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A COMPARISON OF THE HYDRAULIC PERFORMANCE OF AN ORIFICE AND AN OVERFLOW SPILLWAY IN A NORTHERN APPLICATION USING PHYSICAL MODELLING

by

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A Thesis
Submitted to the Faculty of Graduate Studies
in Partial Fulfillment of the Requirements
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Abstract

The feasibility of operating an orifice spillway in a northern environment was investigated in this thesis. Concerns regarding the operation of an orifice spillway included the potential of forebay trash jamming the flow passage, ice floes being drawn down from the surface and impacting a partially open gate, and an increased potential for cavitation damage due to higher flow velocities than for an overflow spillay. Manitoba Hydro's proposed Wuskwatim Generating Station on the Burntwood River in northern Manitoba was used as a case study since a preliminary study of the site indicated a cost advantage of an orifice spillway relative to an overflow spillway.

The hydraulic performances of the two spillway types were compared using physical hydraulic modelling. A single-bay Froude model (length scale = 36) of each spillway was tested in a flume. The orifice spillway scheme was also examined with a comprehensive site model that had a length scale of 64.

Several design modifications were made to the orifice spillway during the testing program and none of the concerns investigated eliminated the orifice spillway from future consideration. The need for modifications to the overflow spillway preliminary design was also identified during the research.

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Contents

List of Tables

Table		Page

List of Figures

Figure		Page

Figure		Page

List of Plates

CHAPTER 1

Introduction

Depending on local site conditions, orifice-type spillways may be used economically in conjunction with, or in place of, conventional overflow spillways. The uniqueness of conditions at every water development project site necessitates an independent assessment of spillway concepts for each project. Although design guidelines for overflow ogee crest spillways are readily available, "the theory required for mathematical modelling is not readily available for orifice spillways" [17,p2].

The purpose of this study was to investigate the viability of implementing orifice spillways in cold climate locations. Physical models were to be used to evaluate the hydraulic characteristics of an orifice spillway relative to those of an overflow spillway of equivalent design criteria (e.g., same maximum forebay level and required to pass the same design flood). Obtaining design refinements to a site-specific orifice spillway was a secondary objective of the project.

A comparative study was undertaken as a joint research project between Manitoba Hydro and the University of Manitoba's Civil Engineering Department. The study consisted of a hydraulic model investigation of the orifice and overflow spillway concepts for the proposed Wuskwatim Generating Station. The Wuskwatim site, with its northern Manitoba location, provided an excellent case study for the research needs of this project.

1.1 Case Study

Consulting engineers for Manitoba Hydro have investigated the possibility of implementing an orifice spillway at a proposed generating station site in northern Manitoba. The spillway investigation was undertaken for the Wuskwatim Generating Station during a preliminary concept study of the Burntwood River waterway. The findings of the study indicated a potential twelve percent decrease in the capital cost of the spillway if an orifice spillway concept was adopted instead of a conventional overflow spillway configuration [2,p1]. This percentage of capital cost reduction is equivalent to approximately $2,100,000 (1985 $)

[2,p16]. However, the study indicated that the "saving has to be weighed against the lower reliability of operation of the orifices" [2,p1].

Manitoba Hydro approached the Civil Engineering Department of the University of Manitoba with a proposal to undertake a hydraulic model study of the two spillway alternatives in the department's Hydraulic Laboratory. The study was to include segmental model tests in a hydraulic flume, as well as tests involving a more comprehensive river model. Although these tests were proposed at a relatively early point in the overall generating station development schedule, the timing was considered appropriate since the comprehensive river model was still in place following a previous river closure study of the site.

Some of the areas of concern regarding the operation of an orifice spillway were identified as study topics for the hydraulic model study. These topics included: 1. the potential for plugging of an orifice or its gate slots by submerged trash, 2. the possibility of a partially opened gate incurring impact damage from surface ice drawn into the orifice, 3. vertical lift gate stability, 4. energy dissipation requirements, and 5. structural damage due to cavitation [5,p1].

The proposed Wuskwatim Generating Station site is located in northern Manitoba on the Burntwood River. This waterway includes 165 km of lakes and river reaches between the Notigi Control Structure, situated on the Rat River, and Split Lake, at the downstream end of the Burntwood River (see Figure 1.1). Manitoba Hydro proposes to develop the 90 metres of head between these two points by constructing four hydroelectric stations along the waterway [8,pES-1]. One of these stations, the Wuskwatim Generating Station, is located at Taskinigup Falls, one of the many rapids which comprise the majority of the total drop along the Burntwood River system. A 200 metre section of the Burntwood River drops approximately fifteen metres at this location [8,p9.1]. The development site is in an area of the Precambrian Shield where the bedrock appears to be suitable for the proposed structures [8,pES-9]. Figure 1.2 shows the preferred arrangement of the principal structures.

As part of the Churchill River diversion project, the operation of the Wuskwatim Generating Station will be subject to its agreements and licences [8,pS-1]. The flow and stage constraints contained in the agreements were used in determining the Wuskwatim Reservoir's optimum nominal forebay level of elevation 243.2 m [8,pES-9]. A normal maximum forebay level of elevation 242.0 m [2,p10] was specified for ice-covered conditions. The total installed capacity of the four-unit powerhouse will be 362 MW [8,pES-A].

The project design flood for the Wuskwatim Generating Station was taken as the Probable Maximum Flood (PMF) as recommended by the International Congress on Large Dams (ICOLD), which classifies the project as a "large dam with a high hazard potential" [8,pS-5]. The PMF to the Wuskwatim reservoir, including the normal maximum outflow from Notigi Control, was estimated to be 2715 m^3/s [8,pS-5]. It is assumed that the reservoir will be at its normal full supply level at the start of the flood runoff and the concrete structures will have a 1.5 metre maximum allowable surcharge above this level. The reservoir storage that this surcharge provides during a flood allows the peak inflow of 2715 m^3/s to be reduced to a maximum outflow of 2450 m^3/s [8,pS-5] through the overflow spillway. The spillway will be designed as the sole outlet of the reservoir during passage of the PMF [8,pS-5].

A 1250 m^3/s [2,p10] construction design flood with a 1 in 20 year return period was chosen for the Wuskwatim site. Since the closure of the Stage II cofferdam can be delayed until the construction design flood has passed, a design flow of 1100 m^3/s [2,p10] was chosen for cofferdam closure. The maximum upstream water levels at the cofferdam during closure and construction were elevation 230.0 m and elevation 230.5 m, respectively [2,pp10-11]. For second stage concreting of the spillway, the design flow and maximum upstream water level were

1250 m^3/s and elevation 240.0 m [2,pp10&12]. Tailwater levels at the spillway location are expected to be low enough so as to not affect upstream conditions.

The spillway design adopted for the Stage III Concept Studies of the Wuskwatim Generating Station was an overflow spillway with three 8 metre wide bays controlled by vertical lift gates. The Stage III Concept Study indicated that money could be saved by replacing the 8 metre overflow spillway bays with either three 7 metre wide overflow bays [2,p11] or with three 6 metre wide orifice bays [2,p1]. While the final selection of the spillway concept to be used at Wuskwatim was left for Stage IV studies, the less expensive 7 metre bay overflow concept was chosen for the comparison study with the orifice spillway.

1.2 Scope of Study

This thesis is divided into six chapters. Chapter 2 provides an overview of physical hydraulic modelling, including the development of Froudian model scale relationships.

The third chapter describes orifice and overflow spillways in general and the role each plays in the operation of the Wuskwatim Generating Station. The Orifice Spillways section of this chapter includes a generic operational and economic comparison between the two schemes and a short list of similar precedents for orifice spillways.

The first section of Chapter 4 describes the testing procedures for the flume model study of the orifice and overflow spillways. Each sub-section presents a distinct study topic and discusses different approaches or requirements for the two spillway alternatives. The second section describes the orifice spillway comprehensive model study in a similar manner.

Chapter 5 discusses the results of the model testing program with a separate section for the flume and comprehensive models, and a sub-section for each study topic.

The final chapter of the thesis presents the conclusions resulting from the model study.

CHAPTER 2

Hydraulic Modelling

2.1 General

Physical modelling is often the most efficient, or only, available method for solving many complex problems involving hydraulic phenomena. Mathematical modelling of the performance of hydraulic structures can be complicated since three dimensional flow patterns and time dependency must be reproduced. Furthermore, the conditions at many development sites necessitate innovative design concepts for which no theoretical or empirical relationships exist. For these innovative concepts, physical modelling becomes a particularly important part of the design process.

2.2 Similitude

Model behavior is related to prototype behavior through principles of similitude. Although many types of similarity exist, hydraulic modelling is usually based on achieving geometric (form), kinematic (motion), and dynamic (force) similarity. Dynamic similarity implies kinematic similarity which implies geometric similarity. For true dynamic similarity, the ratio of prototype force to model force must be equal for all of the types of forces present. Hydraulic phenomena can generally be described by the forces caused by pressure, gravity, viscosity, surface tension, and elasticity, as well as the hypothetical inertia force which balances their vector sum. Since no fluid can be found which will simultaneously give the same ratios of prototype force to model force for all of these types of forces, true dynamic similarity is impossible to achieve. However, for systems in which some types of forces are negligible, model studies can be used to achieve valid data without achieving true dynamic similarity [9,p348].

For most hydraulic structures, the dominant force affecting flow is gravity. If the other types of forces are assumed to have a negligible effect on hydraulic behavior, then dynamic similarity requires that:

$$\left(\frac{F_{ip}}{F_{im}}\right) = \left(\frac{F_{gp}}{F_{gm}}\right) \qquad (1)$$

or

$$\left(\frac{F_i}{F_g}\right)_p = \left(\frac{F_i}{F_g}\right)_m \qquad (2)$$

where F_{ip} and F_{im} are the inertia forces of the prototype and model, respectively, and F_{gp} and F_{gm} are the corresponding gravity forces.

If the inertia force is given as a product of mass and acceleration and the gravity force is the product of mass and gravitational acceleration, g, then either side of equation (2) can be written as:

$$\frac{\left(\rho L^3\right)\left(L/T^2\right)}{\left(\rho L^3\right)g} \tag{3}$$

or

$$\frac{L/T^2}{g} \tag{4}$$

where L is a standard length, T is time, and ρ is density. If T is replaced with L/V (where V is velocity), then expression (4) becomes:

$$\frac{V^2}{gL} \tag{5}$$

which is commonly referred to as the Froude number. Substituting equation (5) into both sides of equation (2) gives:

$$\left(\frac{V^2}{gL}\right)_p = \left(\frac{V^2}{gL}\right)_m \tag{6}$$

It can therefore be seen that to approach dynamic similarity in a fluid system dominated by gravity forces, the Froude number of the model and prototype must be equal.

Based on the requirement of equal Froude numbers for dynamic similarity, the following equations can be used to convert measurements from model to prototype quantities for a given length scale, (L_p/L_m), if the same fluid is used in the model and prototype:

Velocity, V

$$\frac{V_p^{\,2}}{g_p L_p} = \frac{V_m^{\,2}}{g_m L_m}\ ; \quad \frac{V_p}{V_m} = \left(\frac{g_p}{g_m}\frac{L_p}{L_m}\right)^{1/2} = \left(\frac{L_p}{L_m}\right)^{1/2} \qquad (7)$$

Discharge, Q

$$\frac{Q_p}{Q_m} = \frac{A_p V_p}{A_m V_m} = \left(\frac{L_p}{L_m}\right)^2 \left(\frac{L_p}{L_m}\right)^{1/2} = \left(\frac{L_p}{L_m}\right)^{5/2} \qquad (8)$$

Pressure, P

$$\frac{P_p}{P_m} = \frac{g_p \rho_p L_p}{g_m \rho_m L_m} = \frac{L_p}{L_m} \qquad (9)$$

Force, F

$$\frac{F_p}{F_m} = \frac{g_p \rho_p L_p^{\,3}}{g_m \rho_m L_m^{\,3}} = \left(\frac{L_p}{L_m}\right)^3 \qquad (10)$$

Time, T

$$\frac{T_p}{T_m} = \frac{L_p/V_p}{L_m/V_m} = \frac{L_p}{L_m}\left(\frac{L_p}{L_m}\right)^{-1/2} = \left(\frac{L_p}{L_m}\right)^{1/2} \qquad (11)$$

Conversion equations for other measurements can be similarily developed as required.

The ease with which equations (7) to (11) can be used to convert model measurements to prototype values, or vice versa, can be illustrated by the following example. If a discharge of 2.02 m^3/s is measured on a model with a length scale of 1:36, the corresponding prototype discharge would be:

$$Q_p = Q_m\left(\frac{L_p}{L_m}\right)^{5/2} = 2.02 * [36]^{5/2} = 15700\ m^3/s$$

CHAPTER 3

Spillway Alternatives

3.1 Orifice Spillways

3.1.1 General

Orifice spillways are spillways which function by passing water through, instead of over, the structure, but do not include siphon or shaft spillways. These spillways (orifices) are generally used in situations where space is limited or a wide range of reservoir levels is required. However in some situations, the selection of an orifice spillway may be made on an economic basis.

Crippen Acres Engineering, in a generic comparison of orifice and overflow spillways [2,pp2-9], outlined several aspects of orifice spillways which may provide this concept with an economic advantage over the overflow-type spillway. One such advantage results from the reduced or eliminated requirement for second stage concrete. The low level positioning of an orifice inlet usually allows the structure to be used to pass both diversion flows and final design flood flows with essentially the same configuration. Since the optimum crest elevations of overflow spillways are generally too high to allow passage of diversion flows (water levels during diversion must be kept low to minimze cofferdam construction costs), most, or all, of the final rollways have to be placed after the diversion stage of the construction schedule has been completed. The relatively higher heads on low level orifices, in contrast to overflow spillways, allow smaller openings to be used, resulting in smaller, cheaper gates. The low level of the gates may also allow hoisting mechanisms to be located within the structure itself, eliminating the need for a hoist tower. However, for spillways subjected to cold climates, the orifice concept will have the added cost of a heating system for this gate chamber. Both types of spillways require heating of the gates and guides.

One of the features which detracts from the economic benefits of an orifice spillway scheme is the cost of steel armouring for the gate guides. The armouring is required to protect against the increased potential for cavitation damage resulting from higher flow velocities. The lower operational reliability of an orifice spillway necessitates the provision of an emergency gate as a second means of closing a bay if for some reason (eg., due to trash jamming) the main gate is prevented from closing. This is generally not a requirement for an overflow spillway. An orifice spillway may also require an excavated rock trap at its entrance to help prevent debris from collecting in the gate slots. If an orifice spillway is the sole means of conveying flood discharges, the cost of a low-head trash/ice sluice to pass floating debris may have to be added to the cost of the spillway, assuming that passing trash is an acceptable means of cleaning the reservoir.

As mentioned previously, two advantages of the orifice spillway concept are the adaptability to sites with limited space and the wider range of forebay levels it can accomodate. An added operational advantage is that gate cables can be situated within the closed structure, thereby eliminating problems which may arise due to ice build-up.

Offsetting the possible economic and operational advantages of the orifice concept are several potential operational disadvantages. A major concern with the operation of an orifice spillway is the potential for submerged trash to become wedged within the structure, causing a decrease in flow capacity. A decrease in spillway discharge capacity during a flood could ultimately result in a breach of the main dam. Trash wedging could also prevent the closure of a gate, which could result in a loss of revenue due to unwanted spillage. Although trash wedging may also occur with an overflow spillway, gaining access to the blocked flow passage of an orifice spillway would be more difficult.

Compared to the overflow-type spillway, the orifice scheme has a greater potential for gate closure problems due to higher heads and smaller, lighter gates. The increased flow velocities associated with the high heads of an orifice spillway may require that the gates be sealed to prevent air from being drawn through the gate chambers. These wind seals would be in addition to lintel seals used to prevent water from entering the chambers. Cavitation and erosion damage are two more concerns that increase with increased flow velocities.

Operational difficulties unique for orifice spillways located at sites with severe winter conditions arise from the presence of ice in the surrounding waters. Large pans of ice drawn down from the reservoir surface into a partially open orifice spillway may cause significant impact damage to the spillway gate if contact is made. With overflow spillways, large impact loads will not occur for most gate openings since the drawn down pans tend to slide down along the upstream face of the gate. At some sites, there is also the potential for plugging of an orifice by a mass influx of ice pans released from an upstream ice jam. An orifice spillway gate may also be damaged by ice hitting its downstream face when tailwater levels are high. The relatively high positioning of the crest gates eliminates this concern for overflow spillways.

In a review of current operational spillways [2, Appendix A], Crippen Acres Engineering found many precendents for the orifice spillway concept. For parallels to the Wuskwatim site, however, few cases were found in cold climate conditions. The number of precedents in which the orifice scheme was the only method of passing flood flows was found to be limited, with combinations of orifice and overflow spilllways being a common scheme. As a means of improving reliability, current orifice spillways are usually equipped with at least one upstream emergency gate to provide access to an orifice in the event that the main gate is unable to close.

The orifice spillway with conditions most closely approximating those of the Wuskwatim site was found to be the Brisay Control Structure in northern Quebec. Design aspects of this diversion structure, which was not designed to pass flood flows, included a single upstream emergency gate. To reduce the potential for cavitation damage to the structure, the sizes of the orifices were chosen so as not to exceed a maximum flow velocity of 18 m/s. Operational difficulties identified and corrected using model studies included drawdown of ice floes and significant air movement through the gate shaft. The vertical lift gates and the gate shafts of this stucture are heated throughout the winter months.

