OPTIMISATION, DESIGN AND CONSTRUCTION
OF
INTAKE STRUCTURE

Claudio R. Vissa

Major Technical Report
in
The Department
of
Civil Engineering

Presented in Partial Fulfillment of the Requirements for the Degree of Master of Engineering at:

Concordia University,
Montreal, Quebec, Canada.

October 1979

ABSTRACT

Optimisation, Design and Construction of Intake Structure

Claudio R. Vissa

This report outlines the main activities leading to the construction of the LG-4 intake structure. A quantitative analysis is given which leads to the choice of the layout of the water intake. The design of the partially submerged reinforced concrete water intake towers is presented with particular reference to the impact forces of large blocks of ice floating on the reservoir. Reference is made to the hydraulic model study which dictated the perimetrical shape of the towers and walls. The critical construction concepts are outlined from foundation treatment to concreting of the intake towers. Basic planning and an execution schedule are also presented.

RESUME

Optimisation, Conception et Construction d'une Prise d'Eau

Cette étude donne un aperçu des points saillants nécessaires à la réalisation de la construction de la structure de la prise d'eau de LG-4. Une analyse quantitative est donnée, qui porte au choix de l'aménagement de la prise d'eau. Le calcul de la tour en béton armé, partiellement summergée dans l'eau, est presenté avec une réference particulière à l'impact de force des blocs de glace flottants sur le réservoir. Un aperçu de l'étude sur le model qui a dicté la forme des tours et des murs bajoyers est inclus. Certains concepts de construction du traitement de la fondation au bétonnage des tours de la prise d'eau sont notés. Une planification et un échéancier pour la construction sont aussi présentés.

ACKNOWLEDGEMENTS

The author wishes to express his sincere thanks to his supervisor, Dr. O.A. Pekau Eng. of Concordia University for his help and guidance in the completion of this major technical report.

Thanks are also extended to Mr. M.A. Anton Eng., Dr. T. Barutciski Eng., and Mr. A.D. McConnell Eng., of Rousseau, Sauve, and Warren, and not least his many colleagues on the LG-4 project for their encouragement and valuable assistance during the preparation of this report.

Sincere appreciation is extended to Mr. P.S. Harel Eng., Project Engineer of LG-4 at the "Societe d'Energie de la Baie James", and the S.E.B.J. for their permission to use some of their documents in this work.

Many thanks are due to his wife, Giovanna, for her great patience in typing this report.

Finally, the author would like to express his deepest appreciation and thanks to his wife and family for their encouragement and patience during these long years of schooling.

LIST OF FIGURES

LIST OF SYMBOLS

Symbols are defined in the text. Below is a resume of those symbols used in the text.

A	=	surface area of foundation
b	=	projected width of structure in the direction of ice motion
C	=	unit cohesion
E	=	weight of water
E_1	=	hydrostatic pressure
f'_c	=	compressive strength of concrete at 91 days
f_y	=	yield stress of steel
F_i	=	ice impact force
G	=	ice forces in most unfavorable direction
g	=	gravitational acceleration
h	=	ice thickness
h_s	=	height of backfill
H	=	summation of horizontal forces
H_e	=	hydrostatic head
K	=	contact coefficient in percentage
M	=	shape factor
M_o	=	summation of overturning moments
M_r	=	summation of resisting moments
P	=	weight of concrete mass
P_o	=	backfill pressure
Q	=	shear friction safety factor
S	=	weight of backfill

S.F.	=	safety factor
T	=	compressive strength of ice
ΣU	=	summation of uplift forces
ΣV	=	summation of vertical forces
W	=	snow load on ground
β	=	slope of backfill

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 General

Situated in the James Bay region, the La Grande Complex, the biggest hydro-electric project in the world, is scheduled for a two-phase construction. LG-2, LG-3 and LG-4 (La Grande Four) are the three major power plants constituting the first phase.

With installed capacity of 2,650 Mega-Watts (9 generating units) LG-4 will produce 30% of the total energy output of the first phase.

Fig. 1 is a map of the La Grande Complex in Northern Quebec which outlines the relative position of each site. Fig. 2 shows the installation of the LG-4 power plant, the dam, the spillway, the dykes, the temporary diversion tunnel and the intake - penstock - powerhouse system, including the forebay and tailrace channels. A cross section of the intake and the powerhouse of LG-4 is shown on Fig. 3.

1.2 Intake Structure

The function of the intake structure is to lead the water from the reservoir into the penstock such that head losses are minimized, and air entrainment due to vortex action does not occur. The intake is fitted with a trash rack to prevent the passage of floating debris, and a gate to permit interruption of flow in emergencies or when dewatering the penstock for maintenance or inspection.

For LG-4 there is one intake passage and penstock per

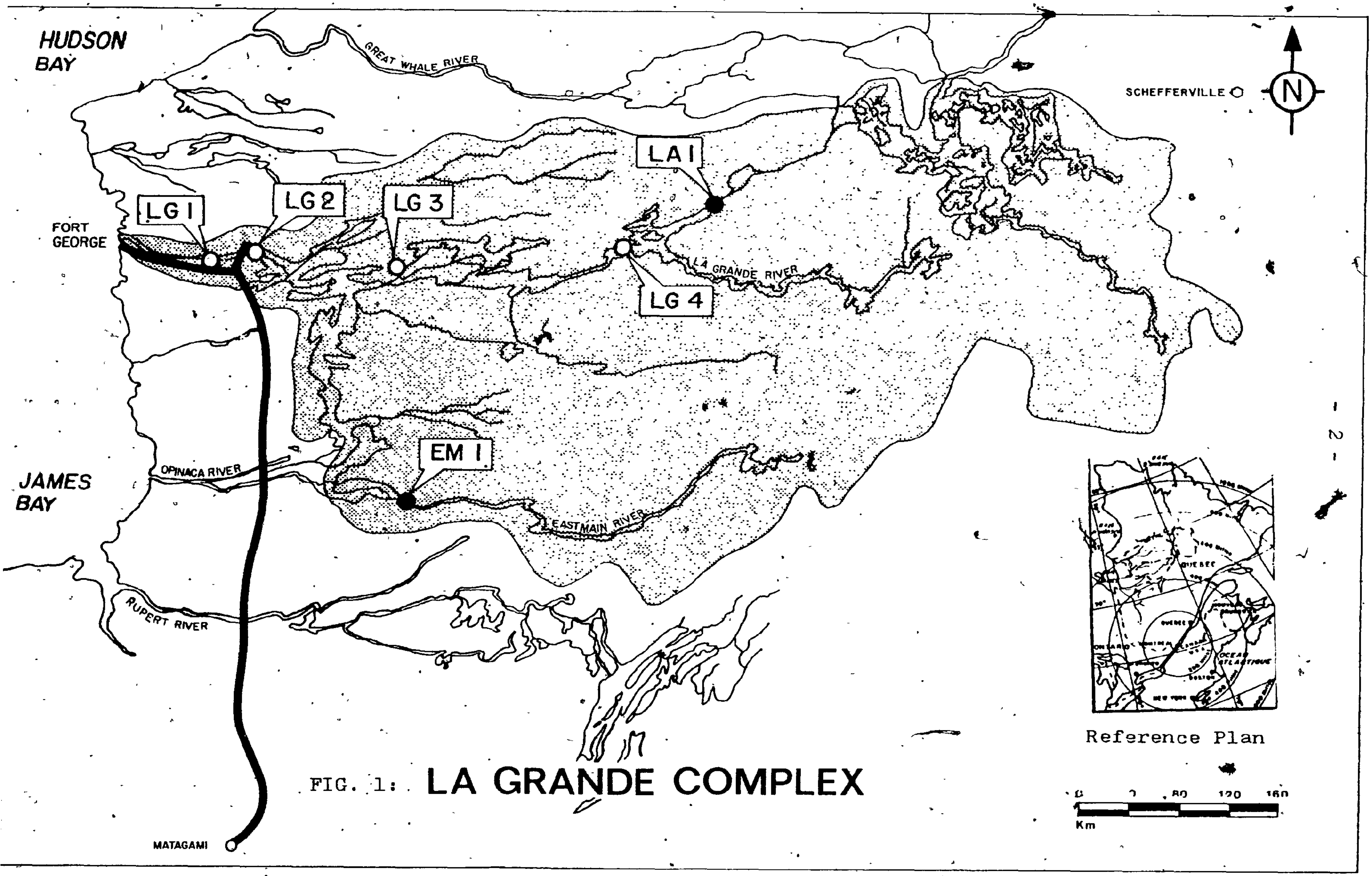


FIG. 1: LA GRANDE COMPLEX

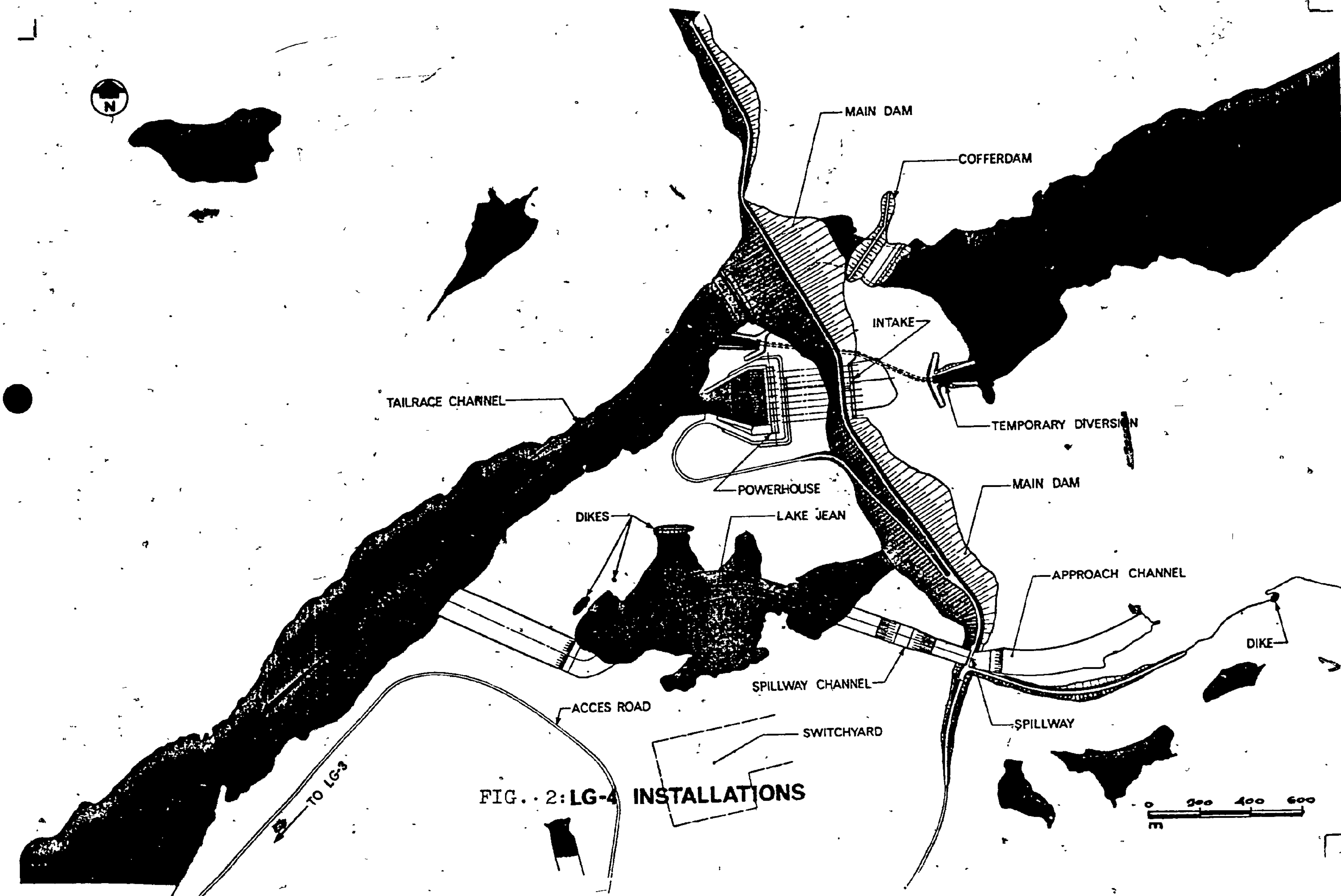


FIG. 2: LG-4 INSTALLATIONS

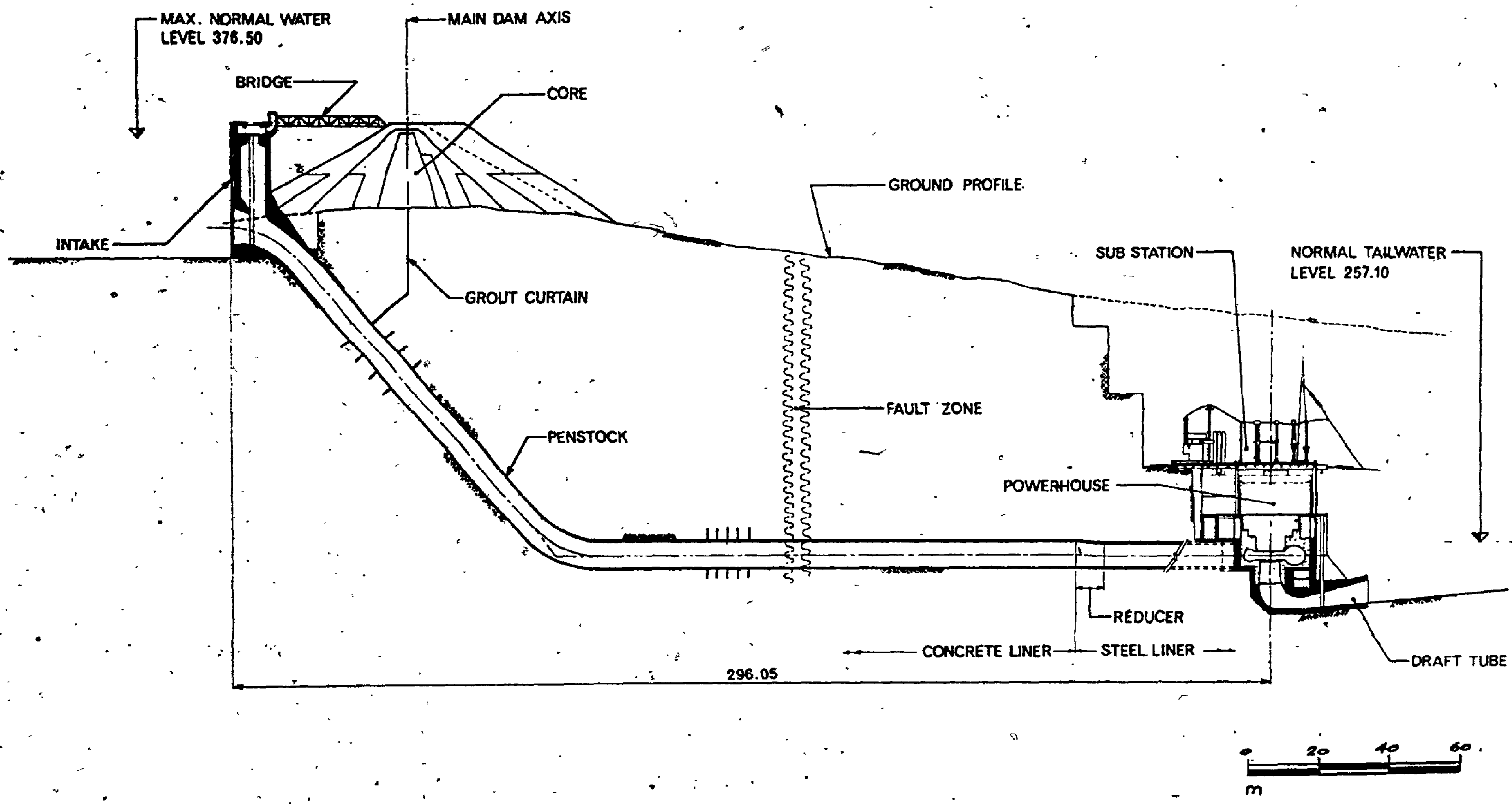


FIG. 3: **CROSS-SECTION POWER INSTALLATIONS**

generating unit. The structure thus consists of a group of nine adjacent towers with a common bridge deck.

