdam. Studies determined that this problem could be eliminated by constructing the dam 10 ft higher or allowing 10 ft additional freeboard for ice buildup or installing an ice boom. From the cost curve presented in Figure 5 an additional 10 ft of freeboard on the dam costs approximately $700,000. It was decided an ice boom can be installed more economically.

The operation of the tunnels during diversion is as stated previously. The lower tunnel will handle the majority of the flows alone. The entrance will be submerged all the time therefore the gates will always be open. The upper tunnel will always be closed until the reservoir reaches elevation 1520. Then the gates should be partially opened half way. From the rating curve presented as Figure 10 it can be seen that this can handle flows up to __________ cfs. If the reservoir continues to rise open the gates fully. If the reservoir elevation falls, close the gates accordingly. The operation of the upper tunnel will operate as a free flow tunnel. The slope is great enough that flow in the tunnel is supercritical and a hydraulic jump will not take place.

After final closure the lowe level outlets will have the capacity to discharge 30,000 cfs at full reservoir elevation of 2020. They will be capable of operating at heads up to 550 ft and capable of withstanding static heads up to 750 ft. The rating curve for the low level outlet discharge is presented in Figure 11.

The reservoir draw down time incorporating the low level outlets is estimated at 14 months and is presented graphically in Figure 12 for various "start" times of the year.

5.3 - Final Closure and Reservoir Filling

Converting one of the diversion tunnels to a low level outlet after using it as a diversion tunnel necessitated a scheme or sequence for final closure.

It was estimated a one year duration is required to construct and install the permanent low level outlets in the existing diversion tunnel. This required the lower or submerged tunnel pass all flows. The lower tunnel can pass 50,000 cfs alone without overtopping the cofferdam. This is a __________ recurrance period flow. Considering the construction duration is only one year, a 10 year recurrence period flow to be handled is consistent with the entire project design flow risks. Therefore the lower tunnel alone is considered to have sufficient capacity to act as the diversion for one year. During the construction of the low level outlets the intake operating gate in the upper tunnel will be closed. The gate and intake structure will be designed for reservoir elevation of 1540 ft which is an external pressure head of 120 ft. Prior to commencing operation of the low level outlets coarse racks will be installed in the upstream intake structure in slots provided.

Upon completion of the low level outlet in the upper tunnel the intake gate will be opened and the low level outlets will commence operation. Upon commencing operation of the low level outlets the lower tunnel will be temporarily closed with the intake gates and construction of the permanent plug will also commence ~~the~~ upon filling of the reservoir. It is estimated it will take a years duration to completely place and cure the plug.

During this time the upstream gate and intake structure will be designed to for a reservoir elevation of 1800 ft. which is an external pressure head of 400 ft.

The filling of the reservoir will take 4 years to complete to full reservoir operating elevation of 2185. After 3 years of filling the reservoir will be at elevation 2150 and will allow operation of power plant to commence.

The filling sequence was determined from the main dam elevation at that time during construction, the starting reservoir pool elevation at that time during construction, and the capability of the reservoir storage to absorb the inflow volume from a 500 year recurrence period inflow without overtopping the main dam. The 500 year recurrence period flood volume was selected to be consistant with the recurrance period flows and risks used for the design of the diversion and entire project.

This information is presented graphically as Figure 13.

5.4 - Continuing Studies and Recommendations

Subsequent studies have shown the structural design of the tunnels to be better accomodated if the tunnels are circular in diameter. Due to the tunnel size optimization determining a 35 ft diameter modified horshoe shaped tunnel to be the optimum size, this equates to a 38 ft diameter circular shaped tunnel section.

Layout studies of the energy dissipation chamber in the low level outlet necessitate the chamber be 45 ft diameter to physically accomodate the gates and passages required for the chamber. It is necessary to construct the larger diameter tunnel section in the initial construction of the diversion tunnel rather than afterwards which would require removal of the tunnel lining and enlarging.

Studies conducted or constructing the lower tunnel showed that leaving an "original rock cofferdam" in place and removing it prior to commencement of operation of the diversion tunnel, is economically advantageous over constructing a sheet pile cofferdam for construction of the portal and the tunnels.