Another orifice spillway subjected to a cold climate is also located in the province of Quebec. The Lac Ste-Anne outlet structure has six orifice and three overflow spillway bays. Ice growth on the gates and in the gate shafts of the orifice bays was found to be a problem with the gates fully open despite the installation of a hot air heating system in the shafts. Ice growth is avoided by operating the gates in partially open positions during the winter.

A portion of the spilling capacity of the Outardes 2 project in Quebec is provided by six orifice spillway bays. Frequent operating problems have arisen due to outages of the gate heating system. The collection of debris in the gate shafts was another problem experienced at this site.

3.1.2 Wuskwatim Generating Station

The preliminary general arrangement of the orifice spillway scheme for the Wuskwatim Generating Station is presented in Figure 3.1. This arrangement was prepared for the Stage III Concept Studies of the site and includes three orifice spillway bays and a low-head trash sluice.

Each orifice bay has a rectangular opening which is 6 metres wide and 7.4 metres high at the gate location -- a width-to-height ratio which should provide jam-free operation of the gates [2,p12]. The maximum head on sill (i.e., the vertical distance from the spillway invert level to the maximum forebay level) of the orifices is 26.2 metres. Lintel seals prevent water from jetting up the fronts of the gates, while wind seals on the downstream side of the gates and in the gate checks minimize air movement through the gate shafts. The gates, guides, and gate chambers will be heated during the winter. A single emergency gate is included in the design to increase reliabilty of the spillway and will be used in the gate checks located upstream and adjacent to the bellmouth opening of each orifice. The emergency gate will also be operated as an upstream service gate for the orifices. A gantry crane will be used to raise and lower the emergency gate, and will also operate the trash sluice gate.

To decrease the potential of emergency gate closure difficulties, a concrete curtain wall is located upstream of the emergency gate slots to keep the slots free of debris. The curtain wall extends 4.5 metres below the Full Supply Level of the reservoir. A 3 metre deep trash trap is located adjacent to the spillway entrance to minimize the potential for debris to collect in the gate checks.

The chosen method of energy dissipation for each spillway bay is a flip bucket with a 30 degree lip angle and a 20 metre radius of curvature. The flip bucket will be constructed at the end of the diversion stage of the construction schedule. A portion of the spillway floor between the main gate and the flip bucket slopes down towards the bucket to allow for dewatering of this area through drainage pipes embedded in the piers. A temporary bridge crosses over the tops of the downstream piers for use during the construction period.

The diversion stages of the Wuskwatim Generating Station development are the same for both the orifice and overflow spillway concepts and are shown in Figure 3.2. During Stage I diversion, a cofferdam will be placed from an island at the brink of Taskinigup Falls to the east shore of the Burntwood River. While flow is passed through the remaining portion of the falls, construction will begin on the spillway, powerhouse, approach and tailrace channels, and on the transition structures. Upon completion of the spillway diversion structure, part of the Stage I cofferdam will be removed and the Stage II cofferdam will be constructed upstream of the falls. During Stage II diversion (including construction of the Stage II cofferdam) flow will be diverted through the spillway structure via the excavated approach channel. The Stage II cofferdam will eventually become part of the main dam. For the orifice spillway concept, during passage of the diversion flows, the flip bucket will not be in place and the main gates of the spillway will be left fully open.

Following Stage II diversion, the spillway bays will be individually closed off and dewatered for flip bucket construction. During construction of the flip buckets, the Wuskwatim Reservoir will be impounded. Two spillway bays will be available to pass the design construction flow throughout this construction period.

Debris floating in the Wuskwatim Reservoir can be passed through the trash sluice located between the orifice spillway and the powerhouse. The 8 metre wide opening of the sluice has a 3.2 metre maximum head on sill and is controlled with a vertical lift gate.

3.2 Overflow Spillways

3.2.1 General

Overflow spillways are the most common type of spillway used. They are essentially broad-crested wiers whose discharges are normally controlled by either vertical lift gates or by radial gates. The crest shapes of the weirs are designed to approximate the curvature of the underside of a nappe flowing over a sharp-crested weir. This conformity to a natural nappe maximizes the discharge for a given total head and spillway bay width. It also minimizes the potential for the occurrence of cavitation damage resulting from negative pressures along the rollway. The U.S. Bureau of Reclamation (USBR) has developed empirical formulae which can be used to aid in the design of crest curvatures [1,p8]. A generic comparison of overflow and orifice spillways is presented in section 3.1.1 of this thesis.

3.2.2 Wuskwatim Generating Station

The overflow ogee crest spillway configuration selected as a base case for the Stage III Concept Studies of the Wuskwatim Generating Station consists of three 8 metre wide bays. A hydraulic model study to determine the optimum invert level of this configuration during diversion has already been undertaken. However, final optimization of the spillway arrangement for this site will not be made until the Stage IV Concept Studies. Along with the orifice spillway concept, a preliminary overflow spillway design using three 7 metre wide bays has been shown to have economic benefits over the base case. This 7 metre overflow spillway was chosen for the comparison to the orifice concept.

The preliminary design of the overflow spillway chosen for this comparison study is shown in Figure 3.3. The structure will be used to pass diversion flows as well as the required normal operating discharges. During diversion, Bays 1 and 3 will have the form of diversion ports, while Bay 2 will be used as a diversion sluice. The partial rollways forming the diversion ports provide structural support to the spillway piers and allow smaller gates to be used to close off the ports for final concreting. Since Bay 2 does not have a partial rollway, it will be closed first, while the upstream water levels are still relatively low. The crest gate in Bay 2 will be lowered to the sluice invert level so that stoplogs can be placed in the closure stoplog guides to form an upstream seal. A temporary bulkhead will be installed in the guides located at the downstream end of the piers to completely seal the bay. While the rollway and flip bucket are being constructed in the sluice bay, diversion flows will be passed through the two ports.

Following the completion of Bay 2, one of the diversion ports will be sealed. The crest gate will be used to close this port so that stoplogs can be placed in the closure guides up to the lintel elevation of the port soffit. Stoplogs in the permanent guides above the crest in each of the port bays will allow higher water levels during this construction period. Required discharges during the completion of the second spillway bay will be passed over the one completed rollway and through the remaining port.

Two completed rollways will be available to discharge water during completion of the last bay. This bay will be sealed in the same manner as the first port before receiving second stage concrete.

Discharge through the completed spillway bays will be controlled with vertical lift crest gates. These gates will be equipped with fixed rollers and will be operated with a hoist tower. During the winter, the gates and guides will be heated to prevent ice build-up.

Energy dissipation for the high velocity flow exiting the structure is provided in the form of flip buckets. The buckets are seperated from the crest portion of the rollways by flat sections to accomodate the proposed construction bridge which crosses the spillway piers.

CHAPTER 4

Testing Procedures

The comparative spillway study described in this thesis was based on flume model testing of both the orifice and overflow spillway concepts, and comprehensive site model testing of the orifice scheme only. Concerns that were investigated using the two-dimensional flume models included the ability of the structures to pass diversion and final design discharges, the potential for cavitation damage to the structures, the performance of the flip bucket designs, and operating difficulties resulting from the presence of ice or trash.

A more comprehensive, but smaller-scaled, three-dimensional model, which included the proposed powerhouse and main dam of the Wuskwatim site, was used to examine the inlet conditions for the diversion and completed configurations of the orifice spillway. The inlet conditions for the completed configuration were examined for two different sequences of gate operation. Also, many of the results obtained during the flume model study were verified using the site model.

4.1 Flume Model Tests

4.1.1 Model Description

The two-dimensional spillway model tests were performed in a 13.9 metres long by 0.91 metres wide by 0.75 metres high steel and glass hydraulic flume. During a previous model study, the upstream portion of the flume was increased in height by 0.4 metres. This plywood addition was maintained for the Wuskwatim study, to facilitate larger-scaled flume models.

Model discharges were supplied by a closed water circuit which pumped water from an underground supply reservoir to the flume via 8 inch pipes. Water exited the downstream end of the flume and flowed through either of two volumetric rating tanks and back into the supply reservoir (see Figure 4.1). Discharges into the flume were controlled with three sets of valves located between the pumps and the flume. Steel grates and air filter materials were used to dissipate the turbulent water entering the flume. Discharges were obtained by determining the length of time required to fill a closed-off volume tank.

The added height of the flume, as previously described, allowed for the construction of Froudian models of the two spillways built to an undistorted length scale of 36. This scale was close to the upper limit of the normal range of 30 to 100 for spillways [9,p351]. The discharge capacity of the laboratory's pumping system limited the model configuration to include only a single bay for the scale chosen.

Using principles of similitude and the chosen length scale of 36, the scales of other relevent measurements for this study were as follow:

Velocity scale =	$(36)^{1/2}$	=	6
Discharge scale =	$(36)^{5/2}$	=	7,776
Pressure scale =	$(36)^{1}$	=	36
Force scale =	$(36)^{3}$	=	46,656
Time scale =	$(36)^{1/2}$	=	6

The main construction material of the spillway flume models was 1/2 inch acrylic plastic. Aluminum sheeting (1/16 inches thick) was used for most of the curved surfaces -- soffits, flip buckets, and rollway. Both of the flume models were constructed with removable sections to allow for design changes and for easy conversion of the structures from their diversion form to their completed configuration. Piezometric pressure taps were installed in assumed critical locations along the right piers and floors of the spillways. Taps were also located along the centerline of the soffit of the orifice spillway and the rollway of the overflow spillway. Pressures were read on a connecting manometer board.

The spillway models were installed asymetrically in the flume, where the built-up plywood walls met the shorter, glass walls. The asymetric placement was chosen to create enough clearance on the right side of the model to install a manometer board inside the flume. Upstream conditions in the flume (see Figure 4.2) were constructed of plywood and extended 101.2 metres (all distances are prototype values, unless otherwise indicated) upstream of the spillway entrance location. The horizontal scale of the approach channel, in the longitudnal direction, was distorted to accommodate the limited space available. For better representation of the relatively straight flow lines anticipated in the approach channel during diversion, low-level walls were extended from the pier noses to decrease the effective width of the flume. Features downstream of the spillway were not modelled, since the tailwater levels expected at the prototype site are not considered to be high enough to have an effect on upstream conditions [3,pp3-4].

Upstream water levels in the flume were measured using a point gauge positioned 113.2 metres upstream of the spillway entrance location. The point gauge was 56 mm (model) away from the flume wall and was placed within an open-ended, clear acrylic standpipe. Masonry blocks were placed around the standpipe to decrease water fluctuations inside the pipe while testing the spillways in their completed configurations.

Orifice Spillway

The single-bay flume model of the orifice spillway concept (Plate 1) was based on the preliminary drawings presented in Figure 3.1. The soffit and flip bucket sections were made removable so that design changes to these features made during the study could be reflected in the model. The sectional feature of the model also eased the task of converting the structure from its diversion to its completed phase, by simply installing the flip bucket section.

Acrylic plastic (1/2 inches thick) was used for almost all of the flume model, with the exception of the curved surfaces of the three soffits that were tested and one of the three flip buckets tested. Aluminum sheeting was used for these curved surfaces, while thin acrylic was used for subsequent flip buckets, due to material availability. Aluminum was also used to reflect design changes to a portion of the spillway floor. Plastercine was used to fill any small gaps at joints and to cover the heads of counter-sunk screws.

A sealable, acrylic gate with an aluminum lip, was used for tests that required partial gate openings. This gate precisely modelled the bottom of the proposed prototype gate, as well as its positioning within the gate guides.

Upstream flume conditions for the orifice spillway model differed from those for the overflow model by the inclusion of a trash trap adjacent to the entrance of the orifice.

Overflow Spillway

The flume model (see Figure 4.2) of the overflow spillway concept represented a single bay of the three-bay preliminary prototype design shown in Figure 3.3. When neither the partial nor completed rollway sections were installed, the model reflected the diversion sluice configuration. Installation of the two partial rollway pieces transformed the sluice model into a diversion port model. A model of a completed spillway bay was obtained by removing the diversion port and installing the completed rollway sections. The model rollway was constructed based on the design presented in Figure 4.3. When installing the rollways, the downstream portion of the diversion structure floor, beneath the rollway surface, was removed to make room for the tubes which connected the rollway pressure taps to the manometer board.

Similar to the construction of the orifice flume model, acrylic plastic was used for almost all of the components of the overflow model, except for the curved surfaces of the soffit and the rollway, which were made of 1/16 inch aluminum sheeting. Aluminum was also used for the lip of the crest gate.

4.1.2 Invert Level Tests

A primary function of the spillway at the Wuskwatim Generating Station will be to pass river flows during Stage II diversion. Therefore, tests were performed to determine if the preliminary invert level of each of the spillway options was low enough to meet the design criteria for the diversion conditions. The criteria to be satisfied included a design flow passage of 1100 m^3/s for the 230.0 m maximum allowable upstream water elevation during closure of the Stage II cofferdam. A design flow of 1250 m^3/s was required when the upstream water level was at its maximum elevation of 230.5 m during Stage II diversion. The 1250 m^3/s discharge was also used as the design flow through two bays, during second stage concreting of the spillway. Water levels were allowed to rise to elevation 240.0 m during this construction period, which coincides with the impounding of the reservoir.

Diversion discharges at the Wuskwatim site will be directed from the natural river channel to the diversion structure via an excavated approach channel. However the two-dimensional characteristic of the flume could not properly model the head losses expected in the prototype approach channel. Since the design upstream water elevations for diversion pertain to the cofferdam location, losses in the approach channel had to be estimated for the flume model study. Using upstream water levels that were 0.5 metres below the design elevations was considered to be a reasonable approximation of losses for the purpose of confirming the invert level settings of the structures.

Rating curves for the diversion structures were obtained by measuring the discharges conveyed by each of the single-bay spillway models for several values of upstream water levels. For each water level setting, the discharge was measured three times with each of the two volumetric rating tanks.

Orifice Spillway

The diversion configuration of the orifice spillway differs from the completed form by the absence of the flip buckets. During diversion, all three vertical lift gates will remain fully open. For the invert level tests, therefore, the model flip bucket section was not installed and the gate was not used. The invert level of the spillway was set at elevation 218.5 m [2,p12].

Overflow Spillway

The diversion structure of the overflow concept has two ports and one sluice. The sluice is a totally open channel, whereas the ports act as orifices when the upstream water levels become high (i.e., during second stage concreting of the spillway). The vertical lift gates of the structure will remain fully open during Stage II diversion. During construction of the spillway's final rollways, the design discharge of 1250 m^3/s will apply to the following three combinations of open bays: two ports, one port and one completed rollway, and two completed bays.

During the invert level tests, the flow capacities of only the sluice and ports were determined and the bays with completed rollways were assumed to have sufficient capacity to each pass at least fifty percent of the required 1250 m^3/s with the upstream water level at elevation 240.0 m. For diversion port tests in which the upstream water levels were high enough to submerge the port, a piece of plywood was used in the closure stoplog guides to seal the area above the partially completed rollway.

A model of the diversion sluice was obtained by removing the soffit sections of the diversion port. The preliminary design drawing (Figure 3.3) of the overflow spillway indicated a diversion port/sluice invert level of elevation 219.0 m.

4.1.3 Soffit Design

Orifice Spillway

The determination of an adequate soffit design for the orifice spillway was scheduled to follow the selection of an invert level, due to the potential effects that the soffit design may have on subsequent tests, such as the discharge rating. In an effort to decrease the potential for cavitation damage, the main criterion chosen for determining an acceptable soffit design was the minimization, or elimination, of negative pressures along the model soffit. The soffit section of the flume model was made removable so that changes to the soffit curvature could be modelled and tested, if necessary.

Prior to the installation of the preliminary soffit design, tests were performed to determine the water surface profiles of jets of water released from the upstream lintel location. Paralleling the theory of overflow crest design, which approximates the underside of a nappe flowing over a sharp-crested wier, these profiles were to be used in the design of alternative soffits, in the event that the preliminary design proved to be unacceptable.

The jets of water were created by placing a sharp-edged gate in the spillway's emergency gate checks so that the lip was set at the soffit's bellmouth opening location, 11.7 metres above the invert level. The Winter Full Supply Level, elevation 242.0 m, and the maximum level during passage of the peak maximum flood, elevation 244.7 m, were used as upstream water levels in the flume to create two different jet profiles. A point gauge was used to measure the centerline of each jet profile between the emergency and main gate checks, with horizontal length intervals of 0.5 metres.

With the soffit section installed in the model, pressures were measured with piezometric pressure taps at three points along the centerline of the soffit. Taps in the right pier located 0.5 metres and 0.75 metres below the soffit, at the same longitudinal position as each soffit tap, were also used to record pressures (see Figure 4.4). Pressures were measured with the main gate in the fully open position and the upstream water level set at the Winter Supply Level and the maximum allowable level during passage of a flood. These levels were assumed to be the lower and upper bounds of the forebay level for operation of the completed spillway. Discharges were also measured for each water level using the volumetric rating tanks.

Overflow Spillway

The two diversion ports of the overflow spillway concept have curved soffits along the underside of the partially completed rollways. Due to the relatively short time duration in which water will be in contact with the prototype soffits, hydraulic modelling of the proposed design was not considered necessary. The model soffit was constructed as shown in the preliminary design drawing.

4.1.4 Discharge Rating

The orifice and overflow spillway flume models were rated in their completed configurations with their gates fully open to determine if the preliminary prototype designs will be capable of conveying the design flood of the project without exceeding the maximum design forebay level. The project design flood for the Wuskwatim Generating Station was taken as the Probable Maximum Flood (PMF), which was estimated to be approximately 2715 m^3/s. Along with the rating of the fully open structures, rating curves for several gate openings were also created for each alternative. A point gauge was used to measure upstream water levels and two volumetric rating tanks were used independently to measure discharge.