1.3 Scope of the Report

The scope of this report is to outline the basic engineering design for the building of the water intake of the LG-4 hydro-electric power plant.

The present report includes:

1. The economical analysis of the location of the intake tower in relation to the axis of the dam.
2. The design considerations of the intake tower.
3. Some practical aspects of its construction.

1.4 Site Geology

The predominant underlying rocks in the LG-4 area are granites, gneisses, and migmatites with the regional foliation tending north-east. The south bank of the river of the dam site is formed by a large rounded hill covered by a thin layer of till and frequent outcrops of rock. The intake tower is situated on the upstream side of this hill and the underlying rock is mainly granite.

1.5 Components of the Water Intake Structure

The main parts of the water intake structure are shown on Fig. 4. These are:

(i) The water passage is a bell-mouth opening and transition which directs the water to the penstocks.

(ii) The trash rack is the steel grillage which

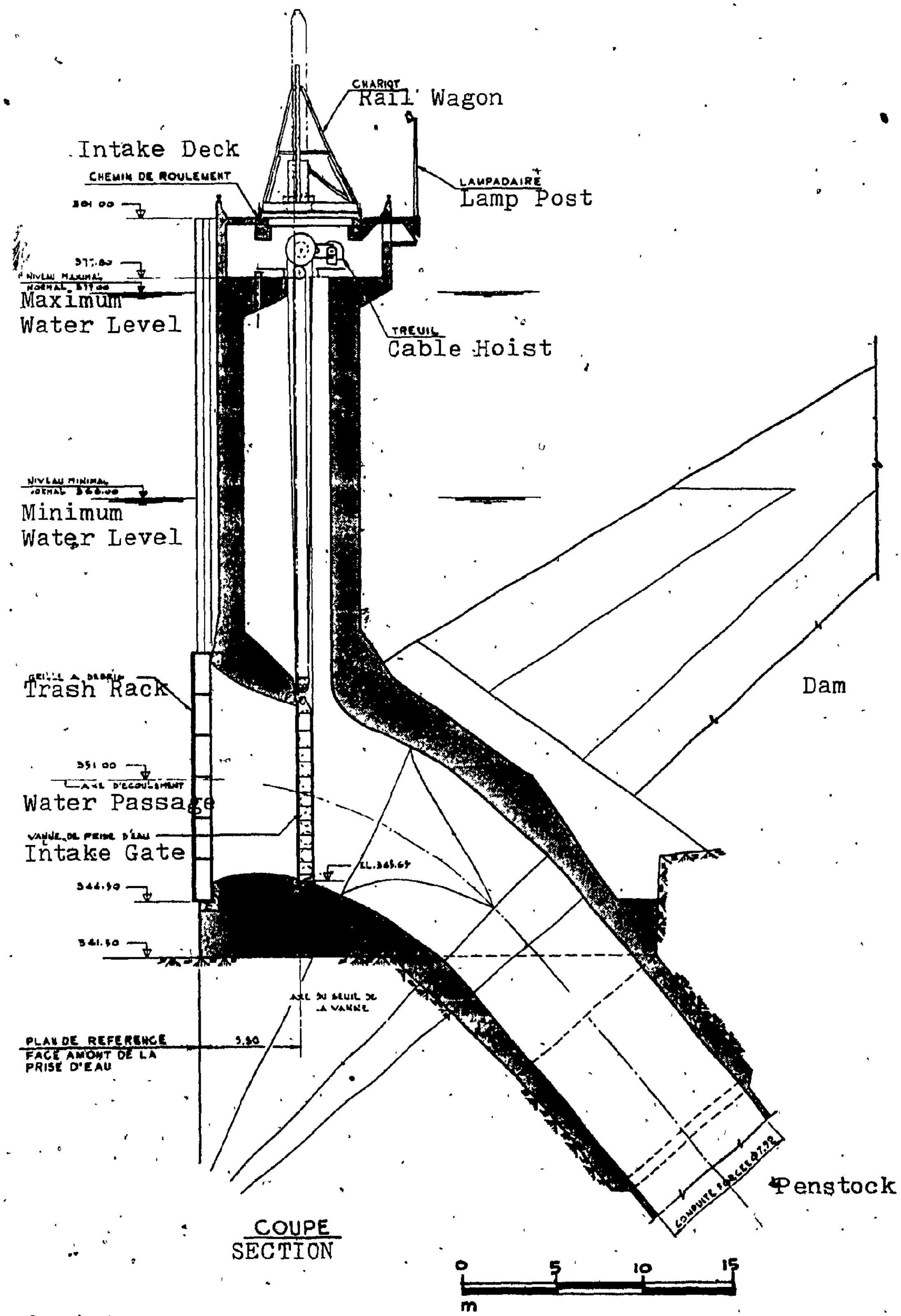


FIG. 4: Components of the water intake structure.

prevents debris from entering the water passage. It consists of four super-imposed sections housed in slots on the upstream face of each tower. The trash rack slots are also used to accommodate stoplogs. The latter are used for closing the structure for inspection and maintenance of the intake tower and the embedded gate guides.

(iii) Intake gate is placed at approximately the center of the tower and is designed to close against maximum flow. This wheel-mounted gate has a seal around the upstream perimeter and closes on a seal beam, located along the sill of the water passage. The gate is lifted by a cable hoist, located at the upper level of the shaft.

(iv) Intake deck is designed to accommodate a mobile crane and a rail wagon for handling the trash racks, stoplogs, the gates and hoists.

1.6 Types of Intakes

In the course of the conception of an Intake Structure, many arrangements have been considered. Each arrangement has its advantages and disadvantages, depending on the parameters involved for a particular site. The parameters mostly considered are topography, geological conditions and technical and economic considerations.

The magnitude of the works and the position of the intake, in relation to the other works, affect the choice. Some of the types of intakes [1]* used in hydro-electric projects are:

(1) Tower intake separated from the dam with a top peripheral inlet and cylinder gate;

(2) Tower or shaft constructed within the embankment or placed upstream from the dam;

(3) Integral intake powerhouse structure, usually straddling the river bed;

(4) Intake built on the sloping face of the dam;

(5) Intake excavated at the side of a hill and separated from the dam with trashrack and stop-logs at the inlet and with the gates housed in the vertical shaft excavated in the rock, down-stream from the inlet.

The type of intake considered most appropriate for LG-4 and outlined in this report is of the second type described above.

---

* Numbers in brackets refer to the list of references given at the end of the report.

2.0 CHOICE OF POSITION AND COST ANALYSIS OF THE WATER INTAKE

2.1 Introduction

For cost analysis, a number of alternatives were considered. The components of each alternative were studied and assessed for feasibility and technical aspects. Then, the total costs of the alternatives were compared singling out the optimum.

A water intake structure and penstock may serve to feed one or more machines. In the case where one water intake structure serves more than one turbine, a manifold is used to distribute the water flow from the penstock into the turbines. This latter arrangement necessitates a valve at each turbine inlet to permit inspection and maintenance of each turbine, without having to shut down the others. However an intake gate is required on each penstock in any case. The choice between the two arrangements is a matter of cost.

2.2 Position of the Intake

In the case of LG-4 [2] three positions of the intake were investigated with respect to the axis of the dam:

(1) Fig. 5(a) depicts an intake structure co-axial with the dam;

(2) Fig. 5(b) indicates an intake structure integrated in the dam, with the core on the downstream side of the structure.

(3) Fig. 5(c) outlines an independent water intake structure upstream of the dam core and filter.

The first two positions have been used on LG-2 and LG-3 projects, respectively, of the James Bay complex. Although those sites do not possess the same dam layout and topography as LG-4, intakes of similar arrangements could not be disregarded as potentially viable alternatives.

It remains to study the relative engineering and economical advantages of the various positions of the intake structure. The penstocks of the third alternative are 50 metres longer than those of the first two. It was feared that this additional length could create an unacceptable pressure in the units or excessive speed rise at shutdown. In such cases, a surge chamber is essential between the intake and the powerhouse. The surge chamber dissipates most of the energy created by the back wave occurring during the shutdown of any group [12].

An investigation in collaboration with a turbine manufacturer, however, indicated that the length of the penstocks was within acceptable limits; i.e., the use of a surge chamber was unnecessary.

2.3 Description of the Positions

The structure illustrated on Fig. 5(a) represents position P_1. This position of the intake creates a discontinuity in the dam core. Cut-off walls at each end are necessary to serve as abutments for the core and transition zone of the dam. These walls are designed in sections so as to prevent movements which create weaknesses in the core material.

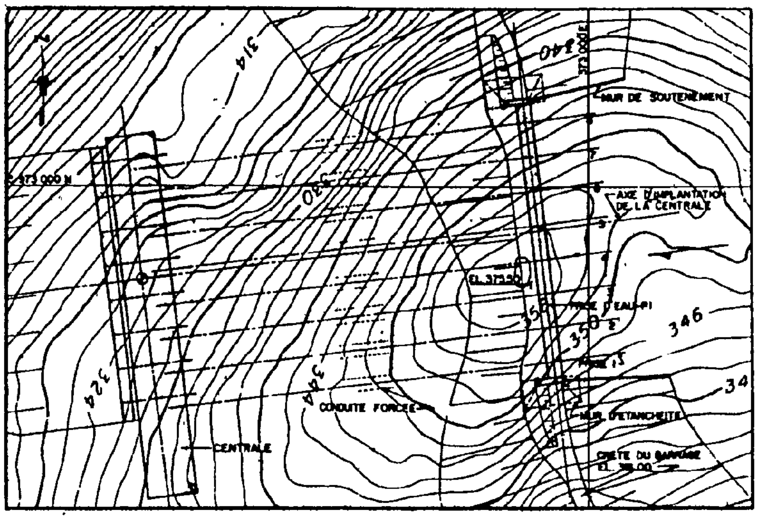
Fig. 5(b) outlines the second position P_2. In this case, the downstream end of the intake structure does not encroach on the dam core. This eliminates the need for cut-off walls. The space between the intake and the dam is backfilled to provide access to the intake deck. To minimize the deflection of the structure subjected to the pressure of the dam fill, the stiffness of the intake had to be increased. Movements of the intake structure have to be minimized, in order to ensure continuous confinement of the core material and to prevent it from cracking. The latter induces particle migration and washouts towards the downstream direction of the dam.

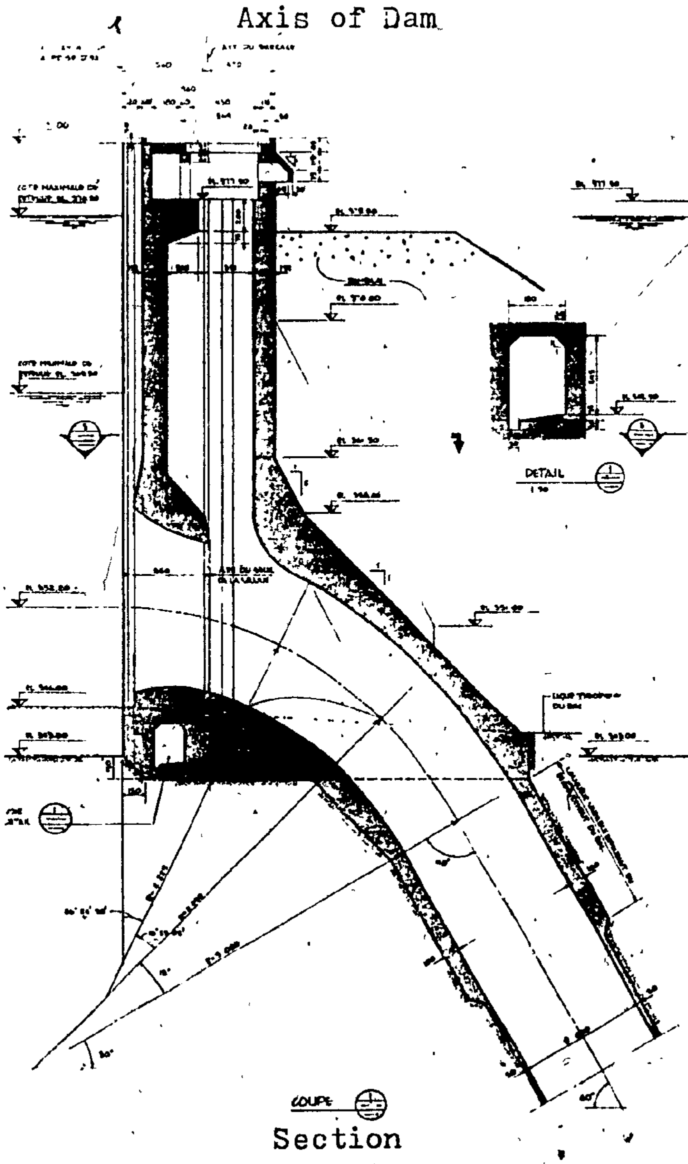
Fig. 5(c) shows the layout of the third position P_3. This position of the intake was influenced by ideal geotechnical criteria which call for the core and the filter to rest on a rigid base, thereby avoiding any interference from the intake structure. The intake was displaced far enough upstream to disengage it from both the filter and the core of the dam, but this positioning required an access bridge between the dam and the intake deck.

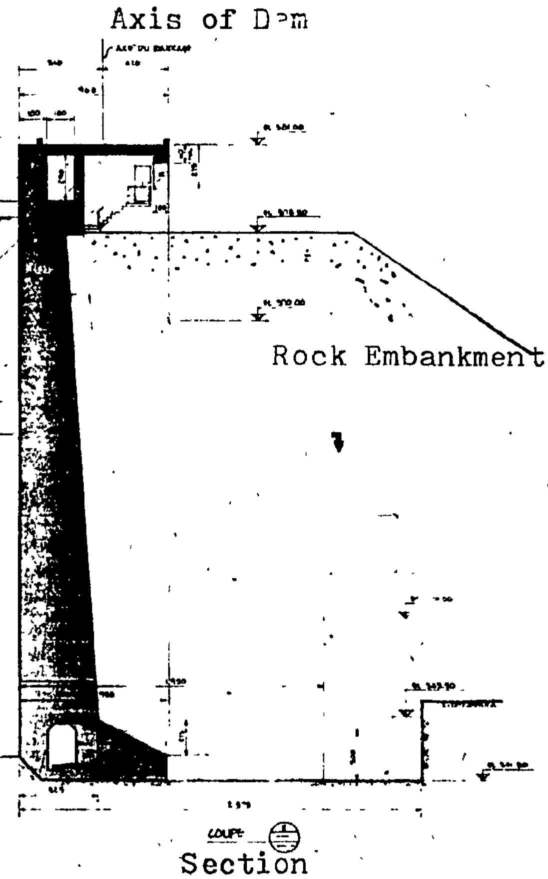
All three positions require abutment or lateral walls to retain some portion of the dam. The heights and lengths of the walls are directly related to the position of the intake with respect to the axis of the dam.