Orifice Spillway

Although a required peak outflow was estimated for the overflow spillway, no such value was quantified for the orifice option, which will have a different outflow hydrograph. However, it was assumed that the orifice spillway will have greater discharge capacity at the beginning of a flood event than the overflow spillway, and therefore the initial rate of reservoir attenuation will be lowered and the peak outflow required will be decreased.

Overflow Spillway

In determining a maximum required outflow for the overflow spillway it was assumed that the reservoir will be at its Full Supply Level (elevation 243.2 m) at the start of a flood runoff, and that the concrete structures will have a 1.5 metre maximum allowable surcharge above this level. Based on these assumptions, the required outflow through the preliminary overflow spillway was estimated to be 2450 m^3/s at elevation 244.7 m.

4.1.5 Pressure Measurements

The potential for damage due to cavitation was assumed to be greater for an orifice spillway than for a comparable overflow spillway. Since orifice spillways will generally have higher heads and smaller outlet openings, it can be expected that flow velocities will be greater. If average velocities tend to be higher, then local velocities at similar irregularities on the flow boundaries of hydraulic structures will also be higher. The increased kinetic energy resulting from high local velocities will be accompanied by a reduction in the potential energy in the form of reduced pressures [9,p359]. If the pressure drops to the vapour pressure of water, small cavities of water vapour will form and be carried with the flow. Upon reaching a location where pressures exceed the vapour pressure, the small cavities will collapse. The rush of water into the collapsed cavities results in very high pressures which can be damaging to adjacent boundaries [10,p35].

To obtain an indication of the potential for cavitation damage occurring on either of the Wuskwatim Generating Station's spillway alternatives, each was tested in the flume by checking for low pressures at potential trouble spots along the flow boundaries. Pressures were measured with piezometric pressure taps downstream of the gate slots and at locations where the floor elevations change. Since the precise location of critical pressures was unknown, two taps were installed at many locations.

Veterinarian hypodermic needles were used for piezometric pressure taps. The sharp tips of these needles were blunted with a grinder so that their ends would be flush with the boundary surfaces when inserted into holes drilled through the acrylic model segments. The holes drilled through the model walls and floors were made on a drill press to ensure that they were perpendicular to the boundary surface. Plate 3 shows a pressure tap and connecting hose, as well as the end of an installed tap. A suggested diameter size limitation of 1 mm to 1.5 mm (model) [10,p72] for piezometric openings was adhered to in the selection of the needles, which had an inside diameter, d, of 1.1 mm (model). The 12.7 mm (model) minimum needle length used also satisfied the recommended lower bound of 2d [10,p72]. Each tap was connected with a hose to a rigid, vertical tube, which was fixed to a manometer board. A grid was attached to the manometer board behind the tubes so that model pressure heads could be easily determined to the nearest millimetre.

Each piezometric pressure tap was zeroed by running pressurized water through the open end of its manometer tube, until all of the air bubbles in the tube and hose had exited through the needle end of the system. Once free of air bubbles, the water in the tap apparatus was allowed to equalize (i.e., the high water level in the manometer tube forced water out of the needle end until the water level in the tube was at the same elevation as the needle) and the zero level was recorded. When running the flume models, pressurized water was again forced through each tube to ensure that no air had entered the system. The water level in each tube above or below its zero level was taken as the model pressure head at the pressure tap location. A tube level above the zero level indicated a positive pressure, while a level below the zero indicated a negative pressure.

Apart from the orifice spillway soffit tests, pressure measurements were made during the rating of the spillways.

Orifice Spillway

The orifice spillway flume model was equipped with a total of forty-one pressure taps, as shown in Figure 4.5. These taps were located along the right pier wall and the centerlines of the soffit and floor. Thirty-one taps were applicable for diversion flows, while a maximum of thirty-two taps were available during testing of the completed structure. Points of particular concern regarding pressures included the area along the wall just downstream of the main gate check, along the centerline of the soffit and where the spillway floor slopes downward.

Overflow Spillway

A total of fifty pressure taps were installed in the flume model of the overflow spillway. The twenty-three taps used during testing of the diversion port/sluice are shown in Figure 4.6. Figure 4.7 indicates the location of the thirty-six taps used for the completed configuration. The most probable locations for the occurrence of low pressures were considered to be along the wall downstream of the gate checks, along the floor downstream of the rollway crest and the sloped section of the diversion port/sluice floor.

4.1.6 Flip Bucket Design

The final decision regarding the type of facility to use for energy dissipation for the Wuskwatim spillway was not made during the Stage III Concept Study of the site. Although the final decision was left for the Stage IV study, a preliminary flip bucket concept was adopted during Stage III. The choice of the flip bucket concept was based on the presence of competent bedrock downstream of the spillway, the limited use of the structure following the diversion stages of the generating station construction and the relative expense of a stilling basin alternative [8,p10-15]. Although there is competent bedrock downstream of the spillway location, it is anticipated that some form of energy dissipation will be required since the head and energy per unit width of the proposed spillway alternatives are greater than those of present Canadian spillways built without such facilities [8,p10-15]. If any scour of this bedrock does occur downstream of the spillway, the flip bucket will be required to throw the overfall jet a sufficient distance such that the toe of the structure will not be jeopardized.

The flume models were used to determine the throw distances provided by the preliminary flip buckets of the two spillway alternatives. Throw distances were measured from the flip bucket lip to the centre of the jet trajectory at elevation 217.0 m for several combinations of gate openings and upstream water levels. To determine throw distances, a point gauge was attached to the flume's rolling bridge and its point was set at elevation 217.0 m. The distances from the flip bucket lip to the upstream and downstream boundaries of the centerline of the jet trajectory were measured, and the average of these values was recorded as the throw distance.

Orifice Spillway

Crippen Acres Engineering provided a theoretically based estimate of the relationship between single bay discharge and the throwing distance of the preliminary flip bucket design of the orifice spillway. The model throw distances were measured and compared to the estimates.

Overflow Spillway

Throw distance were measured for the flip bucket design shown in Figure 4.3 as opposed to that shown in the preliminary design drawing presented in Figure 3.3.

4.1.7 Ice Tests

A concern regarding the operation of an orifice spillway in a northern climate is the potential for damage to a partially opened spillway gate resulting from the impact of a drawn down ice floe. This is a greater concern for an orifice spillay than for an overflow spillway since for most gate openings of the latter, entrained ice pans will tend to slide down along the upstream face of the gate, inflicting only glancing impacts [2,p3].

At the Wuskwatim Generating Station site, use of the spillway during winter months will be limited to periods in which more than one of the four generating units is shut down [2,p14]. During such a plant shutdown, the spillway gates could be partially opened to provide enough flow capacity to prevent a significant rise in the reservoir level. Tests were performed to determine the maximum gate opening that could be used for each of the spillway alternatives without causing ice to be drawn down. The maximum allowable gate opening would also give an indication of the length of time in which an orifice gate would be subjected to possible impact damage if it had to be raised to its fully open position.

The material chosen to model ice floes for the tests was low-density polyethylene because its specific gravity of 0.92 is similar to that of river ice [14,p194]. The sizes of the three square ice pans used in the tests were 1.5, 3.0 and 5.0 metres, with a thickness of 0.6 metres. With the upstream water level maintained at the Winter Full Supply Level, and the main gate of the flume model set at an initial opening, the three ice pans were individually placed in the water upstream of the spillway entrance to determine if they would be drawn through the structure. The procedure was repeated for various gate openings to acertain, to the nearest 0.5 metres, the smallest gate opening at which each floe was drawn down.

The modelling of ice drawdown is complicated by the higher surface tension forces and lower vorticity of the model [3,pp2-3]. The increased significance of surface tension on the drawdown of a model ice floe can be illustrated by the following example. A 1.5 metre square, 0.6 metre thick ice floe with a specific gravity of 0.92 will have a buoyancy force of approximately 2380 Newtons (N). Assuming a surface tension of 0.073 N/m, the resulting surface tension force acting around the circumference of the floe will be 0.44 N. For a Froude model, a piece of low-density polyethylene plastic 1/36 the size of the prototype ice floe will have a buoyancy force of 0.051 N and a surface tension force of 0.012 N. While the surface tension force is only 0.02 % of the magnitude of the buoyancy force in the prototype the corresponding value for the model is 24 %. During the ice drawdown tests the model floes were tapped lightly below the water surface to break the surface tension [3,p3].

Ice drawdown is affected by vorticity [3,p3], and therefore any misrepresentation of vorticity by a model can result in misleading ice drawdown results. Forces which affect vorticity include gravity forces, viscous forces and surface tension forces [10,p186]. Of these forces, only those resulting from gravity were considered when forming the Froudian scaling relationships. Although the effects of surface tension forces have been found to be small, operating a Froude model with prototype velocities may be required to compensate for scaling errors of the viscous forces [14,p79]. However, it should be noted that the increased velocity will distort basic flow patterns [14,p79]. An alternate method of promoting a more violent vorticity in the model is to decrease the submergence of the spillway opening [3,p3]. For this research, the submergence of the crest gate and orifice were reduced by 10 % and 15 % to determine the effects on ice drawdown.

4.1.8 Trash Tests

The inevitable presence of trash in an impounded reservoir creates a possible disadvantage for the orifice spillway option. The unknown potential for trash wedging of the orifice, or its gate checks, may result in significant clearing difficulties, given the decreased accessibility to the flow passage region of this type of spillway. To reduce the risk of gate closure problems, the orifice spillway design includes an emergency gate located upstream of the orifice opening which can be used if the main gate becomes jammed. The preliminary design of the Wuskwatim site with an orifice spillway includes a small trash/ice sluice located between the spillway and the powerhouse. This sluice would be used periodically to pass floating trash if future environmental studies find this method of reservoir cleaning to be acceptable.

The hydraulic model study of the spillways included tests in which the general behavior of simulated floating and submerged trash could be observed when released upstream of the spillway. Realistic trash jamming scenerios could not be achieved easily due to difficulties in scaling the strength of the model trash and the drag forces exerted on it. For the model to accurately simulate the fracture of logs, which affects the trash jamming potential, the model logs would have to be made of a material which has the same density as a prototype log, but with a significantly reduced strength [4,p4]. For the cases studied, the scaled drag force exerted on a simulated log would be greater than that exerted on the prototype. Increased drag forces on the model trash may also affect its jamming potential.

Orifice Spillway

Three sets of trash tests were performed using the single bay flume model of the orifice spillway alternative. The objective of the first set of tests was to observe the ability of the model to pass large pieces of saturated trash released in the upstream approach channel. Several constant gate openings were used with the upstream water elevation at the Summer Full Supply Level. The logs simulated were approximately

12 metres long with 0.2 metre diameters. Scaled-down twigs were used to model branchless trees and were submerged in water for 2 1/2 months prior to testing to achieve a level of saturation sufficient to cause them to sink. The pieces of trash were released both individually and in groups of 8 to 10.

The behavior of trash floating on the water surface in front of the spillway was studied in the second set of trash tests. Tests to determine the maximum allowable gate opening to prevent the drawdown of floating trash is highly dependent on the saturation level of the trash and a wide spectrum of saturation levels will be encountered in the prototype condition. Twigs were used to simulate eight floating trees similar in size to those modelled in the first set of trash tests. The pieces of trash were released individually upstream and were allowed to accumulate in front of the spillway. Various gate openings were used with the upstream water elevation at the Summer Full Supply Level.

The third set of tests modelled situations in which large masses of saturated trash accumulate and settle in front of the spillway entrance while its gates are closed. The modelling of this situation was achieved by piling a mass of simulated trash in front of the flume model prior to starting the pumping system. With the spillway gate fully closed the upstream end of the flume was slowly filled with water. Once the

Summer Full Supply Level was reached, the spillway gate was slowly opened and the discharge into the flume was correspondingly increased to maintain a constant upstream water level. The movement of the trash was observed until all of the trash had been drawn through the spillway or until the gate was completely open and the trash was no longer moving.

In light of the fact that currents in the prototype forebay will be directed towards the powerhouse while the spillway is closed, a scenerio was proposed in which a large portion of the trash in the spillway trash trap will be located in front of the bay closest to the powerhouse (i.e., Bay 1). Tests were performed to determine the effects of opening Bay 3 prior to opening Bays 1 or 2. This was accomplished by placing or packing the simulated trash mass in the trash trap adjacent to the flume wall located farthest from the single-bay spillway model.

Other variations in the test set-up included covering the 3 metre deep trash trap, removing one wall of the approach channel, packing or not packing the trash, and changing the percentage of large logs within the trash mass. Table 4.1 provides the percentage of trash sizes represented in the mass for most of the saturated trash tests.

Overflow Spillway

Tests were performed to observe the behavior of a large amount of floating trash released upstream of the spillway. Various gate openings were used while the forebay remained at the Summer Full Supply Level. The sticks and twigs used in the submerged trash portion of the orifice spillway test program were allowed to dry-out for use in these tests.

4.2 Comprehensive Model Tests

A Froudian site model (see Plate 4) with a length scale of 64 was used to study the inlet conditions for the orifice spillway only. Figure 4.8 shows the area represented by the model, which included the powerhouse, central transition structure, spillway, south transition structure and part of the main dam.

The selection of model boundaries was removed from the model design process, since the model had previously been constructed for a river closure study. Construction and installation of the aforementioned structures and rehabilitation of the model's overburden were required steps to prepare the model for the spillway tests.

The bedrock contours of the prototype site were simulated in the model with a 3 to 4 cm layer of concrete which was supported by a sand and wooden substructure. Sand was also used to model the overburden and the main dam. A sheet of plastic was incorporated in the model dam to act as an impermeable core which would minimize leakage. Wood was used to construct the transition structures and powerhouse. The intakes of the four-unit powerhouse were modelled according to preliminary drawings and the structure was calibrated to pass its design discharge of 1430 m^3/s [8,pES-10] with the forebay at the Summer Full Supply Level.

The model of the spillway (Plate 5) was constructed from acrylic plastic and aluminum sheeting. Although the spillway model design for the site model was based for the most part on the same preliminary drawings as the spillway flume model, certain changes proposed by the project's consulting engineers were specified based on the results of the flume model study. The changes included a three metre vertical extension to the bottom of the curtain wall, a convex rounding of the base slab where it changes elevation, and an increase to the downstream pier elevation to 228.4 m. An invert level of elevation 217.9 m and Soffit 3 were also adopted.

A closed water circuit supplied water for the site model tests. Water entered the upstream end of the model via two wier boxes. The larger of the two wier boxes was used to measure low discharges and an orifice meter located along the supply pipe provided measurements of high discharges. Baffles placed within, between and downstream of the weir boxes helped diffuse the water entering the model. Near the upstream edge of the model, a drain pipe was placed flush with the bedrock at an area of low elevation. The drain pipe was connected to a vertical pipe which contained the point gauge used for measuring upstream water levels. A point gauge was also placed downstream of the model to measure tail water levels.

Using the same Froudian scale relationships used for the flume model, the scales used for the comprehensive site model with a length scale of 64 were as follows:

Velocity scale = 8

Discharge scale = 32768

4.2.1 Diversion Tests

The comprehensive site model was used to confirm the rating of the orifice flume model. Upstream water levels between elevations 226.0 m and 230.5 m were measured with the point gauge located upstream of the cofferdams. For each upstream water level, the head inside the large 90 degree V-notch wier box was measured and used to calculate the discharge. Flow conditions in the approach and exit channels of the spillway were also noted.

During the final concreting of the spillway, when the flip buckets will be constructed one at a time, the forebay will be impounded to a maximum level of elevation 240.0 m. The design discharge for this construction phase will be the same as the stage II diversion discharge of 1250 m^3/s, although for the most part there will be only two spillway bays available for passing the flow. The suggested sequence of flip bucket construction which was investigated on the site model was

Bay 3, then Bay 2 and lastly Bay 1. With the model, the discharge was set to 1250 m^3/s while all three bays remained fully open and were without flip buckets. After the upstream water level was recorded Bay 3 was closed. With Bay 3 closed, the level of the stabilized forebay level was measured and the inlet and downstream conditions were noted. The following steps included installing a flip bucket section in Bay 3 and closing Bay 2. When the stabilized conditions had been recorded, Bay 2 received a flip bucket and was opened, and Bay 1 was closed.

4.2.2 Discharge Rating

The orifice spillway was rated for the comprehensive model for comparison to the data obtained with the single-bay flume model. The model was rated with each of the three bays individually fully open as well as with all three bays completely open. When each bay was rated individually, the powerhouse was operated at full capacity. However, due to pumping system discharge limitations, the powerhouse was completely closed while rating all three bays fully open. The spillway was also rated with one bay, Bay 3, operating at gate openings of 2, 4, and 6 metres while the powerhouse was fully open. Inlet and outlet conditions for each of the aforementioned situations were also recorded.

4.2.3 Operating Procedures

Two preconceived sequences of gate operation were investigated with the comprehensive model using gate opening intervals of 2 metres (i.e., 0, 2, 4 and 6 metres and fully open). In the first sequence, Bay 3 was initially opened while Bays 1 and 2 remained closed. When Bay 3 was fully open, the lifting of the Bay 2 gate began until it too was completely open. Bay 1 was then opened. The second sequence consisted of each bay being opened to the 2 metre mark in the same order as in the first sequence. Once all three gates were 2 metres above the invert level, they were simultaneously raised until they were all fully open. An upstream water level of elevation 243.2 m was used for all of the situations tested and the inlet and outlet conditions for each were noted. Although all four units of the powerhouse were operating at the begining of each sequence, it became necessary to close some units as the discharge through the spillway was increased.

4.2.4 Ice Tests

The ice drawdown tests performed in the hydraulic flume were repeated, with some variation, using the comprehensive model. The purpose of the repeated tests was to determine the effects of three-dimensional flow on the drawdown potential of the orifice. The three spillway bays, the operating powerhouse and the wider upstream conditions of the site model would all affect vorticity, and therefore ice drawdown. For all of the tests, powerhouse Units 3 and 4 were operated, simulating a shutdown of the remaining two units. The maximum spillway gate opening to prevent drawdown was determined for the condition of all three bays open the same amount and for just Bay 3 open. A second variation included the addition of an ice cover around the spillway entrance. The floating styrofoam cover (see Plate 6) had a prototype opening of approximately 19 metres by 37 metres.

As with the flume model, there were scaling errors in surface tension and viscous forces. For this reason the forebay level was lowered to increase vorticity by decreasing orifice submergence, and the square ice pans (1.5, 3.0 and 5.0 metre sizes) were tapped lightly to break the surface tension.

4.2.5 Trash Tests

The comprehensive model was used to study only trash that floated. During the flume model study, tests were done in which trash was piled up in the trap at the spillway entrance. Since those tests simulated a scenerio in which floating logs gather in front of a non-operating spillway and then sink when saturated, the comprehensive model was used to determine if logs would in fact come to rest by the spillway when its gates were closed. With the powerhouse units open and the spillway closed, trash was scattered across the forebay to see if any would get caught in front of the spillway. Trash was then released as a group at the west end of the main dam with the powerhouse open for the following conditions: 1. spillway and sluice closed, 2. Bay 1 open, sluice closed, 3. Bays 1 and 2 open, sluice closed, and 4. spillway closed, sluice open. The pieces of trash used for the tests were taken from the overflow flume model tests and included logs with lengths of 4.5, 6.4 and 9.6 metres, as shown in Plate 7.

CHAPTER 5

Results and Discussion

5.1 Flume Model Tests

5.1.1 Invert Level Tests

Orifice Spillway

The determination of the adequacy of the initial invert level (elevation 218.5 m) of the orifice spillway was accomplished by measuring the discharge capacity of the flume model for the maximum upstream water elevation during Stage II diversion (230.5 m) and during the closure of the Stage II cofferdam (230.0 m). Assuming a 0.5 metre head loss in the prototype approach channel, the discharge measured for the upstream water elevation of 230.0 m in the model was taken as the capacity during Stage II diversion. With this method, it was found that the diversion structure was capable of conveying a maximum diversion flow of approximately 1068 m^3/s, which was 182 m^3/s (15 %) below the design discharge of 1250 m^3/s. This deficiency necessitated a change to the preliminary design of the orifice spillway.

The invert level of the orifice spillway was lowered by 0.6 metres to elevation 217.9 m as a means of increasing the capacity of the diversion structure. The lowering of the invert level was modelled in the flume study by raising the soffit section of the model by the equivalent of 0.6 metres, and by recalibrating the upstream point gauge relative to the new invert level. With the model adjusted to reflect the new setting, the diversion structure was rated for a range of water elevations between 226.0 m and 230.5 m. Adding 0.5 metres to the upstream water levels to account for channel head losses resulted in the discharge rating curve shown in Figure 5.1, which also contains the rating data for the structure set at the preliminary invert level. Although the discharges measured for the maximum water levels during Stage II cofferdam closure and diversion were slightly below the design values, they were considered to be sufficient.

Overflow Spillway

The discharge for the diversion ports and sluice was rated with the spillway set at its preliminary invert level of elevation 219.0 m. Water began to touch the lintel of the port soffit when the upstream water elevation was approximately 228.3 m. This level was taken as the point at which the sluice rating diverged from the port rating. Figure 5.2 presents the single-bay discharge rating curve for each diversion configuration. Since the rating curve includes water levels which significantly submerge the approach channel no adjustments for head losses were made on the figure (i.e., all water levels shown are as measured in the model).

From Figure 5.2, it can be seen that for a prototype water level of elevation of 230.5 m (using elevation 230.0 m on the rating curve to account for assumed channel losses, since the approach channel is not submerged at this level) the Stage II diversion discharge for two ports (2 * 370 m^3/s) and one sluice (395 m^3/s) was approximately 1135 m^3/s. Similarily, using elevation 229.5 m on the rating curve for the maximum upstream water level (elevation 230.0 m) during Stage II cofferdam closure resulted in a three-bay total discharge of 1070 m^3/s (2 * 350 + 370 m^3/s). These total values fall below the respective target values of 1250 m^3/s and 1100 m^3/s.

Neglecting channel head losses for high upstream water levels, two ports were found to be capable of passing 1330 m^3/s (2 * 665 m^3/s) when the forebay was at elevation 240.0 m. The required discharge capacity for this forebay level was 1250 m^3/s.

As a result of the inability of the diversion structure to pass the design flows, it was decided to rate the diversion structure with a new invert level of elevation 218.5 m. The new invert level was reflected in the model by raising the port soffit 0.5 metres and recalibrating the upstream point gauge relative to the new floor elevation. The results of the discharge rating are presented in Figure 5.3 with the sluice curve diverging from the port curve at elevation 228.6 m. Using the same method as before for approximating approach channel head losses, the following were the discharges measured for the diversion structure with the new invert level of 218.5 m: 1240 m^3/s (2 * 410 + 420 m^3/s) for Stage II diversion, 1170 m^3/s (3 * 390 m^3/s) for the closure of the Stage II cofferdam, and 1440 m^3/s (2 * 720 m^3/s) through two ports with a forebay level of elevation 240.0 m. While the Stage II diversion flow is 10 m^3/s below the design discharge, the other two measured discharges exceed the corresponding target values.

While rating the diversion structure, water profiles were measured along the left pier. The profile recorded when the upstream water level was at its highest value, elevation 240.0 m, is shown in Figure 5.4.

5.1.2 Soffit Design

Orifice Spillway

The water surface profiles of jets of water released from the upstream lintel location of the spillway were measured prior to obtaining soffit pressure readings. Following the measurement of these profiles, the soffit section of the spillway model was installed. The initial soffit shape, Soffit 1, was based on measurements taken from the preliminary spillway design (Figure 3.1). Pressure measurements were made at the nine tap locations shown in Figure 4.4 for upstream water elevations of 242.0 m and 244.7 m. The lowest pressure recorded, -1.73 metres, was measured at Tap 1 with the upstream water level at the lower elevation. All three taps located along the soffit indicated negative pressures for both water levels, while the wall taps recorded positive pressures. Although pressures in the order of 9 metres below atmospheric are considered indications of potential cavitation problems in the prototype [14,p86], the presence of the small negative pressures along the model soffit centerline led to the design of a second soffit, Soffit 2.

Pressures along the centerline of Soffit 2 were also lowest at Tap 1 with the water level set at elevation 242.0 m. However, at this water level, pressures did reach a positive value, going from -1.40 metres at Tap 1 to -0.40 metres at Tap 2 and +0.36 metres at Tap 3. The pressures at each of these locations were slightly higher when the forebay level was raised. All of the wall taps indicated positive pressures. The negative pressures measured along the soffit again prompted a redesign of the soffit curvature.

Figure 5.5 shows the profiles of the first two soffits and the new soffit design (Soffit 3), as well as the water jet profile measured prior to the installation of the soffits. Comparison of Soffit 2 to that of the water jet shows that the beginning of the soffit's curvature is above the natural flow line, which suggests that a flow separation at this point may be causing the negative pressures at Taps 1 and 2. Soffit 3 was similar to Soffit 2, with the main difference being that the initial constant-radius curvature of Soffit 2 was changed to a parabolic shape to cause Soffit 3 to impinge on the natural nappe along its entire length. Pressure measurements along Soffit 3 indicate positive pressures at all three of the soffit tap locations. The lowest of these values was +0.11 metres at Tap 2, with the upstream water level at elevation 242.0 m.

Table 5.1 shows the pressures which were recorded at all nine of the pressure tap locations for each of the soffit designs, while Figure 5.6 graphically presents the pressures measured along each of the soffit centerlines.

5.1.3 Discharge Rating

Orifice Spillway

The single-bay flume model was used to rate the orifice spillway for a configuration which included Soffit 3 and an invert level of elevation 217.9 m. Separate rating curves were determined for the fully open condition, a gate opening of 0.5 metres, and all full-metre openings between (i.e., 1.0, 2.0 , ... , 7.0 metres). For each gate opening, thirty discharges were measured for various upstream water elevations between 242.0 m and 244.7 m. The discharge for the maximum upstream water level (elevation 244.7 m) during passage of the PMF was found to be approximately 2390 m^3/s. This value was considered to be adequate, based on the estimate of the required flow of 2450 m^3/s for the overflow spillway. Figure 5.7 shows the discharge rating curves for each of the gate positions tested. Discharges in this figure are in three-bay prototype values. The rating of the structure in its diversion configuration was included in the Invert Level Tests section of this thesis.

Overflow Spillway

The overflow spillway alternative was rated in both its diversion and final configurations. The Invert Level Tests section of this thesis contains a rating curve for the diversion structure set at the preliminary invert level (elevation 219.0 m) as well as a modified invert elevation of 218.5 m, which was adopted for future tests.

The required change to the invert level of the diversion ports/sluice and a modification (Figure 4.3) to the rollway design made exact modelling of the prototype spillway difficult, due to the inflexibility of the model's pier tops and gate checks. For this reason the vertical distance between the floor of the diversion structure and the crest elevation (230.5 m) of the completed rollway in the model was not exactly to scale. All elevations in the model were therefore taken with respect to the crest elevation only. This resulted in a model invert elevation of 218.81 m instead of 218.5 m for the tests of the completed structure. This difference was assumed to have a minimal affect on test results. Figure 5.8 shows the cantilevered section of the pier tops as given in the preliminary design drawing and as constructed in the flume model.

Results of the rating of the completed configuration of the spillway for free flow conditions and for partial gate openings (1.0, 2.0, 4.0, 6.0, and 8.0 metres) are presented in Figure 5.9. As can be seen in this figure, the spillway did not pass the design discharge with the forebay at its PMF level. The three-bay equivalent discharge of the model at the maximum design water elevation of 244.7 m was approximately 2250 m^3/s, or 200 m^3/s below the design discharge.

Given the equation

$$Q = C * W * H^{3/2}$$

where Q is discharge, W is the spillway bay width, and H is the total head, the discharge coefficient, C, for the spillway model passing 2250 m^3/s was calculated to be 3.63. The coefficient which would be required for an overflow spillway with the given width and crest elevation of the preliminary Wuskwatim structure would be 3.95 for the three bays to convey the required flow of 2450 m^3/s.

Model conditions that were altered and tested for the effects on discharge included: filling the gate checks to create smooth pier walls, removing the spillway bridge, and decreasing the upstream flume width. The flume width was decreased by extending upstream from the pier noses two parallel, 33 metre long walls. As can be seen in the free surface rating curves in Figures 5.10 to 5.12, each of the alterations had little or no effect on the discharge rating. From Figure 5.9 it can be seen that a single completed rollway can pass approximately 400 m^3/s when the forebay level is at elevation 240.0 m. While the second spillway flip bucket is being constructed on the prototype, one diversion port and one completed rollway will be required to pass 1250 m^3/s with an upstream water elevation of 240.0 m. Given that one port can discharge 720 m^3/s at this level and a completed bay can pass 400 m^3/s, the combined capacity is only 1120 m^3/s. When the last flip bucket is being constructed, two completed bays will be available for spilling. However, these two bays would only have a combined capacity of 800 m^3/s with the forebay at elevation 240.0 m.

The rating of the overflow spillway flume model indicated that the design of this structure requires modification to increase discharge capacity. Likely modifications would include increasing the bay width or lowering the crest elevation and altering the rollway curvature accordingly. Model study time restrictions were such that modifications of that magnitude could not be facilitated, and the structure was used for subsequent tests without change.

Figure 5.13 shows the free surface water profile for the structure for the maximum upstream water level (elevation 244.7 m), which was measured as part of the rating procedures.

5.1.4 Pressure Measurements

Orifice Spillway

Pressure measurements were recorded at applicable tap locations during the discharge rating of the spillway in both the diverson and completed configurations. With the structure in its diversion form (i.e., no flip bucket), positive pressures were recorded at all of the piezometric taps located below the water surface profile, with the exception of the two floor taps (Taps 40 and 41) located just downstream of the flip bucket key. These two taps also had the greatest fluctuations in pressures. The largest magnitude of a negative fluctuation below an average pressure was 0.29 metres and the lowest instantaneous pressure recorded was -2.63 metres. Positive and negative pressures increased in magnitude with increases in the upstream water level.

Pressure measurements were recorded for two parallel sets of five upstream water levels between elevations 225.99 m and 230.5 m for the diversion structure. The data for one of these sets was chosen for inclusion in Table 5.2, which gives pressure heads in prototype metres.

Figure 5.14 shows the pressure distributions measured by the floor and pier taps with the upstream water elevation maintained at 230.5 m.

While rating the completed configuration of the orifice spillway, pressure tap readings were recorded for five different water levels for each gate opening of 2.0 metres, 3.0 metres, 5.0 metres and fully open. For openings of 0.5 metres and 1.0 metre, only three water levels were used. Tables 5.3 through 5.8 contain the pressure readings recorded. The pressure distributions for the fully open gate position and a 1.0 metre gate opening with the upstream water elevation near its maximum level are shown in Figures 5.15 and 5.16, respectively.

The results show that positive pressures were measured at all of the tap locations when the main gate was positioned at an opening of 5.0 metres or more. At a gate opening of 3.0 metres, small negative pressures were recorded at the three taps in the floor and pier (Floor Taps 23 and 24, and Pier Tap 22) located at the onset of the drainage slope in the floor. These pressures became most negative at a gate opening of 2.0 metres. The pressure tap in the pier (Tap 13), just downstream of the main gate check also indicated a negative pressure at the 2.0 metre gate opening. This negative pressure increased in magnitude with the gate opened 1.0 metre.

Although no pressure readings approached the level of -9 metres required to indicate cavitation potential [14,p86], attempts were made to eliminate, or reduce the magnitude of the negative pressures at the onset of the floor's drainage slope. This was done by decreasing the slope in the floor as part of the changes to the flip bucket design during the bucket tests. Unfortunately, the taps were no longer usable after being covered by new floor sections. A tap (Tap 22) in the wall, however, indicated that the negative pressures would only be reduced in magnitude, not eliminated.

Overflow Spillway

While rating the diversion port of the overflow spillway with its invert level set at the preliminary elevation of 219.0 m, piezometric pressure heads were measured at the diversion tap locations for nine different upstream water levels. The prototype pressures recorded for five of these water levels are included in Table 5.9. All pressures measured were above atmospheric except for those at three tap locations (Taps D, U, and V). The negative pressures at Taps U and V were measured when the water level within the structure was near the tap elevation and were therefore considered to be unreliable. The negative pressure recorded at Tap D for the upstream water elevation of 230.0 m was considered suspect for the same reason. Due to the absence of significant negative pressures recorded when the spillway invert level was set at elevation 219.0 m, pressures were not measured when the diversion structure was re-rated for the new invert elevation of 218.5 m.

Pressure distributions for the completed configuration of the overflow spillway were measured during the rating of the structure. For each partial gate opening, pressures were recorded for the lowest (240.0 m) and highest (244.7 m) upstream water elevations only. Tables 5.10 to 5.15 contain the prototype pressures recorded for each situation. Figures 5.17 to 5.22 graphically show the pressure distributions for each gate opening with the forebay elevation at 244.7 m.

Small negative pressures were found at a few tap locations for gate openings between 2.0 metres and 8.0 metres. These negative pressures were recorded at the two lower wall taps (Taps 11 and 12) located just downstream of the main gate check and at a few wall taps located close to the water surface level within the structure. A slight negative pressure was also recorded at a rollway tap (Tap 19) located downstream of the main gate when the gate was set at an opening of 2.0 metres. The negative pressures recorded at the wall taps downstream of the main gate check increased in magnitude with increases in the gate opening or the upstream water level. A detailed examination of the gate checks may require a larger-scaled model.

5.1.5 Flip Bucket Design

Orifice Spillway

The estimation of the discharge - throw distance relationship of the preliminary flip bucket design (Bucket 1) was made prior to the invert level and soffit design changes. Tests performed after these changes had been made revealed that actual throw distances with the new conditions were greater than the original condition estimates for small gate openings and shorter for large gate openings. The characteristics of the relationships were also different. The estimated throw distances increased asymptotically with increased discharge and the measured values increased with discharge until the gate opening reached approximately 3.0 metres and then decreased with increased discharge. Figure 5.23 presents the relationship between the upstream water level and the throw distances of Bucket 1 for several gate openings. The estimated relationship of single-bay discharge and flip bucket throw distance for an upstream water elevation of 243.2 m is shown in Figure 5.24 along with the measured curve, as derived from the data in Figure 5.23.

Two flip bucket alternatives were constructed and tested for comparison to the original design (see Figure 5.25). The first alternative (Bucket 2) differed from the original by an increased radius of curvature, higher lip elevation, and smaller lip angle. Figure 5.26 shows that Bucket 2 did not throw as far as Bucket 1 for any of the gate openings tested. Throw distance for an upstream water level of 243.2 m also peaked at a partial gate opening.