2.4 Stability Analysis

The basic criterion in designing a structure is that it must withstand the loads exerted on it. Should the







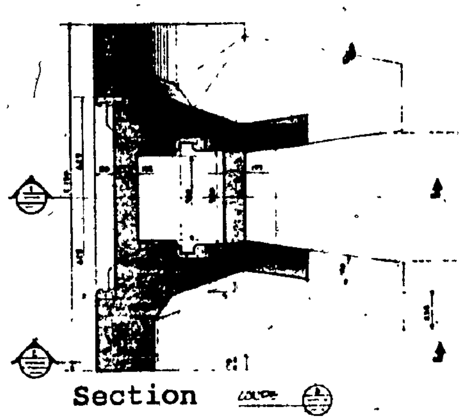
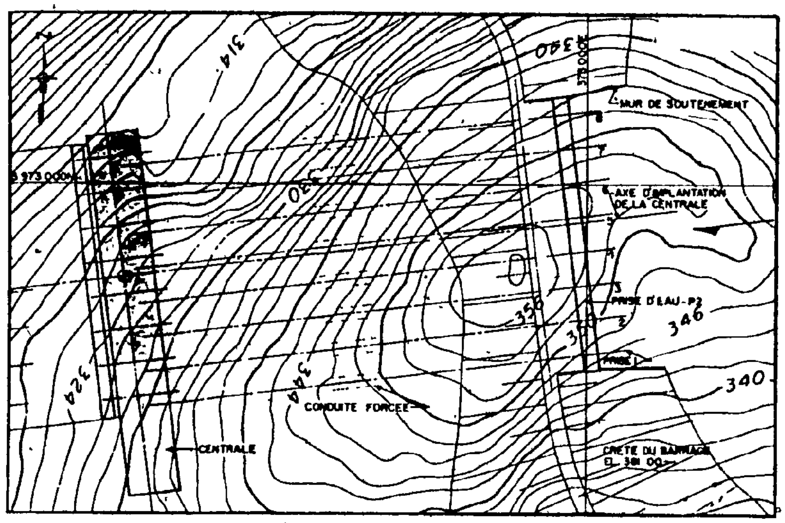
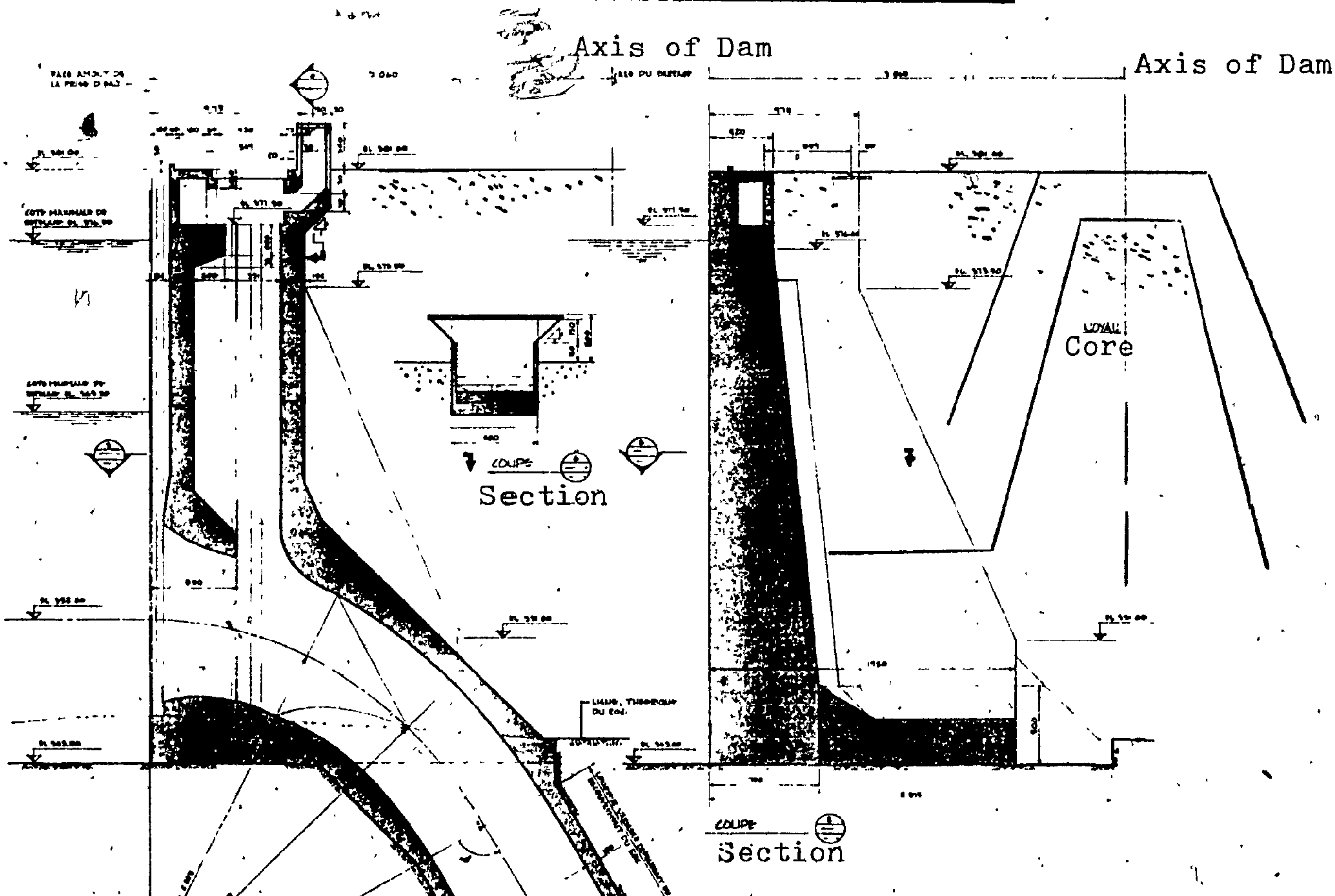


FIG. 5(a): Intake co-axial
with the dam





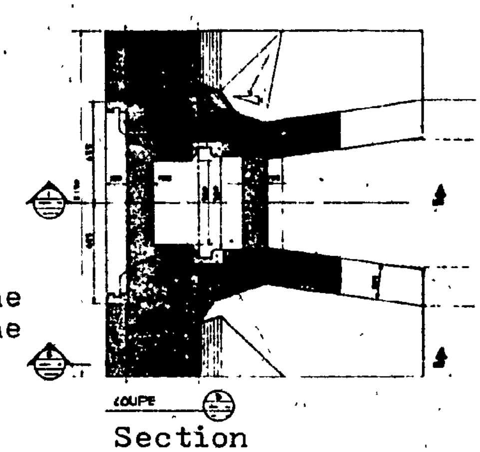
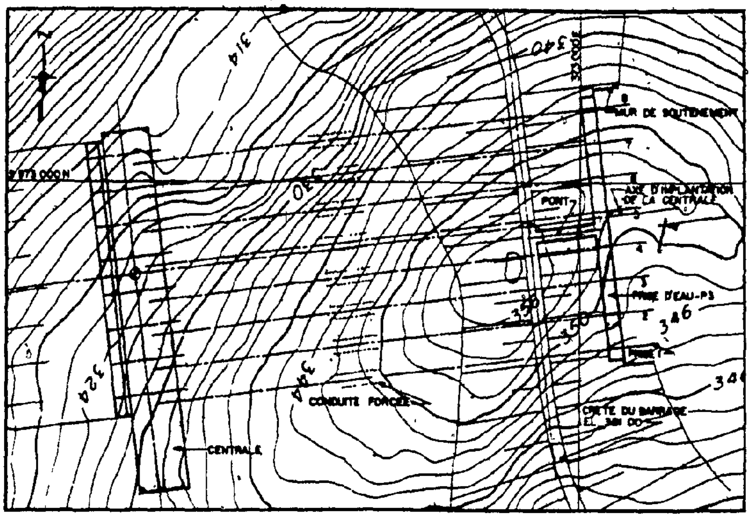
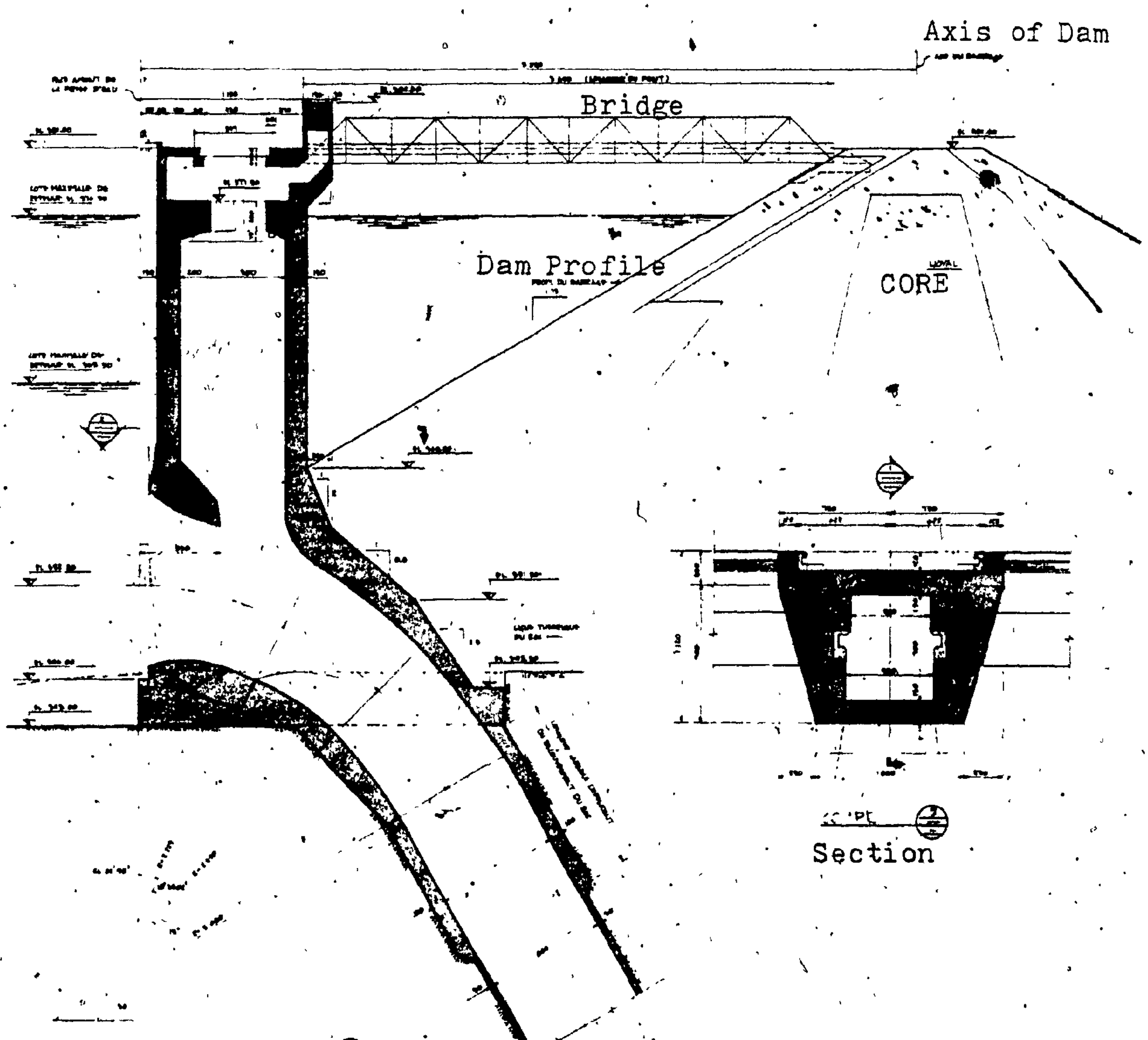


FIG. 5(b): Intake integrated in the dam with the core on the downstream side of the structure.





Section

FIG. 5(c): Intake Structure upstream of the dam core and filter.

intake structure not be stable against overturning, sliding or uplift, concrete is added and/or the form of the structure is altered, taking care to preserve its hydraulic efficiency.

In the cases P_1 and P_2 where, as shown in Fig. 5(a) and Fig. 5(b), heavy backfill loads are applied to the structure, additional concrete was necessary to render them stable.

Fig. 6 illustrates the procedure adopted for the selection and optimization of the proper type and position for the intake structure for this particular project.

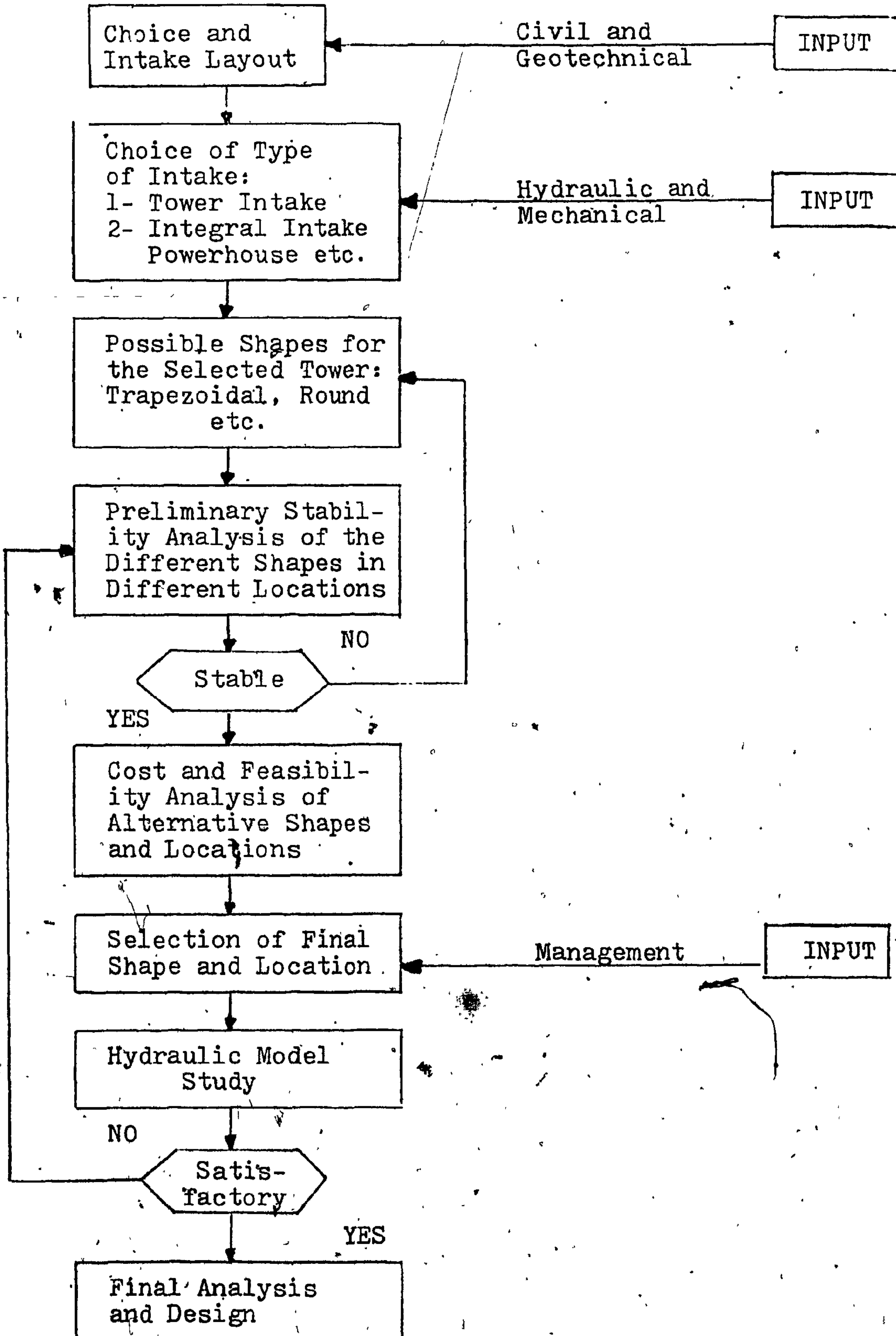


FIG. 6: Flow Chart - procedure for selection of intake structure -

2.5 Cost Analysis

Table 1 [2] summarizes an estimate of civil, hydraulic and geotechnical costs of the three alternatives, P_1, P_2 and P_3.

The mechanical and electrical equipment are omitted from the estimation because they are common to all the alternatives. The estimate includes the cost due to the differential length of the penstocks and the relative value of the head loss.

The cost comparison shows that Alternative P_3 is the least expensive. Based on this and on technical and practical advantages, the management of the project consented to a model study of Alternative P_3.

TABLE I

COST ANALYSIS

ITEMS	UNITS	Alternative P^1		Alternative P^2		Alternative P^3	
		Quantities	Cost $x10^3	Quantities	Cost $X10^3	Quantities	Cost $X10^3
Access Bridge							
- Concrete	m3	0	0	0	0	220	83
- Structural Steel	t	0	0	0	0	220	627
Intake Deck							
- Concrete	m3	880	259	600	176	420	159
- Structural Steel	t	32	91	32	91	132	376
Intake Tower							
Concrete							
- above el.: 377.5	m3	465	137	900	265	950	359
- from el.: 358.5 to 377.5	m3	8040	1914	9600	2285	12300	3223
- below el.: 358.5	m3	9200	2346	9900	2525	12100	3086
- transition	m3	5600	1428	5200	1326	5000	1275
- counterforts	m3	1900	460	2700	624	0	0
- tower wing walls	m3	14500	2617	19800	3335	0	0
Lateral Walls							
Concrete							
- wing walls	m3	14000	2324	12000	1992	2500	810
- cut off walls	m3	30000	4470	0	0	0	0
Reinforcing Steel	t	3980	3781	4200	3990	3200	3040
Cement	t	27000	3391	21000	2638	12500	1570

TABLE I (continued)

COST ANALYSIS

ITEMS	UNITS	Alternative P^1		Alternative P^2		Alternative P^3	
		Quantities	Cost $X10^3	Quantities	Cost $X10^3	Quantities	Cost $X10^3
Rock Excavations of Forebay Channel	m^3	17200	1333	140000	1085	100000	775
Dam							
Dam Material							
- rock	m^3	130000	553	70000	298	125000	531
- core material	m^3	0	0	75000	326	70000	305
- filters	m^3	0	0	52000	273	55000	289
- sand and gravel	m^3	0	0	103000	541	70000	368
- rip-rap	m^3	0	0	0	0	12000	108
Penstocks (differential)							
- excavation	m^3	0	0	12500	750	23000	1380
- concrete	m^3	0	0	2400	833	4250	1475
Head Losses (differential)	M	0	0	0.0353	373	0.0648	685
PARTIAL Cost			25104		23725		20524
DIFFERENTIAL Cost			4580		3201		0

3.0 DESIGN CONSIDERATIONS

3.1 Introduction

Design of the intake should consider:

(i) Structural soundness

(ii) Performance

(iii) Economy

(iv) Head losses due to:
- (a) Trash rack
- (b) Entrance
- (c) Gates

(v) Submergence of the intake and configuration of the structure and approach channel insofar as vortex action is concerned.

The latter two functions are optimized in a hydraulic laboratory. Often, vortices are observed due to inadequate submergence, the entrance shape, the overall structural shape or a combination of the three. These are corrected by altering the overall geometry of the structure, the setting of the structure and/or the bell-mouth shaped entrance of the water intake.

3.2 Design Criteria

3.2.1 Codes and Standards

The National Building Code (N.B.C.) [3] and the United States Bureau of Reclamation (U.S.B.R.) Standards [1] are used simultaneously in hydraulic works. Design assumption made during the conceptual phase of these projects also considers recorded performance data for the Northern Quebec region.

3.2.2 Construction Materials

The properties of materials to be used to build the intake towers are:

- Concrete:
  - (i) Structural: f'_c = 30 MPa
  - (ii) Blockout: f'_c = 35 MPa
- Reinforcing Steel Bars:
  - (i) From 10 to 15 sizes: f_y = 300 MPa
  - (ii) From 20 to 45 sizes: f_y = 400 MPa
- Aggregate:
  - Granite: Specific Gravity = 2.7

3.2.3 Loads

The applied loads are:

- Ice Pressure:
  - (i) Static:
    - Side: = 450 kN/m
    - Upstream Face: = 150 kN/m
    - Downstream Face: = 300 kN/m
  - (ii) Dynamic Upstream Force: = 750 kN/~~m~~
- Water:
  - (i) Hydrostatic pressure = 10 $kN/m^3 x H_e$
  - (ii) Uplift buoyancy pressure = 10 $kN/m^3 x H_e$
- Earth Pressure:
  - (i) Dam fill
    - (a) Specific Gravity: = 2.0
    - (b) Coefficient of Pressure at rest: = 0.41
- Reference wind speed: = 110 Km/hr
- Ground snow load: = 4.8 kPa
- Gravitational acceleration: = 981 cm/s^2
- Seismic factor zone: = 1

3.2.4 Stability Analysis

Because the intake structure rests on a hill top, the towers at the south end are founded on rock at a lower level than those in the center and on the north side. To correct these differences, mass concrete is added beneath the southern towers to elevate the foundation to the level of the remaining towers. This mass concrete is considered to act as an extension of the rock foundation and this is omitted from the stability analysis of the tower.

3.2.4.1 Stability Against Overturning

The safety factor against overturning is calculated by:

$$S.F. = \frac{\Sigma Mr}{\Sigma Mo} \qquad \begin{array}{l} S.F. \geqslant 1.2 \text{ for normal cases} \\ S.F. \geqslant 1.1 \text{ for extreme cases} \end{array}$$

where ΣMr = summation of resisting moments

and ΣMo = summation of overturning moments.

3.2.4.2 Stability Against Sliding

The resistance to sliding [1] or shearing at any level of the structure is computed from the equation:

$$Q = CA + \frac{(\Sigma V + \Sigma U)}{\Sigma H} \tan \phi$$

$Q \geqslant 3$ for normal cases

$Q \geqslant 1$ for extreme cases

where Q = shear friction safety factor;

C = unit cohesion (for LG-4 = 0.7 MPa-concrete on excavated rock surface); A = surface area of the foundation; ΣV = vertical forces; ΣU = uplift forces; ΣH = horizontal forces; $\tan \phi$ =

coefficient of internal friction (0.75 for concrete on concrete or concrete on rock).

3.2.4.3 Stability Against Buoyancy

In case the gates are closed and the tower is dewatered, the gravitational mass of the structure must exceed the buoyancy forces under the foundation. The safety factor against floating [1] is computed from:

$$S.F. = \frac{\Sigma V}{\Sigma U}$$

$$S.F. \geq 1.2 \text{ for normal cases}$$

$$S.F. \geq 1.1 \text{ for extreme cases}$$

3.2.4.4 General Considerations

At certain sections the stresses are relatively low compared to the strength of the structure. Such sections are designed to satisfy the minimum requirements such as temperature and shrinkage as well as minimum reinforcement required by CSA standards CAN3-A23.3-M77 [4]. Based on the fact that the rock foundation under consideration has a much higher compressive strength than the concrete, design calculations have been limited to concrete only.

3.3 Design

3.3.1 Introduction

Once a viable water intake structure is established and before the detailed design commences, a model is constructed to test the hydraulic behaviour.

The water intake tower is shaped for structural efficiency; it must withstand the applied forces at any position of its height. In turn, if any part of the structure needs re-dimensioning for strength, hydraulic capacity and efficiency must be maintained.

3.3.2 Hydraulic Model Study

The purpose of the hydraulic model [5] is to study the behaviour of the flow of water in the forebay toward and around the intake towers and into the intake openings. The hydraulic model which reproduces the designed structure is built of plexiglass to a 1/80 scale of the prototype. Fig. 7 shows two views of the model of the intake and the forebay channel. The parameters affecting the flow are:

(i) The shape and orientation of the structures;

(ii) Different combinations of open and closed intakes;

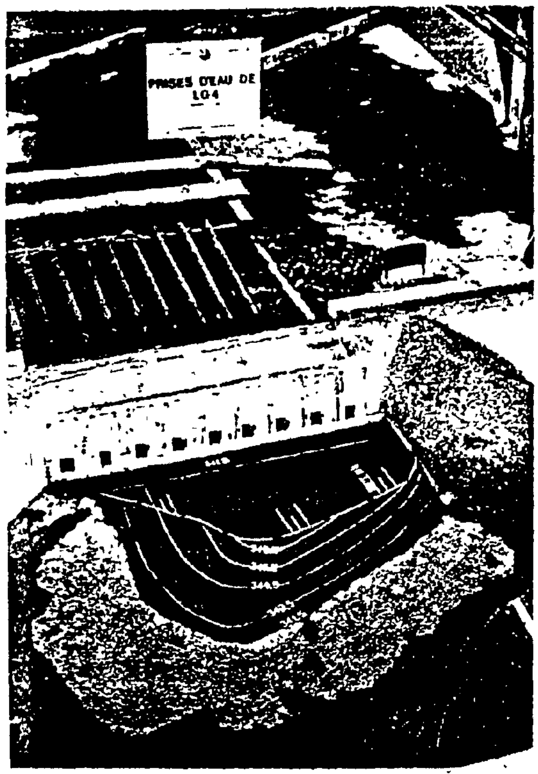
(iii) Approach conditions to the channel;

(iv) Forebay channel elevation;

(v) The depth of submergence of the intake opening.

The model was tested under different simulated conditions of the power house operations. Some openings were operational while others were shut down.

Initially, the model included interconnecting walls between the towers only up to the level required to retain the toe of the dam.



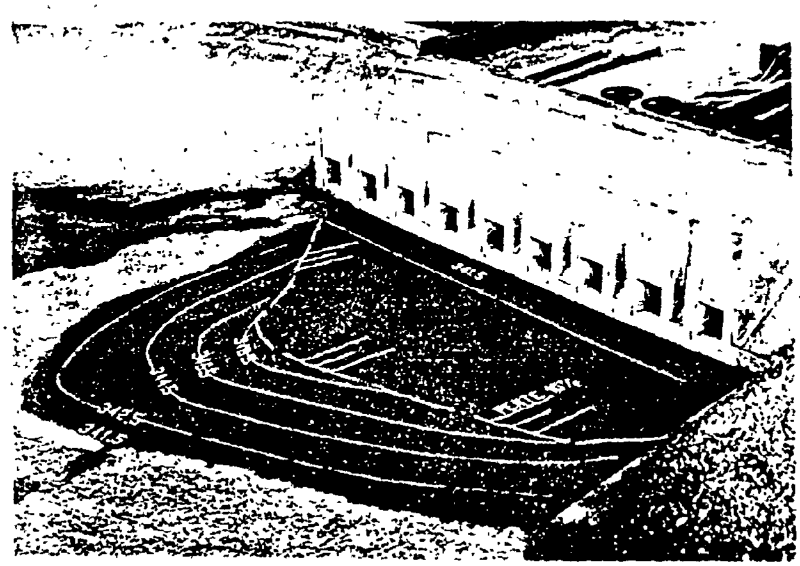


FIG. 7: View of the Model of Intake and Forebay.

Tests were made for maximum and minimum reservoir levels to simulate summer and winter conditions. Partial operation of the power house created turbulence and vortices. In some cases, they entrained air into the penstocks. Vortices are not desirable because, if they entrain air down the penstocks, they reduce the efficiency of the turbines and create undesirable pressure fluctuation in the draft tube.

In the attempt to eliminate vortices, the cross sectional shape of the towers was changed from trapezoidal to round. Vortices persisted under partial plant operation. After several trials with different modifications, vortices were eliminated by interconnecting the towers to full height with a wall having a smooth upstream surface. This wall prevented the circulation of the water around the towers. The water flowing between the dam and the intake structure had no influence on the hydraulic behaviour in the approach to the intake.

At the extremes, funnelling walls were added to direct the flow of the water towards the openings.

The minimum submergence of the intake openings below the water level of the reservoir as required to prevent vortex action was confirmed by the model test. The floor of the forebay channel was tested with and without a slope (see Fig. 8), but no difference was observed. The sloped alternative was adopted because of the reduced quantity and cost of the excavation.

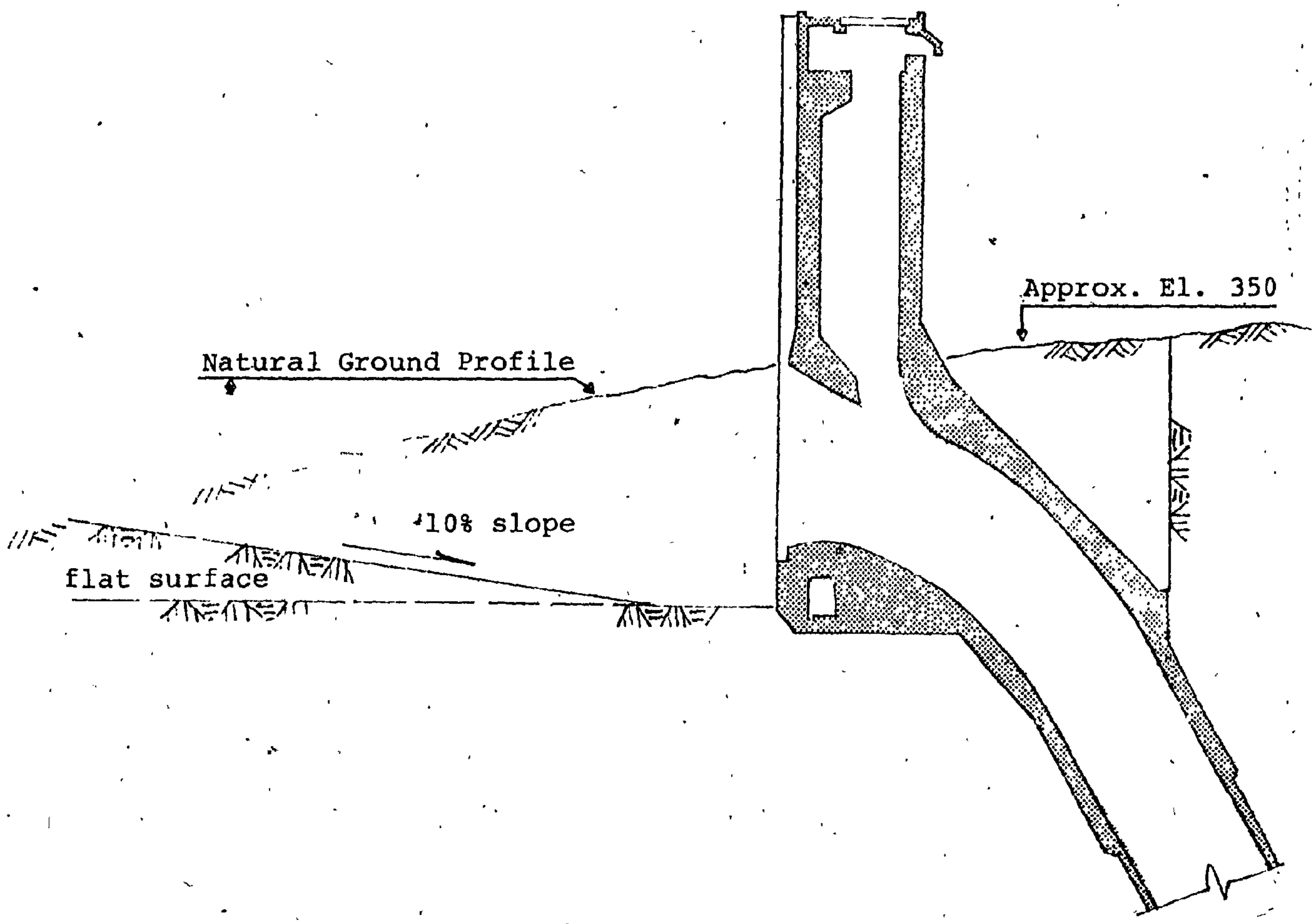


FIG. 8: Forebay approach channel showing the flat and sloped excavation

3.3.3 Stability Analysis

The analysis was made of the entire structure of one intake tower, including the interconnecting walls, for construction and for operation cases. The forces acting on the structure during these two cases are given in Figs. 9 and 10. The Figures show the calculated safety factors and the resultant pressure on the base of the intake structure.