Bucket 3, the second alternative, had a larger radius of curvature, higher lip elevation and larger lip angle than the original design. The radius of curvature was smaller than that chosen for bucket 2. Bucket 3 threw the jet further than Bucket 1 for gate openings greater than 2.0 metres, and shorter for smaller openings. Figure 5.27 and 5.28 present the throw distance data for Buckets 2 and 3, respectively.

Water surface profiles along the left pier of the spillway were measured with Buckets 1 and 3 (Bucket 2 had been eliminated from consideration prior to the measurement of these profiles) installed and with the forebay level at its maximum elevation of 244.7 m. These profiles are given in Figure 5.29. It has been suggested that for a flip bucket to throw effectively, its radius must be at least four times the greatest depth of flow [10,p277]. Based on this criterion and the measured profiles, it appears that Bucket 1 may be ineffective due to an insufficient radius of curvature. For the measured depth of 7.58 metres during PMF conditions, Bucket 1 required a radius of 30.3 metres whereas its curvature was only 20.16 metres.

Both of the alternate designs appeared to be more costly than the preliminary flip bucket since each involves greater quantities of concrete for the buckets and for increased pier heights. In light of the higher costs and lower performance of Bucket 2 in comparison to the preliminary design, it was eliminated from further consideration. While a final decision regarding the energy dissipation facility to be used at the Wuskwatim site has yet to be made, Bucket 1 was chosen for use in the Comprehensive Model Study based on its cost and performance.

Overflow Spillway

The flip bucket design which was used for the overflow spillway flume tests was very similar to the preliminary flip bucket design (Bucket 1) of the orifice spillway. The orifice spillway flip bucket had a radius which was approximately 0.2 metres larger, and a lip elevation which was about 1.6 metres lower. The throw distances of the overflow spillway flip bucket were measured while the structure was being rated and are shown in Figure 5.30. This figure shows that of the gate positions tested, an opening of 6.0 metres resulted in the farthest throw distances. The throw distance for the maximum gate opening with the PMF upstream water level was approximately 35.5 metres. Comparatively, the original flip bucket design for the orifice spillway threw the water jet approximately 33.0 metres under the same conditions.

5.1.6 Ice Tests

Orifice Spillway

The initial conditions for the ice drawdown tests in the flume were a 5.0 metre gate opening and an upstream water elevation of 242.0 m (i.e., the Winter Full Supply Level). When released individually, none of the three ice floes used were drawn through the spillway. To find the maximum allowable gate opening which would prevent drawdown of any of the three sizes of ice floes, the gate was opened by intervals of 0.5 metres while the upstream water elevation was maintained at 242.0 m. The largest gate opening at which a floe was not drawn down was taken as the maximum allowable. The results of the tests showed that the 1.5 metre and 3.0 metre square ice floes were entrained in the spillway flow at a gate opening of 6.5 metres, while the 5.0 metre floe required a 7.0 metre opening.

The effects of increased vorticity on ice drawdown were investigated next. With an initial gate opening of 5.5 metres, the forebay level was lowered from the Full Supply Level to elevations which reduced the orifice submergence by 10 % and 15 % (upstream water elevations 240.72 m and 240.09 m, respectively). After each of the three ice floes was drawn down from both forebay levels, the gate was lowered by steps of 0.5 metres until the floes ceased to be entrained in the spillway flow. Table 5.16 summarizes the results of the ice drawdown tests.

To prevent the drawdown of all three sizes of ice pans with the forebay at the Winter Full Supply Level, a maximum gate opening of 6.0 metres would be required. Corresponding restrictions for orifice submergences of 10 and 15 % would be 4.5 and 4.0 metres, respectively. The maximum gate opening (i.e., 4.5 metres) for the 10 % orifice submergence reduction was chosen as the suggested limit during ice covered conditions.

Based on the rating curves for 4.0 metre and 5.0 metre gate openings, the prototype spillway could pass approximately 1080 m^3/s of water with the forebay at the Winter Full Supply Level with all three gates opened to 4.5 metres. Assuming a maximum inflow of 1000 m^3/s

[2,p14], a rise in the reservoir could be avoided during a total plant shutdown without causing ice drawdown by opening each spillway gate by approximately 4.2 metres. If for some reason a gate had to be raised to the fully open position, and it was lifted at a rate of 1 metre/minute, the risk of the gate being hit by a drawn down ice pan would only exist for approximately 3.8 minutes.

Overflow Spillway

The results of the ice drawdown tests for the overflow spillway flume model are summarized in Table 5.17. Similar to the orifice spillway ice tests, three upstream water levels were used for each of the gate openings -- the Winter Full Supply Level, as well as 10 % and 15 % reductions in submergence of the gate lip.

For the overflow spillway, the incipient drawdown of the largest ice pan occurred at a smaller gate opening than the two smaller pans, whereas the opposite was true for the orifice spillway ice tests. The mechanism of drawdown for the large ice floe through the overflow spillway was by oncoming water flowing over the top surface of the floe and forcing the upstream edge of the ice downward. Due to the lesser submergence of the discharge opening of the overflow spillway relative to the orifice, surface velocities in the forebay would have been greater, increasing the ability of the approaching water to force the ice downward.

With the forebay stabilized at the Winter Full Supply Level, the largest gate opening (to the nearest 0.5 metres) which could be used without causing the drawdown of the ice floe sizes tested, was 2.0 metres. This value was reduced to 1.5 metres for both the 10 % and 15 % reduction in gate submergence.

The discharge that could be conveyed by the overflow spillway with all three gates 1.5 metres open and the forebay at elevation 242.0 m would be approximately 300 m^3/s. The orifice spillway could pass 1080 m^3/s of water when all of the gates were set at the maximum allowable winter partial gate opening of 4.5 metres. However, a gate of the overflow spillway could be fully raised with much less risk of incurring impact damage than an orifice spillway gate. Two of the crest gates would have to be fully open for the overflow spillway to pass over 1000 m^3/s. These values will change slightly when the design of the overflow spillway is modified to increase its discharge capacity.

5.1.7 Trash Tests

Orifice Spillway

When large pieces of floating, partially saturated trash were released upstream of the orifice spillway, either individually or in clumps, there appeared to be no danger of orifice plugging occurring. Trees tended to be drawn down through the spillway one at a time. When a large mass of dry, floating trash was released upstream during PMF conditions, none of the debris was drawn through the orifice.

Submerged trash released upstream of the spillway also posed minimal threat to the operation of the spillway structure. Individual pieces of trash and groups of trees generally became longitudinally aligned with the flow lines as they moved towards the orifice entrance. In some test runs, a few pieces of trash were caught across the pier noses of the structure. One test resulted in a large tree becoming wedged between the emergency gate check and the opposite pier.

The trash scenerio which created most concern, from an operational point of view, was when a large mass of submerged trash accumulated in the trash trap adjacent to the spillway entrance, with the main gates closed. When the gate was slowly opened, a large percentage of the trash often became trapped across the pier noses of the structure

(Plates 8 and 9). In some of these cases, large trees extended as far as the emergency gate checks. One test resulted in the cluster of trash moving into the orifice against the main gate at a partial gate opening. This jam broke loose when the gate was opened further. The potential for jamming was reduced when the trash was placed in the trash trap away from the spillway entrance. Trash wedging problems were not witnessed when trees that were longer than the spillway bay width (6 metres) were not included in the trash pile.

Discussions with Manitoba Hydro field staff confirmed that the possibilty of a large mass of submerged trash accumulating in the spillway trash trap is unlikely. When the spillway is closed, most of the trash will be drawn towards the powerhouse. Trash that does accumulate in the trap will be removed as part of an annual cleaning of the forebay.

The consulting engineers for the project have analyzed the forces acting on a log jammed between a gate check and the opposite pier and determined that a log would almost certainly fail in bending if the gate was fully opened [4,p1]. This investigation reduced the concern that the closure of a gate would be prevented by a log jam. It was thus felt that trash passage considerations would not prevent the implementation of an orifice spillway configuration.

Overflow Spillway

For each of the gate openings tested, occurences of trash jamming at the noses of the piers were observed. For gate openings of 1.0 metre and 2.0 metres, none of the pieces of trash were drawn down through the spillway. Trash was drawn through the spillway with the gate open at 4.0 metres and 6.0 metres if the pieces were present in small numbers. If a large amount of trash accumulated at the crest gate, vorticity and therefore drawdown were inhibited. Trash that accumulated in front of the gate was successfully flushed through the structure by opening the gate to 8.0 metres. When the gate was set at an opening of 8.0 metres prior to releasing the trash upstream, the potential for trash jamming at the pier noses or accumulating in front of the gate was still observed. Flushing of the accumulated trash was achieved by opening the gate to a higher level. When the gate was left fully open, most of the incoming trash was easily pulled through the spillway. However, large pieces of trash still showed the potential for getting caught across the pier noses, thereby preventing smaller logs from advancing further.

5.2 Comprehensive Model Tests

5.2.1 Diversion Tests

The discharge rating data obtained with the comprehensive model for the diversion structure was compared to that from the flume model. Figure 5.31 shows the rating data as well as the flume model study rating curve, which assumed an approach channel head loss of 0.5 metres. The figure displays a good agreement between the two data sets. The maximum deviation of the comprehensive model data from the curve drawn through the flume model data was estimated to be approximately 6 % (i.e., for an upstream water level of 227.05 m. the comprehensive model discharge was 6 % less than indicated by the flume model rating curve for that level). The maximum deviation, from the rating curve, for the flume data was 3 %. An upstream water elevation of 231.33 m was required to pass the design flow of 1250 m^3/s through the diversion structure, which was greater than the design level of elevation 230.5 m.

It was observed that a hydraulic jump formed in the approach channel of the spillway during the passage of low flows. However, the jump moved out of the approach while the discharge was still below the flow values expected for the prototype, and therefore no jump should develop in the prototype channel. A symmetrical criss-cross pattern of standing waves was observed within the exit channel for all flows tested.

The final concreting of the spillway was simulated with the comprehensive model for the closure sequence of: Bay 3 - Bay 2 - Bay 1. During this procedure, the discharge was maintained at the design value of 1250 m^3/s. It was determined that with only two bays open, an upstream water elevation of approximately 238.90 m was required to pass 1250 m^3/s. The maximum allowable elevation is 240.0 m.

With a single bay closed, stable vorticies formed in front of the two operating bays. Although none of the vorticies had a solid core of air reaching the intake, several did pull air bubbles through.

The exit channel of the spillway was not able to entirely contain the water when Bay 1 was closed and Bays 2 and 3 had flip buckets. The minimal amount of water that breached the right wall of the channel was not sufficient to affect the main dam.

5.2.2 Discharge Rating

The three discharge ratings obtained for the spillway while each of the bays was individually fully open compared well with each other. No discrepency was found for the condition of all three bays simultaneously open. When the spillway was rated with just one gate open, all four of the model powerhouse units were operating. The units were closed when the three spillway bays were open. Bay 3 was rated for gate openings of 2.0, 4.0 and 6.0 metres and the discharge values obtained also compared favorably with the flume model data. The comprehensive model rating data, in three-bay equivalent discharges, is shown for comparison with the flume model rating curves in Figure 5.32.

When only one spillway gate was fully open, vortices would intermittently form in front of the open bay. The vorticies occasionally caused small air bubbles to be pulled through the spillway. A clockwise rotation of surface flow was visible in front of the South Transition Structure while the spillway was operating (see Plate 10). With one bay fully open, the water in the exit channel would wash up over the flip bucket lips of the two closed bays. When all three bays were fully open, voriticies which were capable of drawing down small air bubbles occasionally formed beside the left pier walls in front of Bays 1 and 2 and beside both piers of Bay 3. A counter-clockwise rotation with a slight surface dimple also occurred in front of the trash sluice gate for this condition.

For partial gate openings of Bay 3, surface swirls with slight dimples periodically formed in the area immediately upstream of the curtain wall, but no air was drawn down. The rotational flow in front of the South Transition Structure was present but only had a maximum instantaneous velocity of 0.66 metres/second adjacent to the structure. Water levels in the exit channel remained below the flip bucket lip elevation (220.1 m) for partial openings (up to 6 metres) of the Bay 3 main gate.

5.2.3 Operating Procedures

The two sequences of gate operation which were investigated with the comprehensive model are presented in graphical form in Figure 5.33 which uses the flume model rating curves to estimate discharges. In the first sequence, Sequence A, the bays were opened one at a time starting with Bay 3 and ending with Bay 1. As described in the Discharge Rating section of this thesis, when Bay 3 was partially open (up to a 6.0 metre opening) non-air entraining vorticies intermittently formed in front of its entrance, a clockwise rotational flow pattern existed adjacent to the South Transition Structure, and the water levels inside the exit channel did not exceed the lip elevation of the flip buckets. With Bay 3 fully open, the intermittent vorticies would occasionally draw a few small air bubbles through the spillway and water washed up over the flip bucket lip of Bays 1 and 2.

For conditions where Bay 3 was open and Bay 2 was operated at partial and full gate openings, intermittent vorticies which were capable of entraining small air bubbles into the flume were present in front of Bay 3, and non-air entraining vorticies formed adjacent to the left pier of Bay 2. Non-air entraining vorticies formed beside the left pier of Bay 1 when Bays 2 and 3 were both completely open. The water level in the exit channel exceeded the flip bucket lip elevation of Bay 1 for all gate openings of Bay 2 when Bay 3 was fully open. The maximum water level measured in Bay 1 was 1 metre higher than the lip elevation, and occurred when Bays 1 and 2 were both fully open.

Vorticies capable of entraining small air bubbles continued to form in front of Bays 2 and 3 as the main gate of Bay 1 was raised. The vorticies forming in front of Bay 1 were not strong enough to draw air bubbles until the gate reached the 6.0 metre opening. When all three bays were fully open, the rotational flow adjacent to the South Transition Structure was strongest with a maximum instantaneous velocity along its upstream face of 1.42 metres/second. Velocities of this magnitude were not considered to be threatening to the main dam, which will be protected with rip rap.

The second operating sequence investigated, Sequence B, did not result in noticable surface irregularities in front of the spillway for the initial settings. Intermittent surface swirls with slight dimples formed at the entrances of all three bays when they were operating with their gates set at 4.0 metre openings. With each gate set at a 6.0 metre opening, the vorticies became strong enough to draw small air bubbles into the spillway flow. The clockwise rotational flow in front of the South Transition Structure was present for all of the conditions tested and was again strongest when all three bays were fully open.

The noted approach flow conditions for each of the gate operating sequences tested were considered acceptable. Conditions in the exit channel, however, caused some concern, since water levels exceeded the flip bucket lip elevation, most likely reducing the throwing ability of the spillway.

5.2.4 Ice Tests

The initial ice tests performed with the comprehensive model used ice floes which were somewhat thinner (0.44 metres thick) than those used in the flume model tests (0.63 metres thick). These tests determined the smallest gate opening, to the nearest 0.5 metres, at which any of the three ice floes was drawn down for both open and ice covered conditions. The spillway gates were either all open the same amount or just Bay 3 was operated. The forebay level (elevation 240.72 m) reflected a 10 % reduction in orifice submergence from the

Winter Full Supply Level. Results of these tests are presented in Table 5.18. As can be seen in the table, the most restrictive condition was with all three bays operating and with the presence of a surrounding ice cover. The 1.5 metre floe was drawn down with all three gates set at a 4.0 metre opening. Repeating the worst condition with a 0.63 metre thick ice floe, the 1.5 metre floe was not drawn down until all three gates had 4.5 metre openings. Therefore, the maximum suggested gate opening in the winter, based on the comprehensive model study, would be 4.0 metres. The results of the flume model study indicated a maximum gate opening of 4.5 metres.

When several pieces of ice were placed within the simulated ice cover opening, the potential for drawdown was reduced since the conglomeration of floes inhibited vorticy formation by moving the free water surface farther from the spillway entrance. For openings up to 5.0 metres, no ice pans were drawndown, but instead gathered at the right side of the ice cover surface opening. With open water conditions, the floes tended to be drawn into the rotational flow in front of the South Transition Structure.

5.2.5 Trash Tests

While the powerhouse was operating and the spillway and trash sluice were closed, all of the floating trash was drawn directly towards the powerhouse whether it was initially scattered across the forebay or placed at the west end of the main dam. If one or two spillway bays were opened, some trash was momentarily drawn to the entrance of the operating bays before moving on to the powerhouse. While in front of the spillway or powerhouse, the trash remained floating and was not drawn down. With the trash sluice open and the spillway closed, all of the trash released along the main dam moved directly towards the operating powerhouse.

The results of these tests indicate that the scenerio in which large amounts of trash gather in front of the non-operating spillway and sink to its trash trap seems unlikely. Therefore, the flume model tests in which saturated logs were placed in front of the spillway bay prior to opening the main gate were simulating an event which will likely not occur in the operation of the prototype. If trash temporarily gathers in front of the spillway while it is operating, as it did in the model, it can be expected that any logs that sink will be drawn through the spillway as opposed to gathering in the trap.

CHAPTER 6

Conclusions

Physical hydraulic modelling was shown to be an important element in the design process of a hydro development site. Mathematical modelling is another integral tool which aids in the creation and evaluation of preliminary design concepts and can be integrated with physical modelling to investigate situations such as the fracture of logs due to drag forces, when viscous forces are not properly scaled on a Froudian model.