3.3.4 Ice Impact

During spring thawing, [6] ice blocks in the reservoir gain momentum due to flow and float toward the intake structure. Although the velocity of these blocks is low, the enormity of their masses may be sufficient to cause impact damage to the intake structure. The following empirical formula was used to calculate the ice impact force:

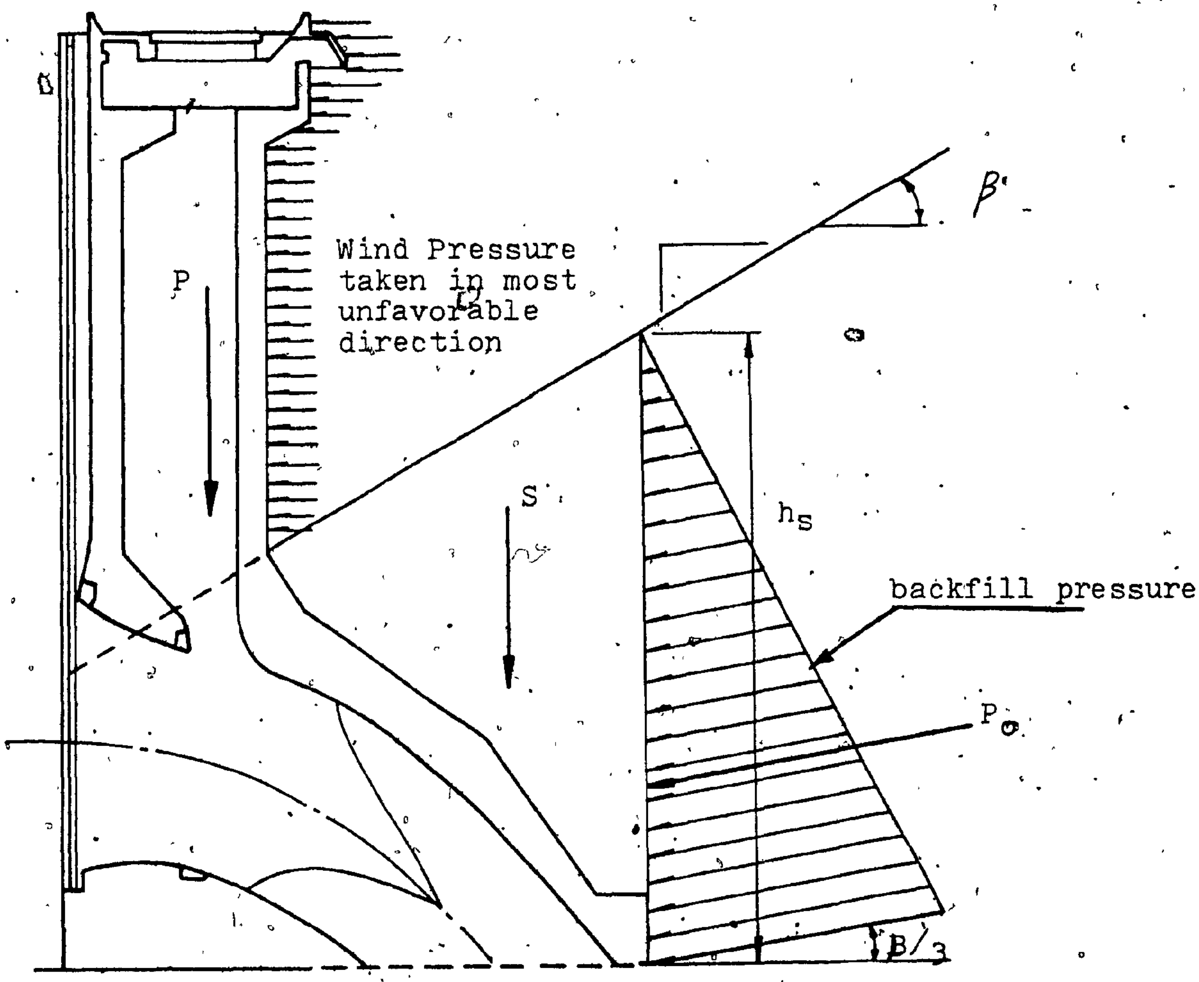
$$F_i = M\ K\ T\ h\ b$$

where F_i = ice impact force in kN; M = shape factor; K = contact coefficient in percentage; T = compressive strength of ice, in kPa; h = ice thickness based on degree days for LG-4 site in meters; b = projected width of the structure in the direction of ice motion in meters.

$$F_i/b = 750 \text{ kN/m for hard ice; or}$$
$$450 \text{ kN/m for soft ice}$$

The impact load exerted by hard ice is treated as an extreme case, because the strength of the ice reduces rapidly in the spring thaw when the continuous ice cover begins to break up and form floes. Since the reservoir level fluctuates from season to season, the structure is designed to

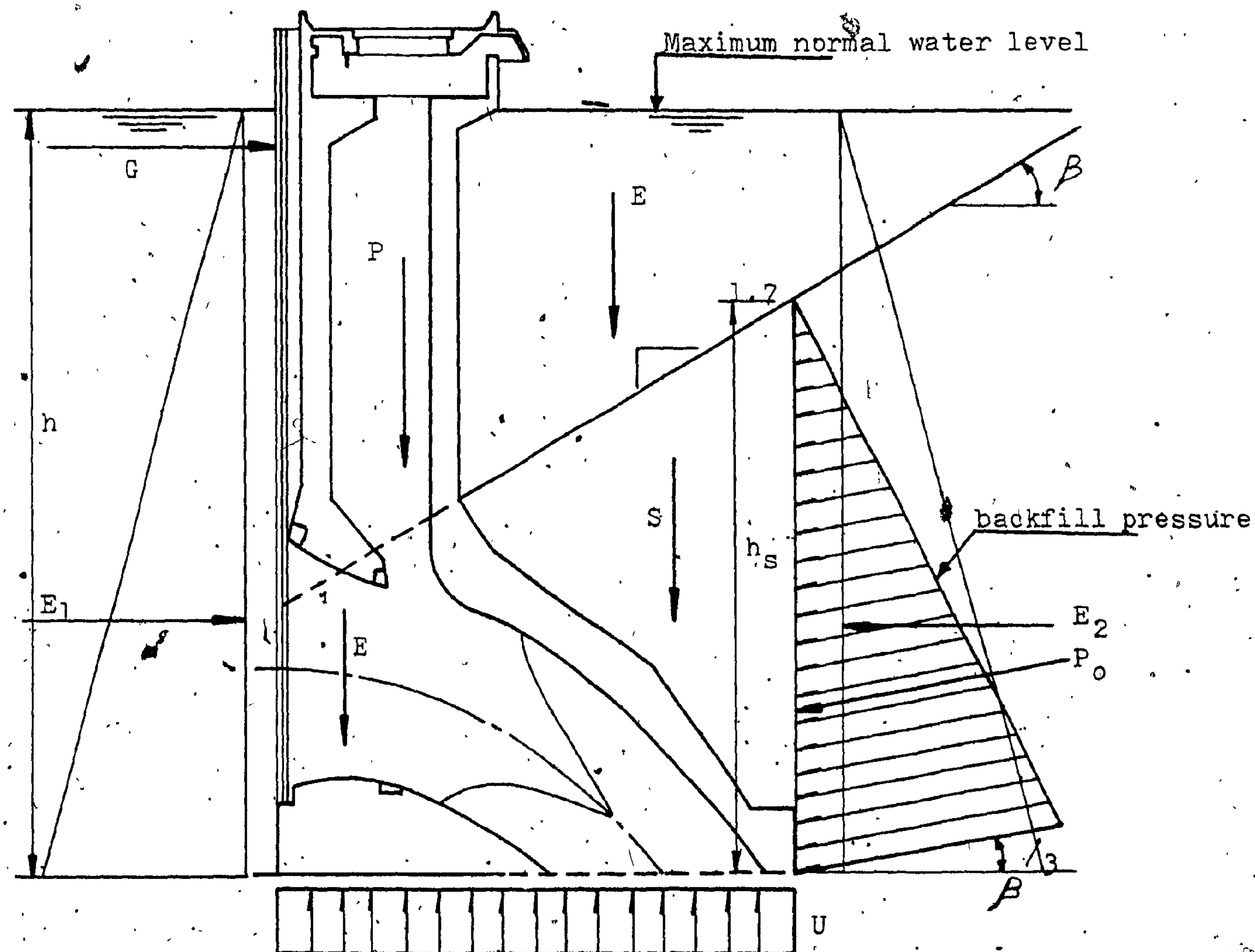
Where P = weight of the concrete mass
S = weight of the backfill
h_s = height of backfill
P_o = backfill pressure
β = slope angle of backfill



Item	Overtur.	Slid.		Foundation Pressure (kPa)
S.F.	2.5	10		650 250

FIG. 9: Forces acting on the structure during construction period.

Where P = weight of concrete mass
S = weight of backfill
G = ice forces in most unfavorable direction
h = hydrostatic head
h_s = height of backfill
E = weight of the water
E_1 = hydrostatic pressure
E_2
U = uplift pressure
P_o = backfill pressure
β = slope angle of backfill



Item	Safety Factor: Overtur	Slid.	Uplift	Foundation Pressure (kPa): Upstream	Downstream	Side
Upstream	1.3	8	1.9	6.4, 251, 1150	500, 300	100, 850
Downstr.	1.7	12	1.9			
Side	1.5	10	1.9			

FIG. 10: Forces acting on the structure during operation.

absorb the ice impact at any level between the maximum and minimum operating levels of the reservoir. Also, the structure should be capable to absorb the impact at any angle and at any point along the upstream face.

3.3.5 Thermal Expansion of Ice

During the winter months, an ice sheet [7] is formed on the surface of the reservoir. As the temperature changes the ice expands and exerts pressure on the towers. Ice between the towers is protected from snow accumulation by the intake deck which leaves the ice sheet uninsulated. Hence, the ice sheet behind and between the towers may be thicker than that of the forebay where the snow accumulation is greater. Consequently, the pressure exerted by thermal expansion of ice behind and between adjacent towers is greater than that exerted on the upstream face of the structure.

3.3.6 Structural Analysis and Design

3.3.6.1 Introduction

The intake structure is designed for all possible loading cases combining ice, hydro-static, backfill forces and all other forces which may act simultaneously or in other combinations. The ice force acts as a static load caused by the thermal expansion of the ice sheet or as a dynamic load due to impact with floating ice blocks propelled by flow and wind toward the intake. The latter is the most critical load on the structure.

3.3.6.2 Shear Resistance

Fig. 11 shows the applied load due to ice impact acting on the full width of the tower and the resisting shear force diagram. All calculations are based on the ultimate strength design method outlined in the reinforced concrete code specified in Reference [4]. The tower is divided in two typical sections; one from the invert to the top of the water passage, and the other above it.

Fig. 15 (Section 10) shows a typical cross-section above the water passage. For shear resistance this section is analysed as a closed box. All of the horizontal reinforcements act as stirrups which aid the concrete to resist the shear forces. The lower portion which is open on the upstream face is analyzed as a U-section. Each side of this U-section is considered to act as a separate rectangular section in which the horizontal steel reinforcement acting as a closed stirrup contributes to the shear resistance of the entire section.

3.3.6.3 Flexural Resistance

The upstream-downstream ice impact force and the flexural resistance of the tower is shown on Fig. 12. The tower is analysed as a cantilevered structure embedded at the base. Both the box and the U-sections are analysed as T-beams, where the two sides are considered as the web and the downstream face of the section as the flange. The critical section for this structure is located just above the water passage. In calculating the flexural resistance of the tower at this level, only the vertical reinforcement on the wings is considered as being effective. The central portion (see Fig. 15 Section 10 and elevation B) is neglected since its continuity is interrupted by the water passage and contributes very little to the overall strength of the cantilevered structure. Figs. 14 and 15, with their respective cross sections, give the reinforcing details of the tower.

3.3.6.4 Torsional Resistance

Fig. 13 shows a cross-section of the tower loaded on half of its width. The assumptions made in calculating the torsional resistance of the tower are similar to the ones made for the shear resistance pertaining to the upper and lower sections. The critical section of the tower is at the interface between the open and closed sections. In the open section, the torsional resistance increases with the increase of cross section with depth.

The analysis and design for the combined effect of shear and torsion are well accounted for throughout the structure. It is found that the torsional stress alone in the concrete of the open section surpasses the permissible stress given in the Reinforced Concrete Standard [4]; while the torsional stress in the closed section is within the permissible value. Thus, the design for the combined effect of shear and torsion is applicable only in the open section.

This is achieved by using the formulae stipulated in the standard and the section is reinforced accordingly (see Fig. 14, Section 2).

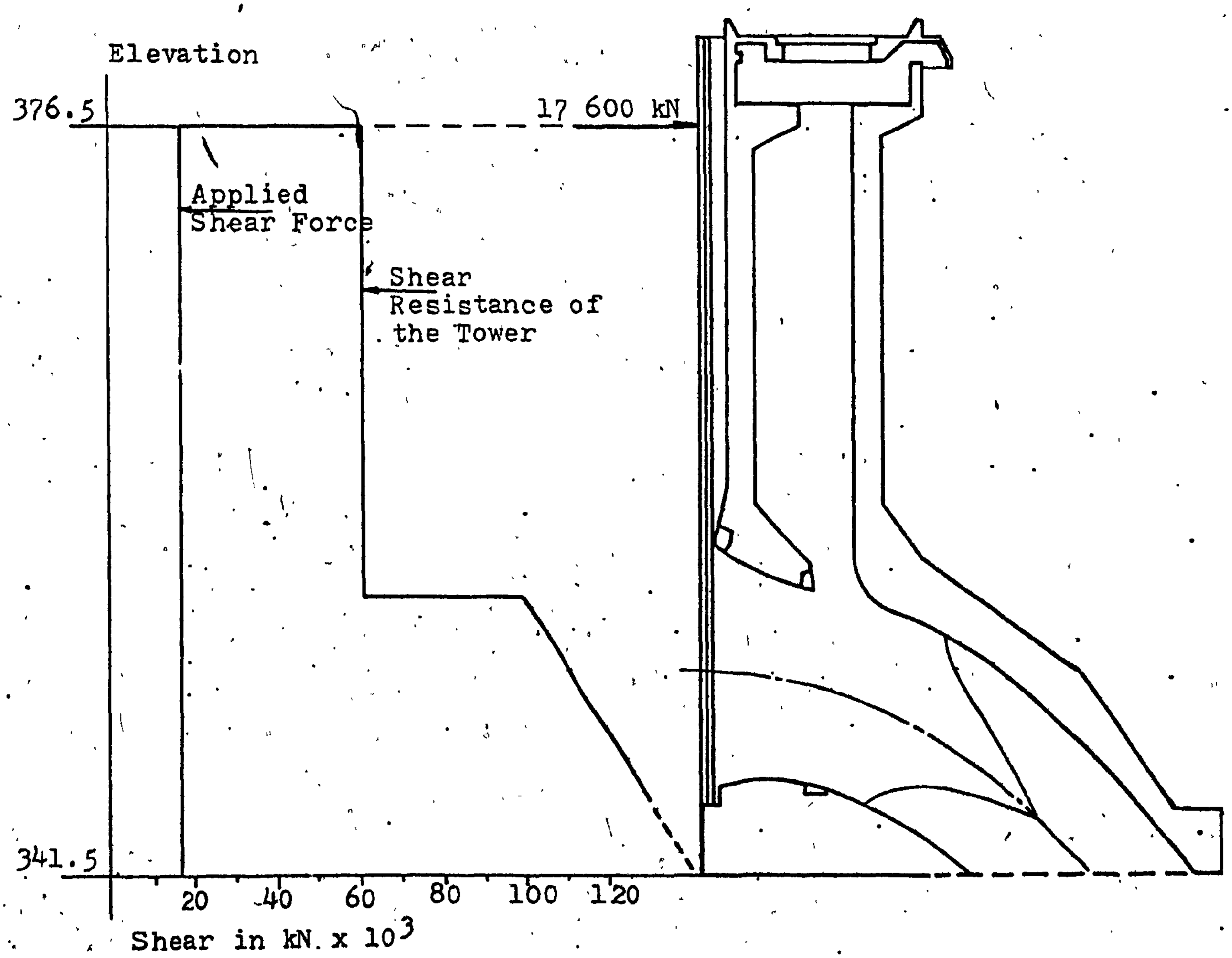


FIG. 11: Applied shear due to ice impact force and shear resistance diagrams.

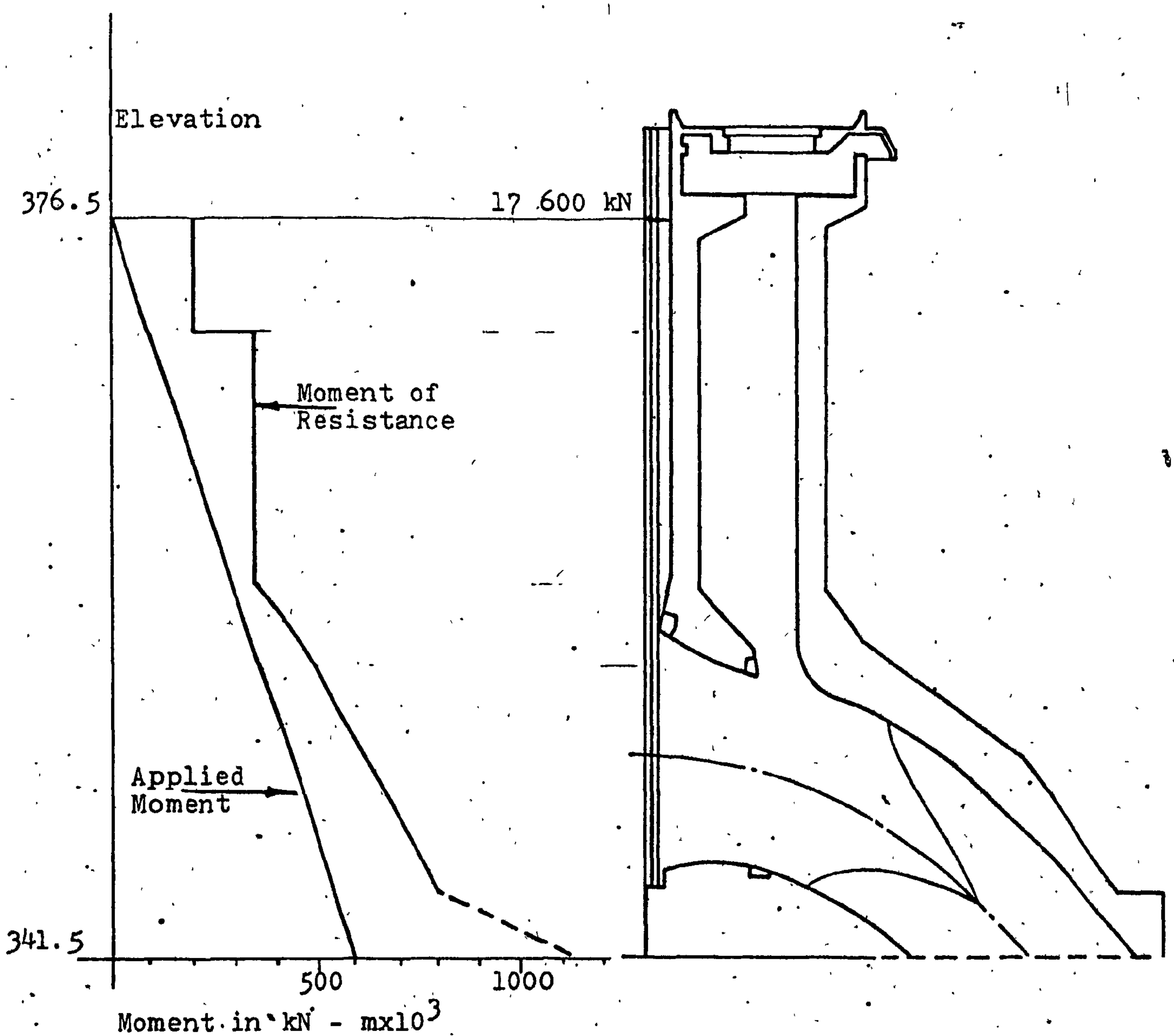


FIG. 12: Bending due to ice impact force and resisting moment diagram.

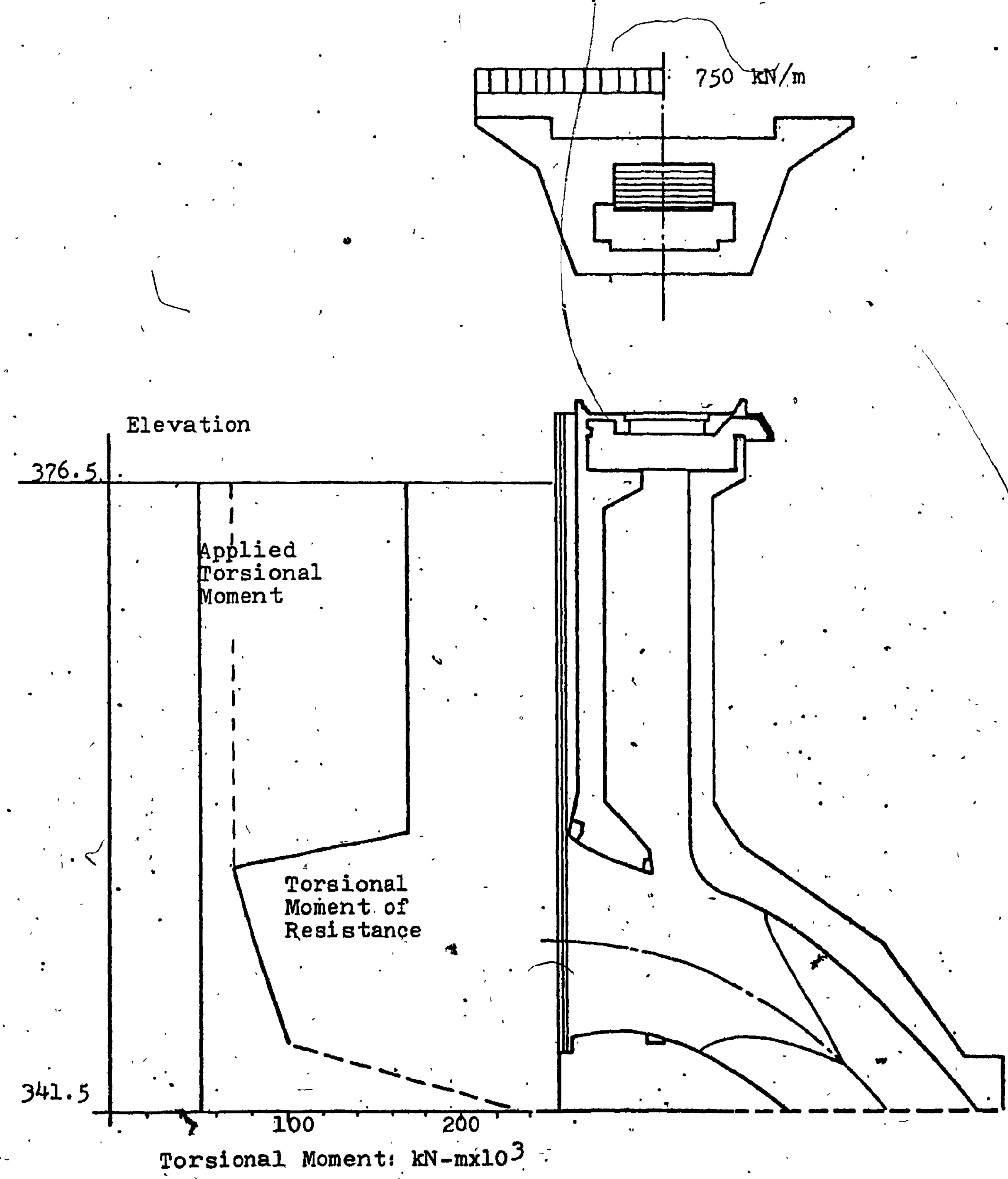


FIG. 13: Applied torsion due to ice impact force and torsional resistance diagram.