The accuracy of the modelling techniques used were, in some cases, limited by the assumptions made. Assumed values included the approach channel head losses and the percentage of orifice submergence required to model vorticity. The effectiveness of the method of breaking the surface tensions acting on ice floes was assumed to be adequate. Also, the accuracy of the trash jamming tests was limited by the inability to precisely model the strength of the logs and the drag forces that they were subjected to.

This research provided results that indicated a need for changes to the preliminary designs of both spillway alternatives for the Wuskwatim Generating Station. The proposed invert levels of the two spillways were found to be insufficient to pass design flows and lower elevations were thus suggested. Two alternate soffit designs were tested for the orifice spillway and were both found to perform better than the preliminary design. The original design of the orifice spillway flip bucket was found to be more desirable than the two alternate configurations tested. Changes to the spillway exit channel will be required for the flip buckets to operate at their full potential. An alteration to the shape of the base slab of the orifice spillway was also proposed in order to eliminate the negative pressures measured along its surface. The original design of the overflow spillway was found to be incapable of conveying the design discharge of the project and changes to the width and/or crest elevation will be required.

From an operational stand-point, no complications were found that would eliminate the orifice spillway concept for the northern site considered. Unlike the overflow spillway, the orifice would be capable of passing anticipated winter flows in the event of a plant shutdown without causing ice drawdown. The results of the ice drawdown tests were found to be slightly different for the flume and comprehensive model studies due to the effects of the three-dimensional flow patterns for the comprehensive model.

The results of the orifice spillway model tests and the mathematical analysis of the strength of logs within the structure indicated that the presence of trash in the impounded forebay will not create operational problems that would jeopardize the safety of the project. If passing trash downstream of the generating station is found to be environmentally unacceptable, a log boom would be required for the overflow spillway to prevent trash passage. The volume of floating trash drawn through the orifice spillway would be minimal in comparison.

The inlet conditions for the two gate operating sequences investigated were found to be acceptable, as were the inlet conditions for the tested sequence of bay closure for final concreteing of the spillway.

Topics yet to be addressed for the Wuskwatim site, but not anticipated to have a bearing on the feasibility of the orifice spillway concept, include an investigation of the stability of the main gate for the orifice spillway and modifications to the spillway exit channel. Completion of the design modifications to the overflow spillway, using the flume and comprehensive models, is also scheduled to take place prior to a final decision regarding the adoption of either of the spillway alternatives.

Although no attempt was made to quantify the reliability of the operation of an orifice spillway, it was concluded that based on the issues of concern addressed in this thesis, no operational difficulties were found that would eliminate the orifice spillway concept from future consideration for the case studied. However, other sites in northern Manitoba may have additional concerns not applicable to the Wuskwatim site, such as orifice plugging due to the release of an upstream ice jam or ice buffeting of the downstream face of the main gate. Site specific investigations may thus be required.

The research contained herein indicated that orifice spillways may be used effectively in northern applications and therefore warrant consideration as alternatives to overflow spillways during the conceptual stages of the development of a hydro site.

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Appendix A

Tables

Table 4.1

Saturated Trash Sizes and Relative Quantities
for the Orifice Spillway Flume Model
Study in Prototype Metres

Length (metres)	Diameter (metres)	Percentage of Total Trash Volume
10	0.28 to 0.30	10 %
5 to 10	0.15 to 0.23	20 %
3 to 5	0.15 to 0.23	20 %
3 to 5	0.08 to 0.15	20 %
less than 3	less than 0.08	30 %

Table 5.1

Comparison of Piezometric Pressure Tap Readings in Prototype Metres for Three Orifice Spillway Soffit Designs

Tap No.	Upstream Water Elevation					
	242.0 metres			244.7 metres		
	Soffit			Soffit		
	1	2	3	1	2	3
1	-1.73	-1.40	0.25	-1.66	-1.33	0.58
2	-0.58	-0.40	0.11	-0.50	-0.25	0.29
3	-1.51	0.36	0.72	-1.58	0.54	0.94
4	0.86	2.34	3.42	1.37	2.92	4.14
5	1.62	2.84	4.14	2.16	3.49	5.04
6	1.12	2.30	2.92	1.44	2.70	3.42
7	1.69	2.88	3.46	2.12	3.42	4.03
8	0.58	2.02	2.48	0.68	2.27	2.77
9	1.22	2.56	0.07	1.44	2.95	0.40

Table 5.2

Orifice Spillway Piezometric Pressure Tap Readings in Prototype Metres for the Diversion Configuration

Tap No.	UPSTREAM WATER ELEVATION (m)				
	225.99	226.99	228.00	229.50	230.50
5	------	------	------	------	0.07
6	------	------	------	------	0.40
7	------	------	------	------	0.61
8	------	------	------	------	0.47
9	------	------	------	------	0.72
10	------	------	------	------	0.04
11	------	------	------	------	0.22
12	------	0.86	1.58	2.66	3.53
13	2.12	3.56	4.28	5.18	6.01
14	3.46	4.93	5.69	6.84	7.63
15	2.30	3.78	4.54	5.69	6.48
16	------	------	------	------	0.25
17	------	1.04	1.73	2.70	3.49
18	2.59	3.92	4.61	5.62	6.37
19	3.38	4.68	5.33	6.41	7.09
20	2.41	3.53	4.10	4.90	5.51
21	3.13	4.32	4.86	5.69	6.34
22	2.27	3.35	3.82	4.64	5.15
23	2.84	3.89	4.39	5.15	5.69
24	2.84	3.92	4.43	5.15	5.72
25	2.38	3.78	4.43	5.26	5.87
26	3.28	4.68	5.33	6.16	6.77
27	2.52	3.82	4.39	5.11	5.62
30	2.20	3.53	4.10	4.82	5.33
31	2.88	4.21	4.79	5.51	6.01
32	2.81	4.10	4.72	5.40	5.90
35	3.13	4.86	5.65	6.59	7.20
36	3.85	5.62	6.44	7.34	8.06
39	1.08	1.80	2.16	2.59	2.84
40	-0.83	-0.51	-1.87	-2.27	-2.48
41	-0.32	-0.58	-1.58	-0.68	-1.04

----- water level below tap elevation

Table 5.3

Orifice Spillway Piezometric Pressure Tap Readings in Prototype Metres for the Fully Open Gate Condition

Tap No.	UPSTREAM WATER ELEVATION (m)				
	241.97	242.43	243.37	244.06	244.69
1	0.25	0.29	0.43	0.50	0.58
2	0.11	0.14	0.18	0.22	0.29
3	0.72	0.79	0.83	0.86	0.94
4	3.42	3.53	3.82	3.96	4.14
5	4.14	4.28	4.61	4.79	5.04
6	2.92	2.99	3.17	3.31	3.42
7	3.46	3.56	3.78	3.89	4.03
8	2.48	2.52	2.63	2.70	2.77
9	2.95	3.02	3.13	3.24	3.35
10	0.47	0.47	0.47	0.47	0.47
11	0.83	0.86	0.90	0.86	0.86
12	5.44	5.51	5.69	5.72	5.83
13	8.86	8.93	9.04	9.11	9.22
14	10.66	10.73	10.91	11.05	11.16
15	9.65	9.72	9.90	10.01	10.15
16	1.15	1.15	1.15	1.19	1.19
17	5.08	5.11	5.22	5.22	5.29
18	8.75	8.78	8.93	9.00	9.11
19	9.25	9.29	9.36	9.43	9.50
20	6.37	6.37	6.37	6.37	6.37
21	7.06	7.06	7.06	7.06	7.06
22	5.51	5.51	5.47	5.47	5.44
23	5.87	5.83	5.76	5.72	5.69
24	5.87	5.83	5.76	5.72	5.69
25	9.04	9.11	9.22	9.29	9.40
26	9.86	9.94	10.04	10.15	10.26
28	9.18	9.29	9.54	9.65	9.83
29	10.94	11.05	11.30	11.45	11.63
33	9.00	9.14	9.36	9.54	9.72
34	11.27	11.48	11.74	11.95	12.17
37	1.69	1.69	1.73	1.73	1.76
38	1.91	1.87	1.91	1.94	1.98

Table 5.4

Orifice Spillway Piezometric Pressure Tap Readings in Prototype Metres for a 5.0 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m)				
	242.03	242.73	243.38	244.11	244.70
1	10.69	11.30	12.20	12.46	12.89
2	12.49	13.10	13.64	14.29	14.72
3	14.26	14.83	15.44	16.16	16.63
4	12.53	13.10	13.64	14.29	14.76
5	12.89	13.54	14.04	14.69	15.16
6	13.82	14.40	14.98	15.59	16.16
7	14.11	14.69	15.23	15.84	16.42
8	15.12	15.73	16.27	17.03	17.50
9	15.16	15.80	16.38	17.06	17.53
10	------	------	------	------	------
11	------	------	------	------	------
12	0.65	0.58	0.54	0.58	0.58
13	7.52	7.70	7.81	8.03	8.14
14	11.09	11.38	11.52	11.81	11.99
15	8.42	8.64	8.78	9.04	9.14
16	------	------	------	------	------
17	0.22	0.22	0.25	0.25	0.25
18	5.98	6.16	6.23	6.34	6.41
19	6.66	6.73	6.80	6.91	6.98
20	2.12	2.12	2.12	2.12	2.09
21	2.88	2.84	2.84	2.81	2.81
22	0.97	1.01	0.97	0.90	0.86
23	1.19	0.97	0.97	0.97	0.97
24	1.26	1.22	1.15	1.08	1.01
25	3.82	3.89	3.92	3.96	3.96
26	4.75	4.75	4.82	4.86	4.90
28	3.38	3.46	3.49	3.56	3.64
29	5.51	5.62	5.69	5.76	9.43
33	4.46	4.46	4.57	4.68	4.72
34	7.13	7.31	7.45	7.60	7.70
37	0.43	0.47	0.47	0.47	0.47
38	0.76	0.90	0.94	0.94	0.94

----- water level below tap elevation

Table 5.5

Orifice Spillway Piezometric Pressure Tap
Readings in Prototype Metres for
a 3.0 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m)				
	242.04	242.70	243.46	244.18	244.69
1	12.96	13.21	14.22	14.94	15.44
2	14.29	14.98	15.59	16.34	16.70
3	15.26	15.91	16.63	17.35	17.89
4	14.29	14.94	15.59	16.31	16.81
5	14.58	15.23	15.88	16.60	17.10
6	15.41	16.06	16.70	17.42	17.93
7	15.66	16.31	16.96	17.68	18.18
8	16.20	16.78	17.53	18.25	------
9	16.45	17.03	17.75	18.50	19.01
10	------	------	------	------	------
11	------	------	------	------	------
12	------	------	------	------	------
13	3.28	3.35	3.35	3.42	3.46
14	8.32	8.46	8.68	8.89	9.07
15	4.64	4.72	4.86	4.97	5.04
16	------	------	------	------	------
17	------	------	------	------	------
18	2.56	2.56	2.59	2.63	2.66
19	3.13	3.13	3.17	3.24	3.24
20	0.79	0.76	0.76	0.76	0.72
21	1.48	1.48	1.48	1.44	1.44
22	-0.14	-0.14	-0.22	-0.29	-0.32
23	-0.14	-0.14	-0.22	-0.25	-0.32
24	0.11	0.07	-0.04	-0.07	-0.14
25	1.80	1.80	1.80	1.80	1.80
26	2.74	2.77	2.77	2.81	2.84
28	0.25	0.25	0.25	0.25	0.25
29	2.48	2.48	2.52	2.59	2.59
33	0.68	0.68	0.65	0.65	0.65
34	4.03	4.07	4.14	4.25	4.32
37	0.00	0.00	0.00	0.00	0.00
38	0.43	0.43	0.43	0.47	0.47

----- water level below tap elevation

Table 5.6

Orifice Spillway Piezometric Pressure Tap
Readings in Prototype Metres for
a 2.0 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m)				
	242.03	242.88	243.54	243.99	244.64
1	13.75	14.22	14.94	15.44	16.06
2	14.90	15.52	16.09	16.67	17.24
3	15.66	16.34	16.96	17.46	18.11
4	14.98	15.55	16.16	16.70	17.28
5	15.23	15.80	16.42	16.96	17.53
6	15.95	16.56	17.17	17.71	18.36
7	16.24	16.58	17.42	17.96	18.58
8	16.60	17.24	17.86	18.36	------
9	16.88	17.50	18.11	18.61	19.26
10	------	------	------	------	------
11	------	------	------	------	------
12	------	------	------	------	------
13	-0.43	-0.47	-0.50	-0.50	-0.50
14	5.51	5.69	5.76	5.83	5.98
15	1.76	1.80	1.80	1.84	1.87
16	------	------	------	------	------
17	------	------	------	------	------
18	1.19	1.19	1.19	1.19	1.22
19	1.51	1.51	1.51	1.55	1.55
20	0.47	0.47	0.47	0.47	0.47
21	1.08	1.08	1.08	1.08	1.08
22	-0.32	-0.36	-0.40	-0.40	-0.43
23	-0.32	-0.54	-0.54	-0.58	-0.58
24	-0.18	-0.25	-0.29	-0.29	-0.36
25	0.94	0.86	0.83	0.83	0.83
26	1.80	1.80	1.80	1.84	1.84
28	------	------	------	------	------
29	1.01	1.01	1.01	1.01	1.01
33	------	------	------	------	------
34	2.09	2.09	2.12	2.16	2.16
37	------	------	------	------	------
38	0.22	0.22	0.22	0.22	0.22

----- water level below tap elevation

Table 5.7

Orifice Spillway Piezometric Pressure Tap Readings in Prototype Metres for a 1.0 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m)		
	242.02	243.27	244.64
1	14.00	15.30	------
2	15.05	16.31	------
3	15.77	17.03	------
4	15.12	16.38	18.00
5	15.37	16.63	18.25
6	16.09	17.35	18.94
7	16.34	17.60	19.19
8	16.67	17.89	------
9	16.96	18.18	------
10	------	------	------
11	------	------	------
12	------	------	------
13	-1.80	-1.87	-1.87
14	1.69	1.73	1.80
15	0.50	0.54	0.65
16	------	------	------
17	------	------	------
18	0.47	0.50	0.58
19	0.61	0.65	0.65
20	0.18	0.14	0.11
21	0.72	0.72	0.72
22	-0.14	-0.14	-0.14
23	-0.22	-0.32	-0.40
24	-0.18	-0.25	-0.32
25	0.07	0.07	0.07
26	0.86	0.90	0.90
28	------	------	------
29	0.04	-0.04	-0.04
33	------	------	------
34	0.47	0.50	0.54
37	------	------	------
38	0.00	0.00	0.00

----- water level below tap elevation

Table 5.8

Orifice Spillway Piezometric Pressure Tap
Readings in Prototype Metres for
a 0.5 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m)		
	241.88	243.39	244.57
1	14.04	15.41	------
2	15.05	16.42	------
3	15.73	17.10	------
4	15.16	16.49	17.68
5	15.37	16.74	17.93
6	16.09	17.42	18.61
7	16.34	17.68	18.86
8	16.63	17.96	------
9	16.88	18.22	------
10	------	------	------
11	------	------	------
12	------	------	------
13	------	------	------
14	0.50	0.50	0.50
15	------	------	------
16	------	------	------
17	------	------	------
18	------	------	------
19	0.25	0.29	0.29
20	------	------	------
21	0.43	0.43	0.43
22	------	------	------
23	0.00	-0.04	-0.04
24	-0.11	-0.18	-0.22
25	------	------	------
26	0.72	0.68	0.65
28	------	------	------
29	------	------	------
33	------	------	------
34	------	------	------
37	------	------	------
38	------	------	------

----- water level below tap elevation

Table 5.9

Overflow Spillway Piezometric Pressure Tap Readings in Prototype Metres for the Diversion Port Configuration

Tap No.	UPSTREAM WATER ELEVATION (m)				
	225.99	230.00	234.05	238.10	240.00
A	------	1.84	2.99	3.78	3.78
B	0.11	4.10	5.72	7.02	7.52
C	2.23	6.37	6.98	8.17	8.39
D	------	-0.11	0.11	0.14	-0.04
E	------	3.56	4.32	5.15	5.62
F	1.98	5.94	7.20	8.39	8.78
G	------	------	0.04	0.07	0.07
H	------	2.45	2.52	2.66	2.74
I	1.87	4.79	5.04	5.29	5.40
J	1.48	3.60	3.31	2.81	2.74
K	2.09	4.21	3.35	2.66	2.23
L	1.80	4.03	3.60	3.06	2.99
M	2.59	4.64	3.89	3.13	2.84
N	2.63	4.64	4.00	3.35	3.20
O	------	0.76	0.76	0.86	0.90
P	------	3.10	3.49	3.74	3.96
Q	0.25	3.06	3.56	3.82	3.92
R	------	2.05	2.70	2.56	2.66
S	0.47	3.02	3.42	3.56	3.67
T	------	1.08	1.44	1.58	1.66
U	-0.11	2.12	2.41	2.52	2.59
V	------	------	-0.11	0.00	0.07
W	------	0.25	0.50	0.61	0.68

----- water level below tap elevation

Table 5.10

Overflow Spillway Piezometric Pressure Tap Readings in Prototype Metres for the Fully Open Gate Condition

Tap No.	UPSTREAM WATER ELEVATION (m)		
	242.00	243.16	244.64
1	4.54	5.11	5.83
2	6.21	6.41	6.98
3	5.15	5.22	5.51
4	1.76	2.38	3.28
5	3.64	4.14	4.90
6	2.20	2.09	1.91
7	1.80	1.40	1.26
8	2.48	2.23	2.23
9	0.58	1.19	1.94
10	2.27	2.63	3.17
11	0.94	0.76	0.22
12	0.76	0.61	0.14
13	1.62	1.40	1.33
14	1.40	1.22	1.12
15	1.19	0.90	0.72
16	1.26	1.01	0.94
17	0.97	0.94	1.01
18	0.79	0.61	0.61
19	1.33	1.22	1.19
20	0.50	0.79	1.26
21	1.69	2.20	2.92
22	2.12	2.59	3.38
23	4.82	6.05	7.67
24	8.32	9.58	11.20
25	4.61	5.69	6.73
26	10.28	12.17	14.08
27	11.70	13.21	15.08
28	7.70	9.47	11.77
29	8.50	10.33	12.64
30	6.84	8.42	10.51
31	8.75	10.33	12.38
32	6.84	8.14	9.76
33	8.93	10.26	11.92
34	2.38	2.92	3.64
35	4.18	4.82	5.62
36	6.34	7.09	7.96

Table 5.11

Overflow Spillway Piezometric Pressure Tap Readings in Prototype Metres for an 8.0 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m) 244.71
1	7.09
2	10.04
3	8.93
4	5.00
5	6.44
6	4.79
7	5.00
8	4.93
9	------
10	3.31
11	-2.92
12	-1.33
13	3.10
14	2.81
15	2.41
16	2.27
17	0.79
18	0.86
19	1.01
20	0.07
21	1.12
22	1.55
23	4.82
24	8.71
25	5.15
26	10.87
27	12.38
28	7.96
29	8.89
30	7.09
31	9.18
32	7.24
33	9.50
34	2.56
35	4.54
36	6.66

----- water level below tap elevation

Table 5.12

Overflow Spillway Piezometric Pressure Tap Readings in Prototype Metres for a 6.0 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m)	
	239.86	244.66
1	4.18	7.99
2	6.88	11.12
3	6.59	10.73
4	1.98	*****
5	3.64	7.13
6	3.24	6.73
7	4.21	6.73
8	4.21	6.01
9	------	------
10	1.58	2.48
11	1.08	-1.80
12	0.97	-0.65
13	2.95	3.49
14	2.66	3.06
15	2.45	2.56
16	2.34	2.30
17	0.97	0.14
18	1.01	0.43
19	1.69	0.54
20	-0.11	-0.07
21	0.79	0.22
22	1.22	0.61
23	1.91	2.92
24	5.22	6.88
25	3.06	4.00
26	6.16	8.28
27	7.60	9.97
28	3.82	5.08
29	4.61	6.23
30	3.10	4.54
31	5.11	6.73
32	3.53	5.22
33	5.76	7.63
34	1.12	1.76
35	2.74	3.64
36	4.50	5.62

----- water level below tap elevation
***** piezometer water level above scale

Table 5.13

Overflow Spillway Piezometric Pressure Tap Readings in Prototype Metres for a 4.0 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m)	
	240.02	244.76
1	5.22	*****
2	8.24	*****
3	8.42	*****
4	2.70	*****
5	4.82	*****
6	5.44	7.16
7	6.23	9.18
8	5.47	7.52
9	------	------
10	------	------
11	-0.50	-1.40
12	0.32	-0.40
13	3.24	3.96
14	2.88	3.42
15	2.48	2.66
16	2.23	2.27
17	0.22	-0.47
18	0.54	-0.07
19	0.94	0.04
20	------	------
21	0.22	-0.22
22	0.61	0.11
23	------	------
24	3.46	4.72
25	2.74	3.13
26	3.53	5.15
27	5.36	7.09
28	1.98	2.88
29	2.81	3.85
30	1.04	1.87
31	3.10	4.21
32	1.33	2.56
33	3.67	5.18
34	0.36	0.79
35	1.87	2.59
36	3.53	4.39

----- water level below tap elevation
***** piezometer water level above scale

Table 5.14

Overflow Spillway Piezometric Pressure Tap Readings in Prototype Metres for a 2.0 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m)	
	240.19	244.84
1	5.80	*****
2	9.29	*****
3	9.65	*****
4	3.06	*****
5	5.76	*****
6	6.88	10.87
7	8.21	*****
8	6.98	9.97
9	------	------
10	------	------
11	0.25	-0.11
12	-0.40	-0.86
13	2.99	3.89
14	2.38	2.99
15	1.69	1.84
16	1.30	1.30
17	------	------
18	------	------
19	0.40	-0.18
20	------	------
21	-0.07	-0.18
22	0.40	0.18
23	------	------
24	1.58	1.55
25	1.84	2.20
26	0.76	1.15
27	3.28	4.00
28	0.76	1.01
29	1.40	1.73
30	------	------
31	1.08	1.48
32	------	------
33	1.26	1.87
34	------	------
35	0.76	1.08
36	2.38	2.92

----- water level below tap elevation
***** piezometer water level above scale

Table 5.15

Overflow Spillway Piezometric Pressure Tap
Readings in Prototype Metres for
a 1.0 Metre Gate Opening

Tap No.	UPSTREAM WATER ELEVATION (m) 239.94	244.78
1	5.72	*****
2	9.25	*****
3	9.76	*****
4	2.81	*****
5	5.69	*****
6	7.99	*****
7	9.14	*****
8	8.14	*****
9	------	------
10	------	------
11	------	------
12	------	------
13	1.98	:::::::
14	1.26	1.69
15	0.47	0.14
16	0.29	0.04
17	------	------
18	------	------
19	0.47	0.18
20	------	------
21	------	------
22	0.29	0.18
23	------	------
24	------	------
25	1.15	1.15
26	------	------
27	1.91	3.02
28	------	------
29	0.83	1.19
30	------	------
31	------	------
32	------	------
33	------	------
34	------	------
35	------	------
36	1.51	1.73

----- water level below tap elevation
***** piezometer water level above scale
::::::: connecting hose became seperated from piezometer

Table 5.16

Summary of Flume Model Ice Drawdown
Tests for the Orifice Spillway

Gate Opening	Upstream Water Elevation								
	242.0 m*			240.72 m**			240.09 m***		
	Floe Size (m)			Floe Size (m)			Floe Size (m)		
	1.5	3.0	5.0	1.5	3.0	5.0	1.5	3.0	5.0
4.0 m							N	N	N
4.5 m				N	N	N	D	N	N
5.0 m	N	N	N	D	D	D	D	D	D
5.5 m	-	-	-	D	D	D	D	D	D
6.0 m	N	N	N						
6.5 m	D	D	N						
7.0 m	D	D	D						

N - Floe not drawn down
D - Floe drawn down

* Winter Full Supply Level
** 10 % reduction in orifice submergence
*** 15 % reduction in orifice submergence

Floe Sizes: 1.5 x 1.5 x 0.63 metres
3.0 x 3.0 x 0.63 metres
5.0 x 5.0 x 0.63 metres

Table 5.17

Summary of Flume Model Ice Drawdown
Tests for the Overflow Spillway

	Upstream Water Elevation								
	242.0 m*			**			***		
	Floe Size (m)			Floe Size (m)			Floe Size (m)		
Gate Opening	1.5	3.0	5.0	1.5	3.0	5.0	1.5	3.0	5.0
1.0 m	N	N	N	N	N	N	N	N	N
1.5 m						N			N
2.0 m	N	N	N	N	N	D	N	N	D
2.5 m	N	N	D	N	N	D	N	N	D
3.0 m	D	N	D	D	D	D	D	D	D
3.5 m		D							

N - Floe not drawn down
D - Floe drawn down

* Winter Full Supply Level
** 10 % reduction in orifice submergence
*** 15 % reduction in orifice submergence

Floe Sizes: 1.5 x 1.5 x 0.63 metres
3.0 x 3.0 x 0.63 metres
5.0 x 5.0 x 0.63 metres

Table 5.18

Summary of Ice Drawdown Tests for the
Orifice Spillway Comprehensive Model

Gate Opening	Bays 1, 2 & 3 Open		Bay 3 Open	
	Open Water	Ice Cover	Open Water	Ice Cover
	Floe	Floe	Floe	Floe
	A B C	A B C	A B C	A B C
3.5 m	N N N	N N N		
4.0 m	N N N	D N N	N N N	N N N
4.5 m	D N N	D N N		
5.0 m	D N N	D D D	N N N	N N N
5.5 m			D N N	D N N

N - Floe not drawn down
D - Floe drawn down

Floe Sizes: A - 1.5 x 1.5 x 0.44 metres
B - 3.0 x 3.0 x 0.44 metres
C - 5.0 x 5.0 x 0.44 metres