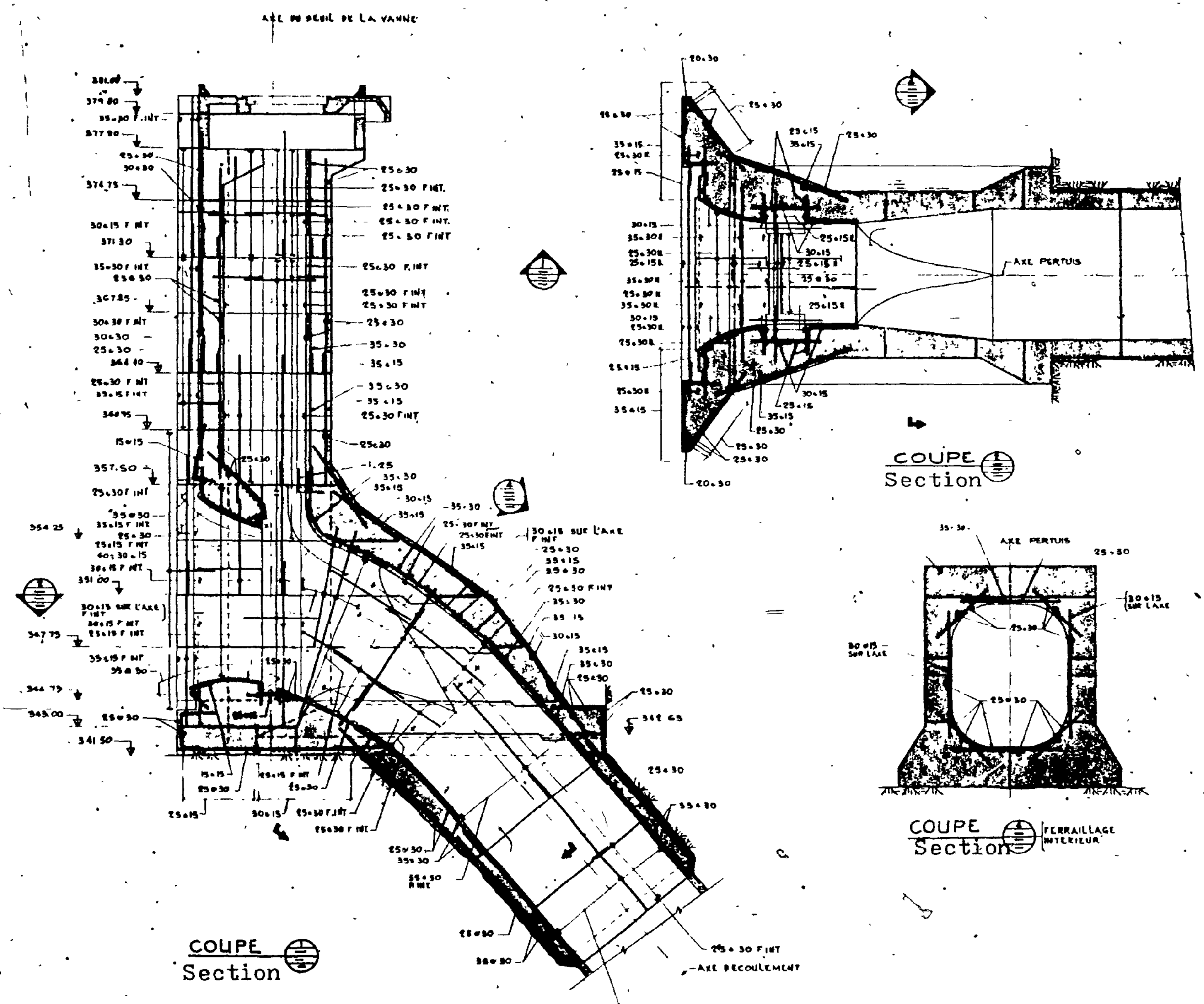


FIG. 14: Reinforcing details

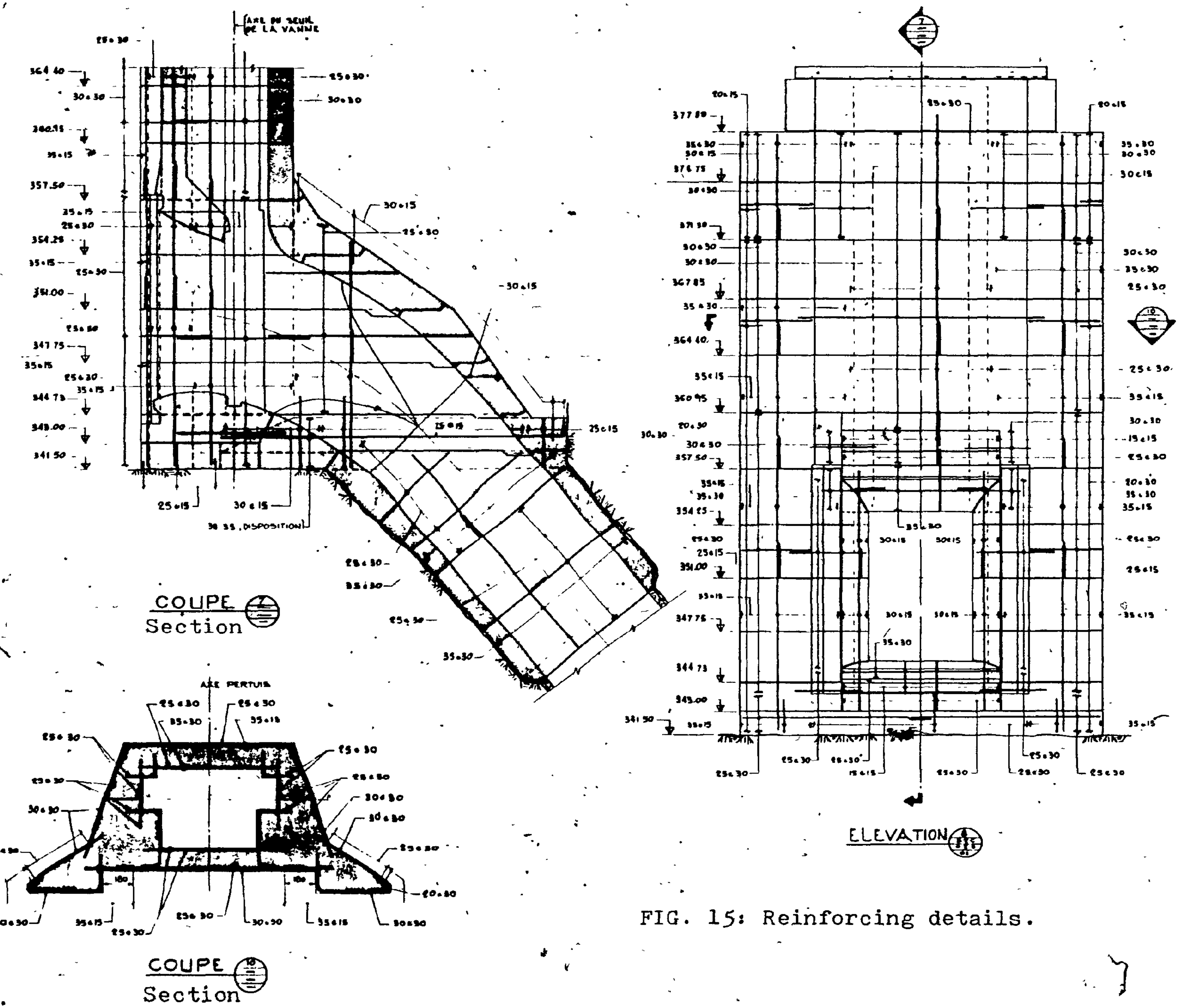


FIG. 15: Reinforcing details.

4.0 CONSTRUCTION

4.1 Introduction

Scheduling construction phases is of paramount importance. In heavy construction, such as this, activities should be well organized so that one operation will not impede or delay another. Where two contractors are in operation simultaneously, the work should be planned such that one contractor does not interfere with the operation of the other.

It is the field engineer's responsibility to see that the construction produces the structure as designed. The foundation must be excavated to sound rock; the joints and the temperature control of the concrete have to be well planned, so as not to develop cracks and excessive shrinkage in the structure.

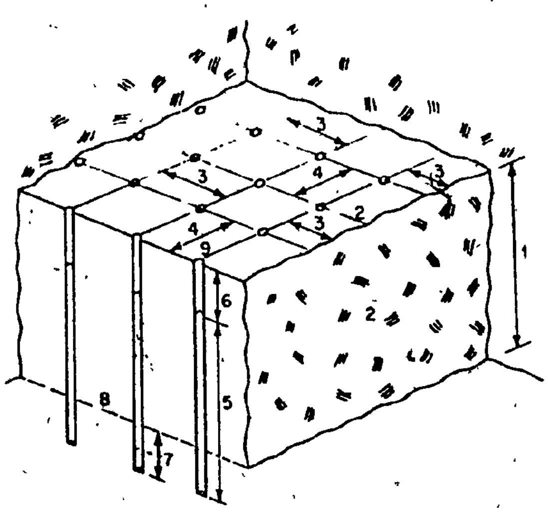
4.2 Excavation and Foundation Treatment

4.2.1 Excavation

The first activity is to excavate the forebay channel. The operation requires approximately 4 weeks with the use of conventional methods for open cut excavation. To expose the rock surface, the overburden material has to be removed before the drilling and blasting begins. The overburden is removed with a scraper or a small dozer.

An air track drill [8], approximately 8 cm in diameter, is used for rock excavation. The depth of the excavation is about 5 m so that only one excavation bench is necessary. The contractor is required to pre-split the rock before any other blasting begins. Pre-splitting means that a single row of closely spaced holes is drilled all along the perimeter of the rock mass to be removed. These holes are loaded with explosives very lightly and are continuously and simultaneously blasted. This action has the effect of separating the rock to be excavated from the surrounding rock, by creating an artificial joint and a smooth rock face. Pre-splitting operations help retain a stable rock face at the back of the intake where the toe of the dam core and filter rest.

The following diagram [8] shows a typical excavation block:



(1) Bench height = 10 meters
(2) Free face
(3) Burden = 20 to 40 explosive diameters
(4) Spacing = 1 to 2 x burden
(5) Power column
(6) Stemming = 0.5 to 1.0 x burden
(7) Subdrilling = 0.3 x burden
(8) Working floor of cut
(9) Collar
(5 + 6) hole depth - 1.5 to 4x burden

Wave propagation is monitored very closely in the vicinity of the blast, where concreting or grouting operations are in progress. The contractor is required to keep particle velocities at the surface of the rock or concrete within 0.05m/s to 0.15m/s. In general, the quantity of powder needed for open cut excavation in granite is approximately 1 Kg/m^3 of rock.

4.2.2 Foundation Treatment

The foundation of the intake [1] has to sound in order to resist the heavy load imposed by the intake structure. To get rid of loose blocks at the foundation level, it is necessary to expose the rock mass. Geological mapping is done and all loose blocks are removed. Shear planes should not have a resistance below the criterion used for the analysis of the stability of the structure; slopes are kept within 10%.

The position of the intake (upstream of the core of the dam) eliminates the need for curtain grouting under the intake, because the continuity of the dam is not interrupted. This operation would have been necessary if either of the other two alternatives discussed herein were adopted.

4.3 Temperature Control, Joint Systems and Formwork

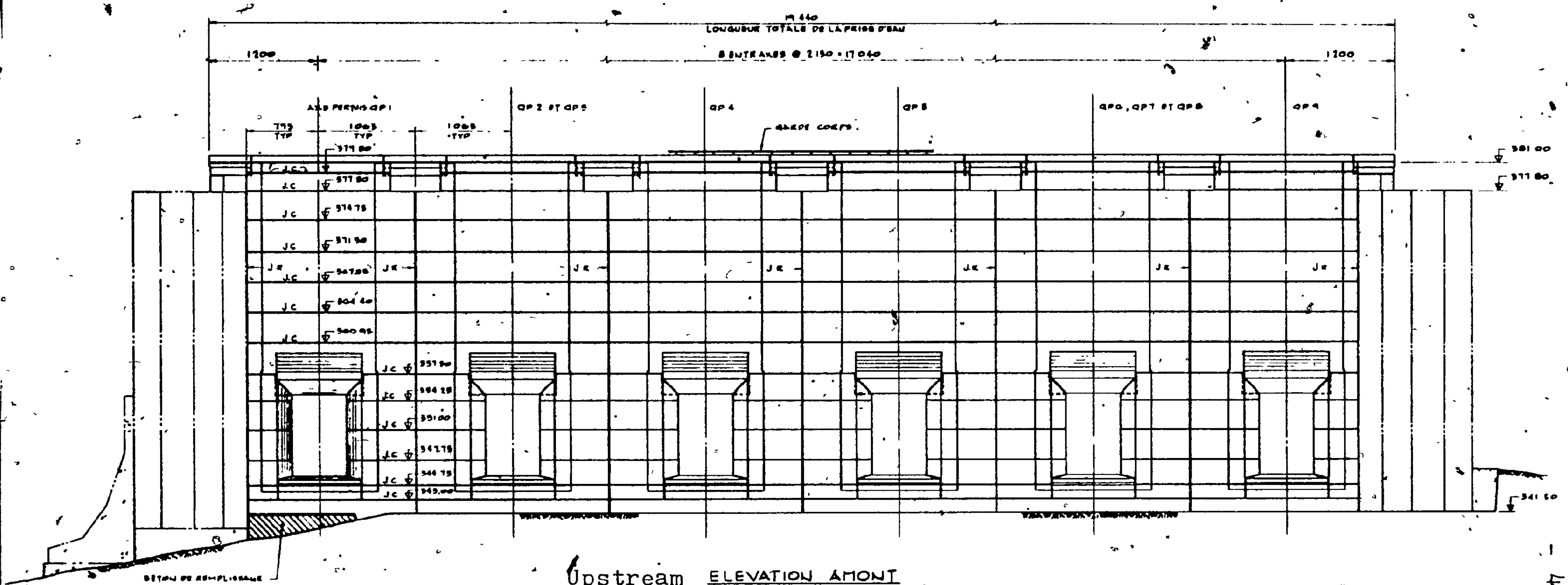
Problems such as shrinkage, temperature cracks, surface roughness are often encountered. These problems can be controlled by eliminating their causes altogether or whenever possible reduce their damage to a minimum.

4.3.1 Temperature Control

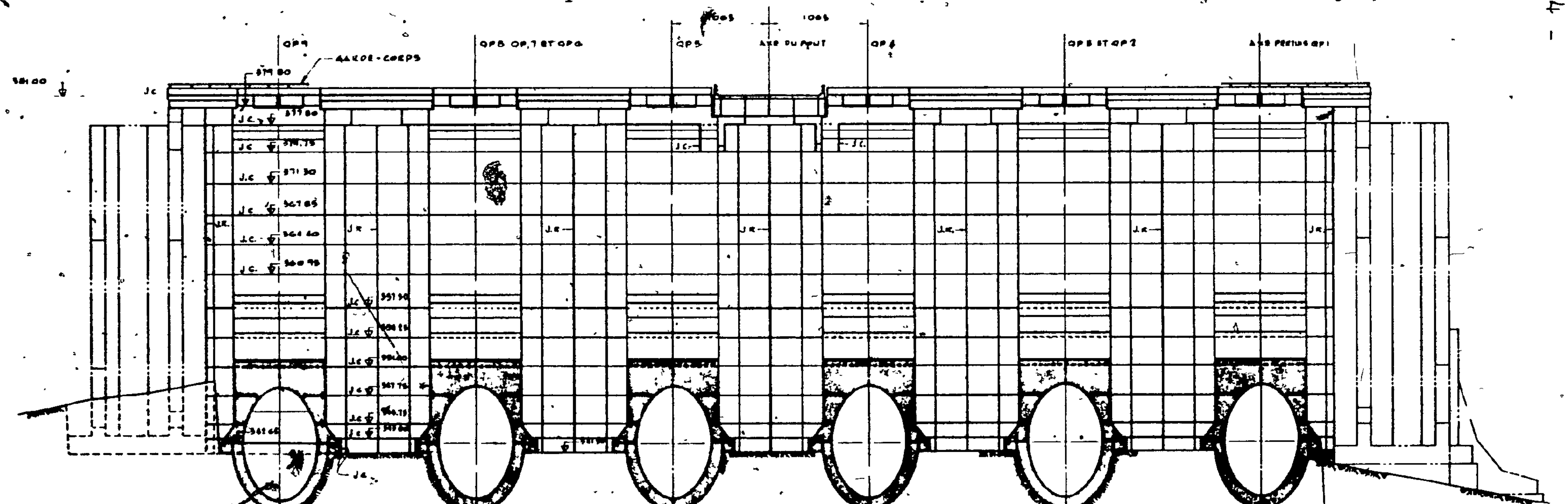
In massive concrete structures [1, 9], temperature control is an important factor. Other problems such as shrinkage and microcracks can be controlled by limiting the temperature fluctuation which the mix undergoes because of its exothermic reaction. Temperature can be controlled by many methods. Adding ice water or crushed ice to the mix is the cheapest method. Other methods consist of adding layers of pipes at different intervals in the lift and running cold water through them. Also, temperature in the concrete mass in the intake towers can be adequately controlled by limiting the maximum placing temperature, the height of the lifts and the time delay between successive pours.

4.3.2 Joints

Construction and contraction joints are chosen [1,9] keeping in mind the geometry of the section and the height of each lift. Figs. 16 and 17 show the construction joints of the intake structure. The individual intake towers are designed as free standing structures which are stable by their own mass. Hence, there is a required vertical contraction joint every 21.3 metres separating adjacent tower units. The size of each lift is a function of batch capacity of the concrete plant, economy of the formwork, and limits of temperature control. Time is also a factor since concrete should remain plastic until the end of concreting of each lift. Speed in pouring eliminates or minimizes cold joints. Cold joints are formed if concreting is interrupted for a period of time exceeding the final set time. The first lift poured is the most critical. Concrete expands during the first 24 hours, due to its rise in temperature.



Upstream ELEVATION AMONT



Downstream ELEVATION AVAL

FIG. 16: Construction Joint System

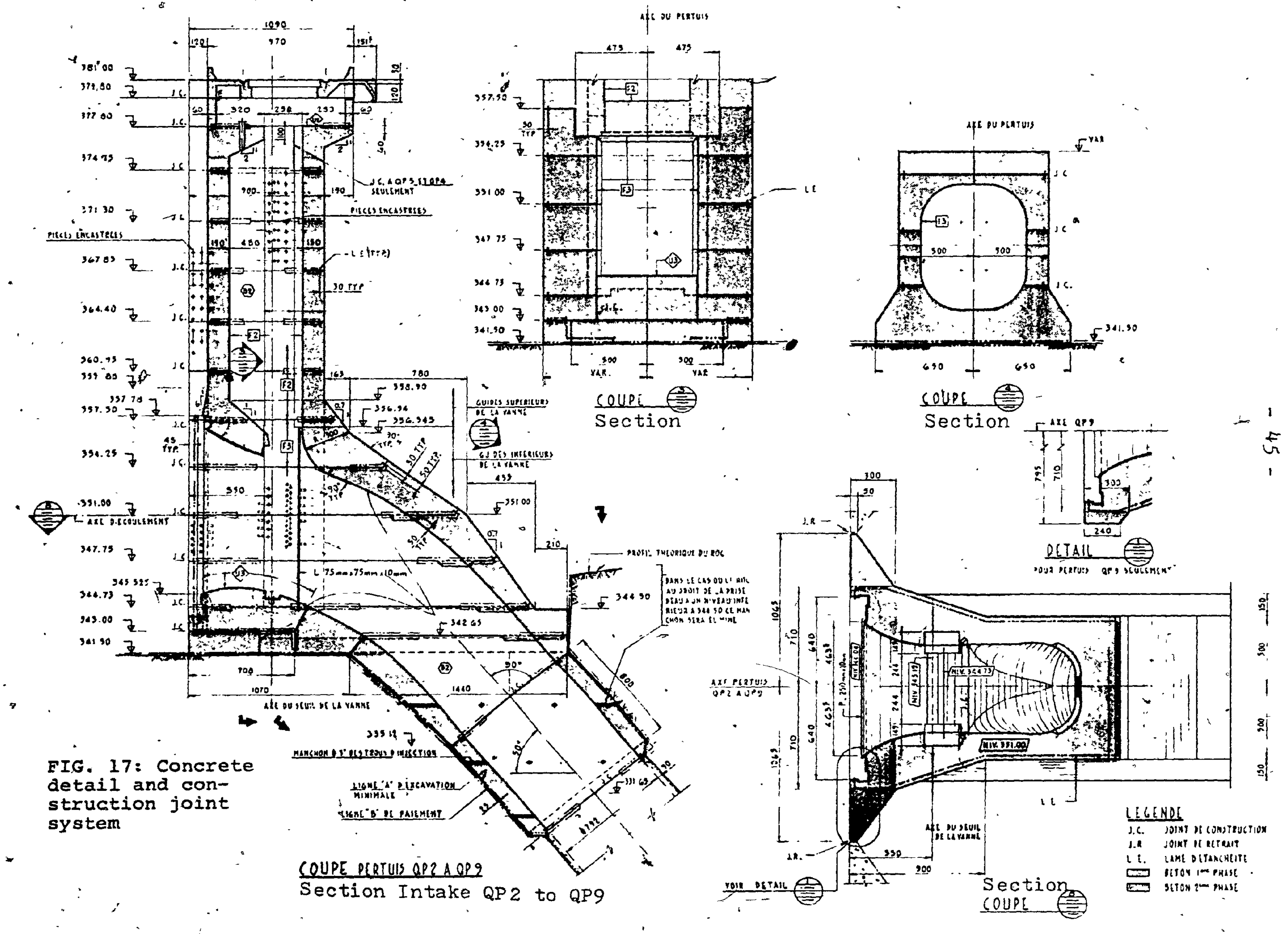


FIG. 17: Concrete detail and construction joint system

The rock, because of its larger mass, does not expand at the same rate. When concrete cools, it will shrink but the rock will not have the same movement of shrinkage as concrete. Concrete will crack because this differential shrinkage exceeds the tensile stresses in the mass. To minimize this effect, the height of the lift is kept at approximately 1 to 1.5 metres; this reduces the concrete volume and the temperature rise of the concrete. Also shown on the cross section of the intake (Figs.14 and 15), reinforcing steel is provided to decrease the possibility of cracks because of shrinkage at the base of the structure. The upper successive joints are free from this problem since the volumes of concrete of succeeding lifts are kept almost equal.

4.3.3 Formwork

A well designed formwork [10] can save money and is safe against loss of lives and materials. The criterion used in designing and planning formwork depends on the importance of the structure, the repetitive use of each panel, and the desired surface finish.

The optimum depth of each lift [1,8,9,10], taking into account all the parameters outlined in this chapter, is between 3.0 to 3.5 metres in the thinner sections of the intake structure. The lifts from the foundation to the hydraulic passage range from 1.5 to 3.5 metres.

The formwork used for this kind of structure is either prefabricated and sometimes of patented design. Because of the modular nature of the intake, the use of patented formwork is economical due to multiple and repetitive use. Patented forms have a sheating in contact with concrete and usually are supported by steel or aluminum studs. The United States Bureau of Reclamation outlines at least three types of formwork finishes for most hydraulic structures [1]:

Finish 1: This type of finish is used for surfaces which are not exposed to sight. Surfaces covered by backfill or second phase concrete are formed by this type of finish.

Finish 2: Used for all exposed surfaces.

Finish 3: Used for all faces exposed to high flow velocities, such as the hydraulic water passage.

The number of form sets needed for a job depends on the judgement of the construction engineer. The frequency of use of the formwork depends on the time the concrete is allowed to cure with the formwork on. For thin wall sections 72 hours of curing are needed before the removal of the formwork. But, for massive thick sections, 48 hours suffice. Formwork should never be removed before the concrete temperature near the exposed face is at or close to the ambient temperature. Great care and precautions are taken to avoid openings in the tower which may induce a chimney effect. The cold air exposes the freshly unformed concrete to a sudden thermal change, thus increasing the chances of shrinkage cracks.

Concreting in winter is usually beneficial to massive structures, but for relatively thin sections such as the intake tower, the formwork must be insulated or heated.

4.4 Concrete Placing and Finishings

4.4.1 Concrete Placing

Concrete is placed by overhead cranes mounted at the center of one of the towers. From this position, the crane can service the two adjacent towers. Concrete pumps may be used by installing additional delivery pipes as the tower rises.

Concrete [11] must be placed within 90 minutes of commencement of batching. Concrete is placed in layers of 50cm/hour; the time interval between successive layers should be as short as possible, to avoid cold joints and must never be less than one hour. Vibration of newly placed concrete is done by experienced men as to avoid segregation or bleeding in the mix. The vibrator must penetrate each successive layer to create a good interaction between them. No vibration is permitted near the forms because this has a tendency to trap air at the interface, forming air pockets in the concrete surface.

4.4.2 Concrete Surface Finishes

The hydraulic water passage [11,12] has the most critical finish. The surface is subjected to high velocities. Rough finishes cause the water flow to be irregular. This reduces the effective area of the water passage which may lead to costly head losses. To avoid this, abrupt irregularities transverse to the direction of flow are not accepted on the concrete surfaces. They have to be removed by grinding or other methods to a maximum slope in every direction of 50:1. Other surface finishes are kept within reasonable tolerances.

4.5 Construction Schedule

No matter how insignificant or important a job, a carefully detailed plan can save both time and money for the contractor. Each contractor bidding on a job of this size is required to present a detailed construction schedule of every operation. Minor modifications are made on the proposed schedule by the project manager if deemed necessary.

Provided the contractor's schedule respects deadlines and other restrictions in the specifications and plans provided for bidding, he is responsible for the setup, for the sequence of operations and for the general handling of the activities required of the job.

The flow chart on fig. 18 gives the general sequence of work. Fig. 19 shows the preliminary construction schedule for the intake structure.

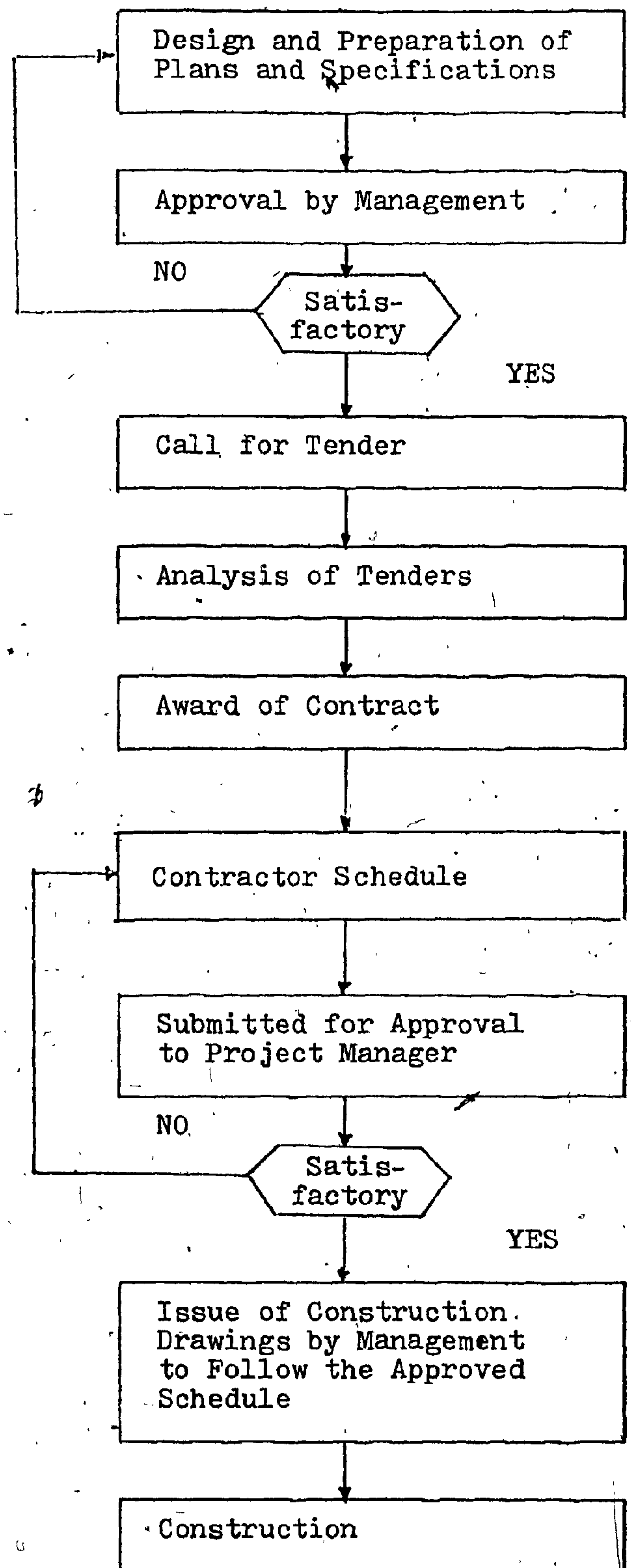


Fig. 18: Flow Chart - sequence of design work

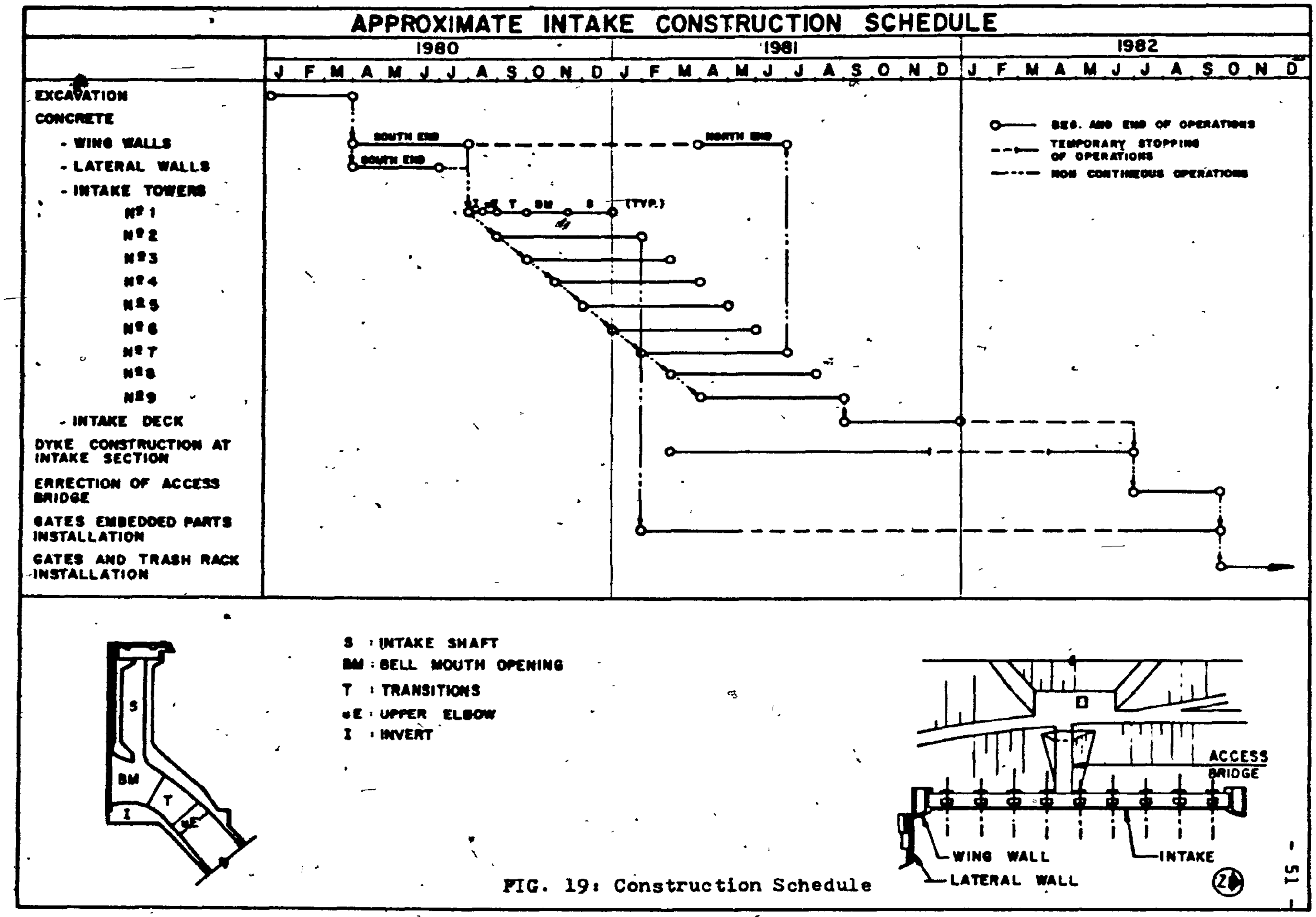


FIG. 19: Construction Schedule

5.0 CONCLUSIONS

On the basis of this study, the following conclusions may be reached insofar as optimisation, design and construction of the intake structure for the LG-4 project are concerned.

1. The type of intake chosen, considering the relatively high head of water, was a system where each intake tower feeding one turbine, rather than a tower feeding multiple turbines through a manifold and valves.

2. The optimum position of the intake structure was found to be upstream from the dam, which avoids any interference with the dam core and the backfill load on the downstream side of the structure.

3. The hydraulic model showed that, raising the wing walls of each tower to maximum operational water level, the vortices and air entrainment into the water passage was eliminated. The model also assured the designers that the bell-mouth opening, the transition and the submergence of the intake opening were adequate.

4. Theoretically, the cantilever tower was found to have more than adequate resistance against the ice impact force in terms of shear, flexure and torsional resistance. The factors of safety against overturning, sliding and buoyancy are found to be within the allowable limits outlined in the references given for this type of structures.

5. Construction was characterized by the different operations which the field engineers must follow to build a sound structure. Special emphasis was given to the foundation treatment after completing a carefully planned excavation. Construction joints and temperature control of each lift were discussed to minimize shrinkage and temperature cracks due to volume change. Surface finishes were treated in order to point out the importance of using good forms to minimize reduction of effective flow area and, hence, head losses in the water passage and transition zone. Finally, the construction schedule was given to show a tentative sequence of operations and their duration.

REFERENCES

[1] United States Department of the Interior Bureau of Reclamation, "Design of Gravity Dams", design manual for concrete gravity dams, a water resources technical publication.

[2] Societe d'Energie de la Baie James, "Communication d'Etude; Choix du Type de Prise d'Eau", internal document, April 1, 1977.

[3] National Building Code of Canada 1977.

[4] National Standard of Canada CAN3-A23.3-M77, "Code for the Design of Concrete Structures for Buildings".

[5] Societe d'Energie de la Baie James, "Amenagement LG-4: Etude Sur Model Reduit de l'Ouvrage de Prise d'Eau", internal document, September 1978.

[6] GOSSTROI, "Instructions for Determining Ice Loads on River Structures (SN76-66)", UBSR, 1973, Technical Translation 1663, National Research Council of Canada.

[7] Michel, B., "Ice Pressure on Engineering Structure", 1970, Monograph 111-B1b, Cold Region Research and Engineering Laboratory, Corps of Engineers, U.S. Army, Hanover, N.H., U.S.A.

[8] Havers, John A., "Handbook of Heavy Construction", McGraw Hill Book Company.

[9] Waddell, Joseph J., "Concrete Construction Handbook", McGraw-Hill Book Company.

[10] Peurifoy, R.L., "Formwork for Concrete Structures", McGraw-Hill Book Company.

[11] Societe d'Energie de la Baie James, "Devis Technique Normalise; Travaux de Betonnage", Norme SEBJ AA-80N-001(SI)", January 1979.

[12] Davis C.V. & Sorensen K.E., "Handbook of Applied Hydraulics", McGraw Hill Book Company.