Appendix B

Figures

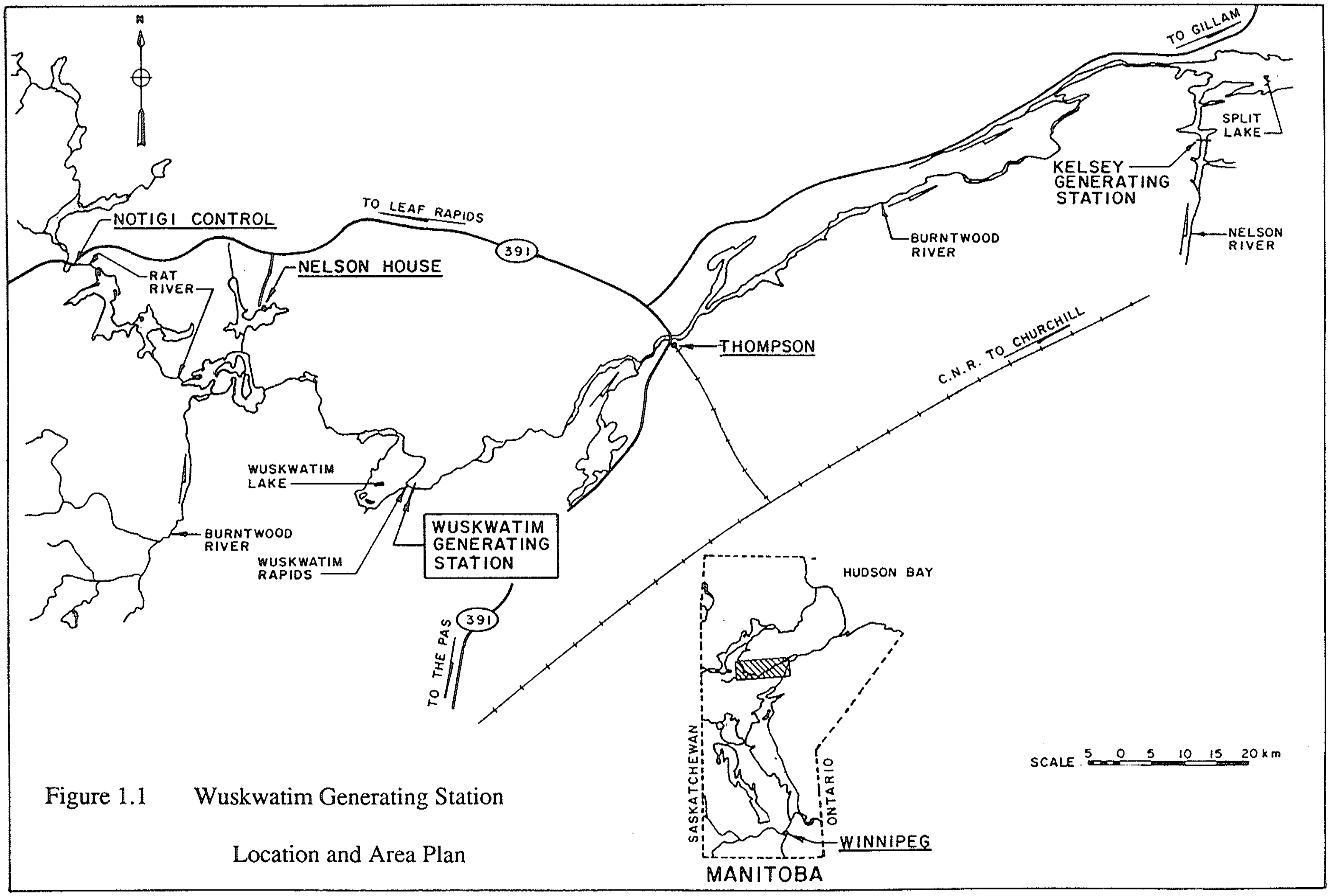

Figure 1.1 Wuskwatim Generating Station

Location and Area Plan

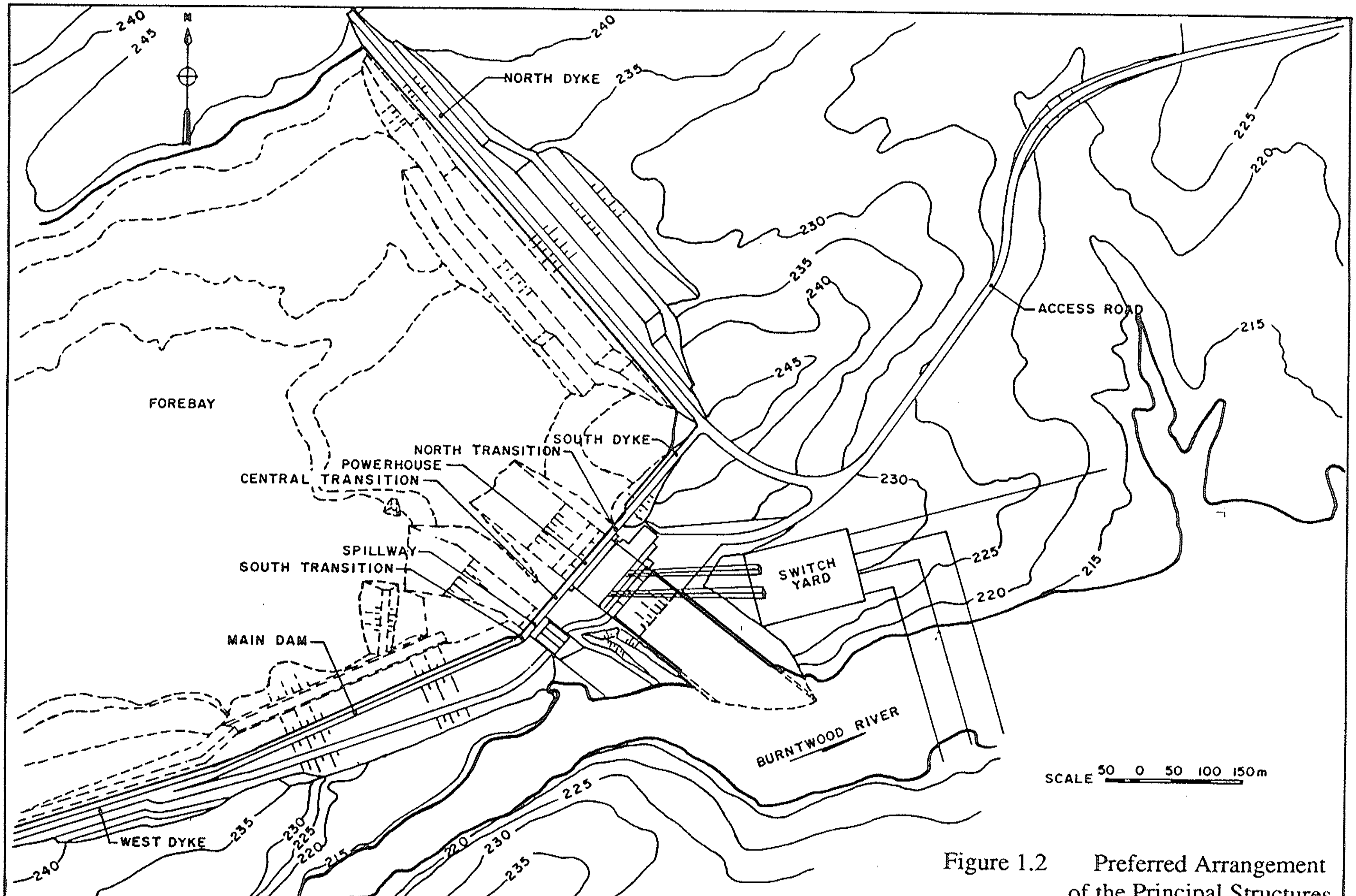

Figure 1.2 Preferred Arrangement of the Principal Structures

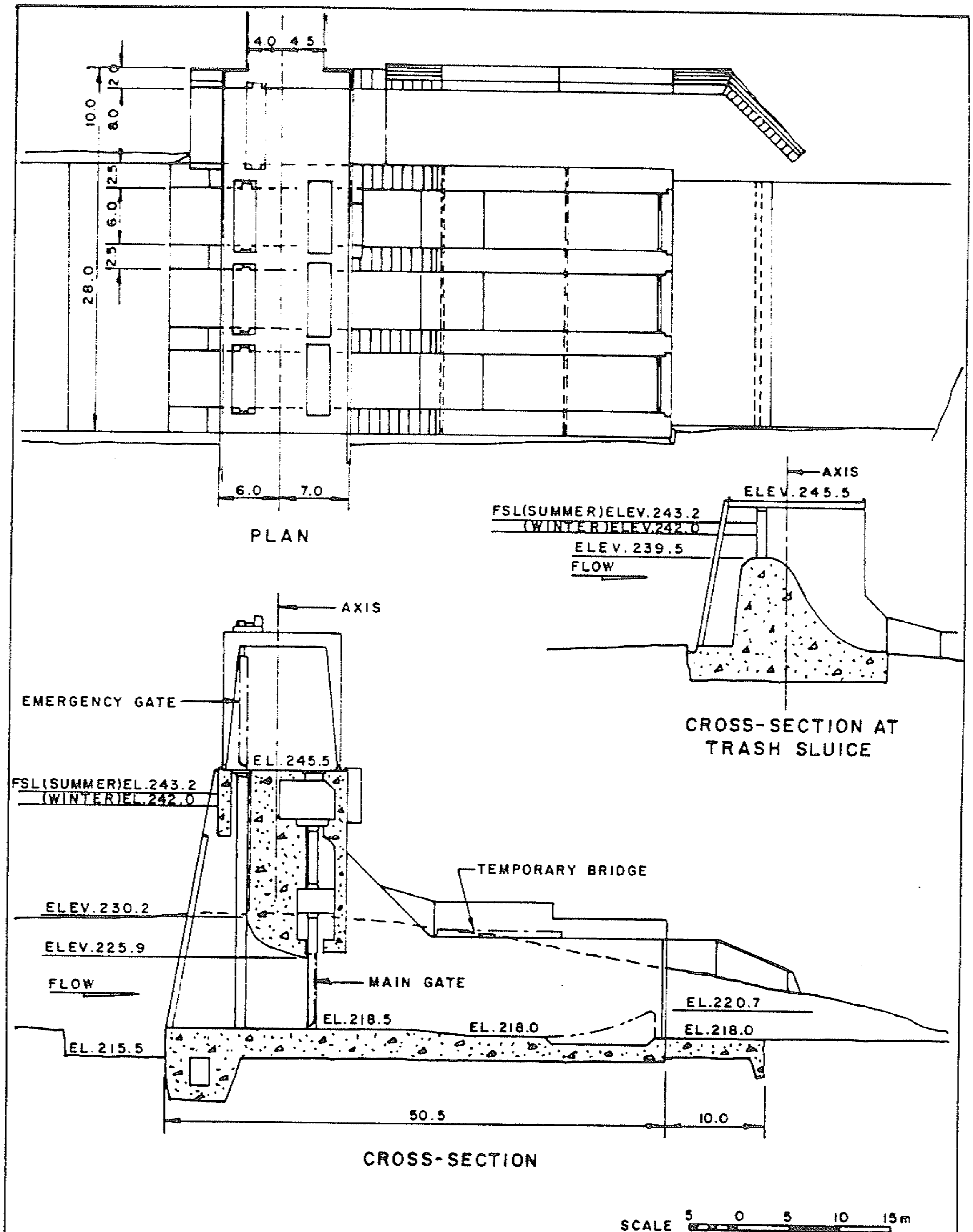

Figure 3.1 Preliminary Design of the Orifice Spillway

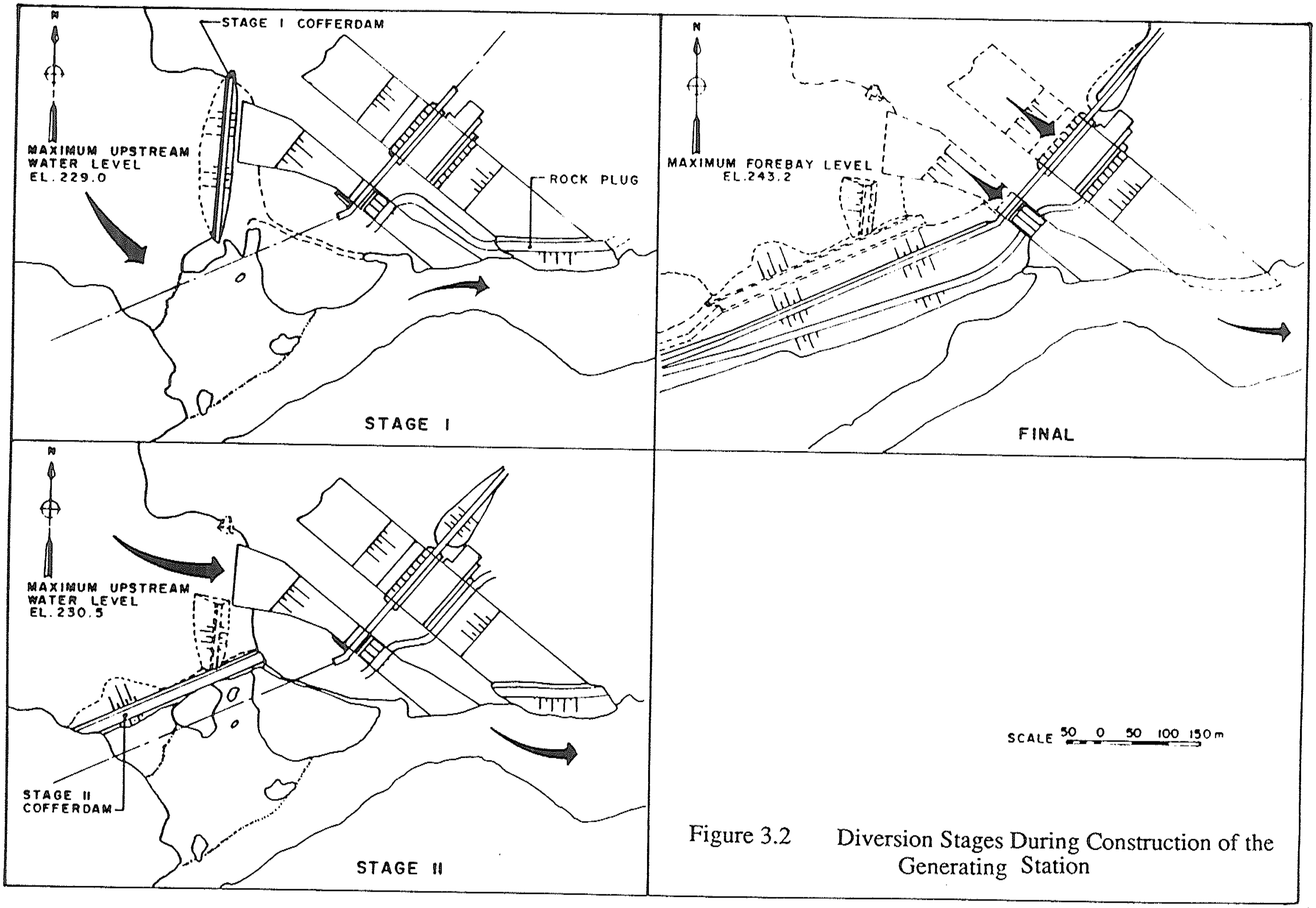

Figure 3.2 Diversion Stages During Construction of the Generating Station

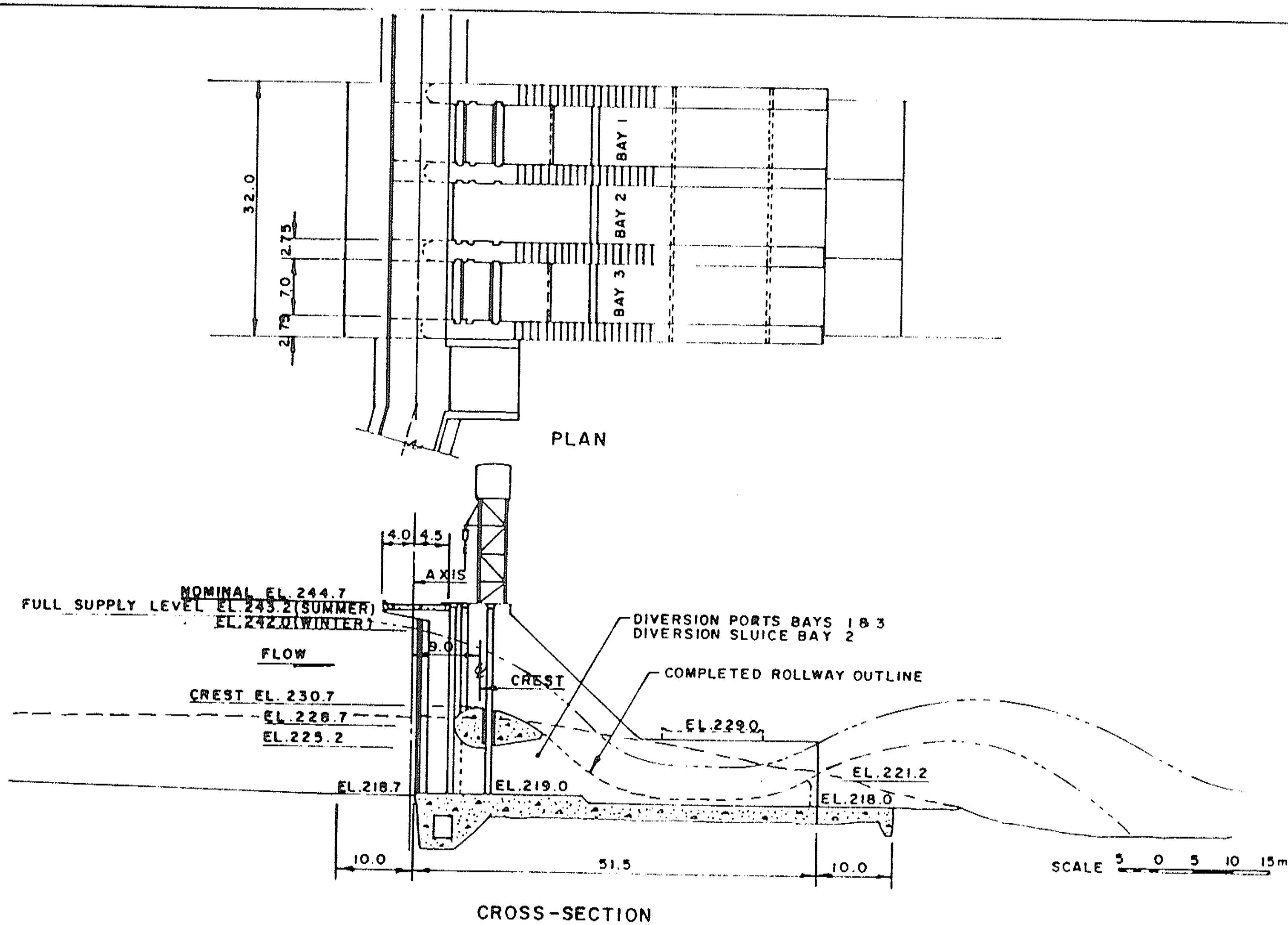

Figure 3.3 Preliminary Design of the Overflow Spillway

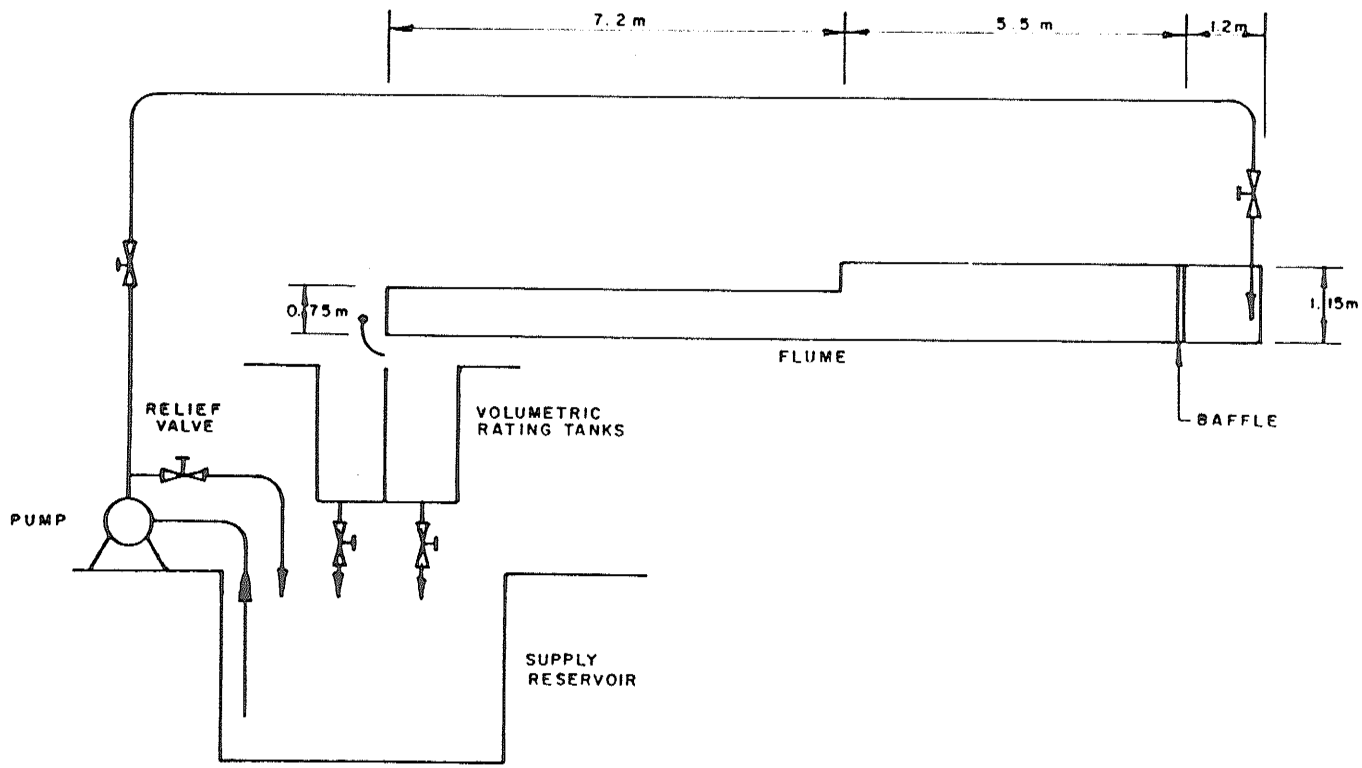

Figure 4.1 Labratory Facilities for the Flume Model Study

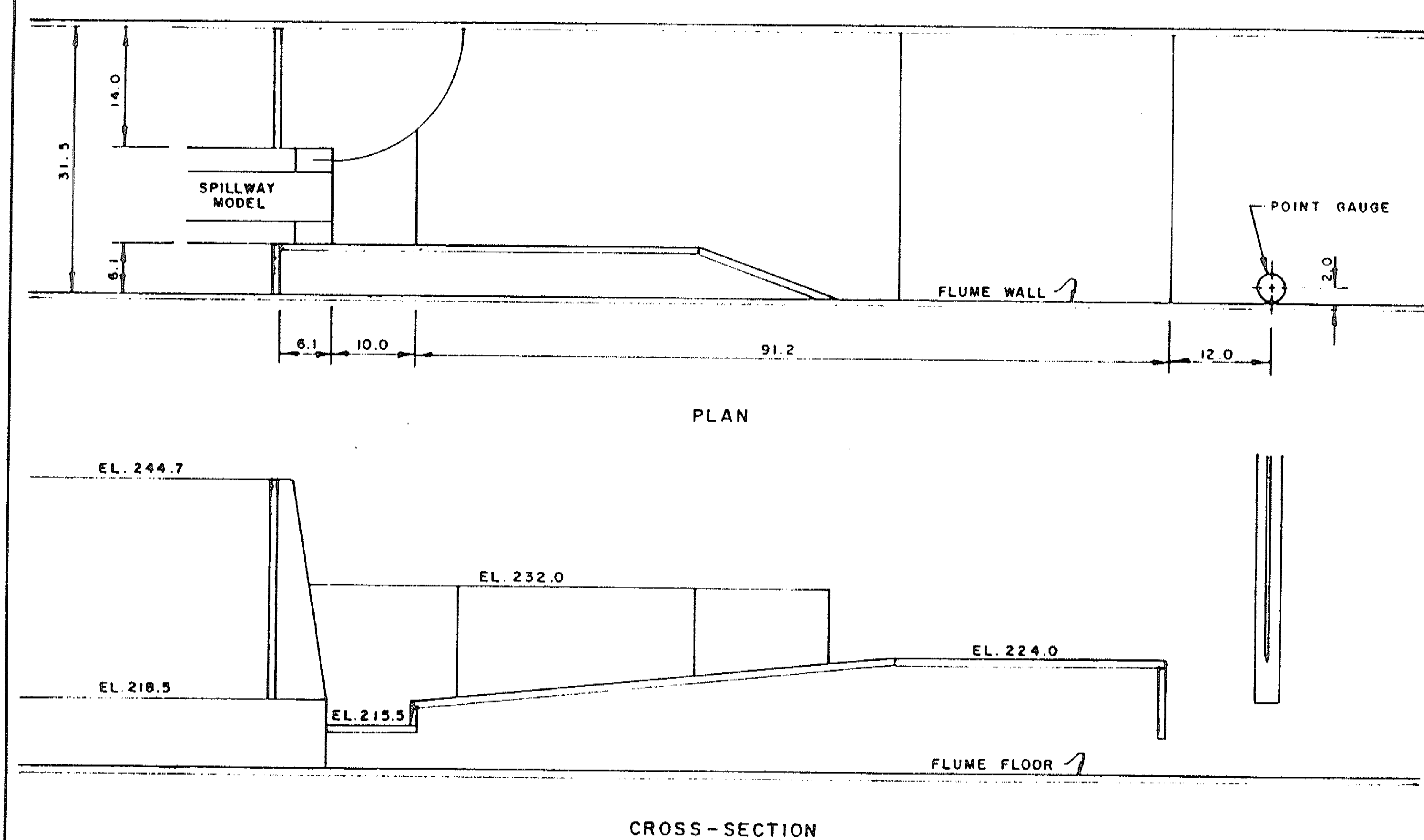

Figure 4.2 Flume Model Approach Conditions in Prototype Metres

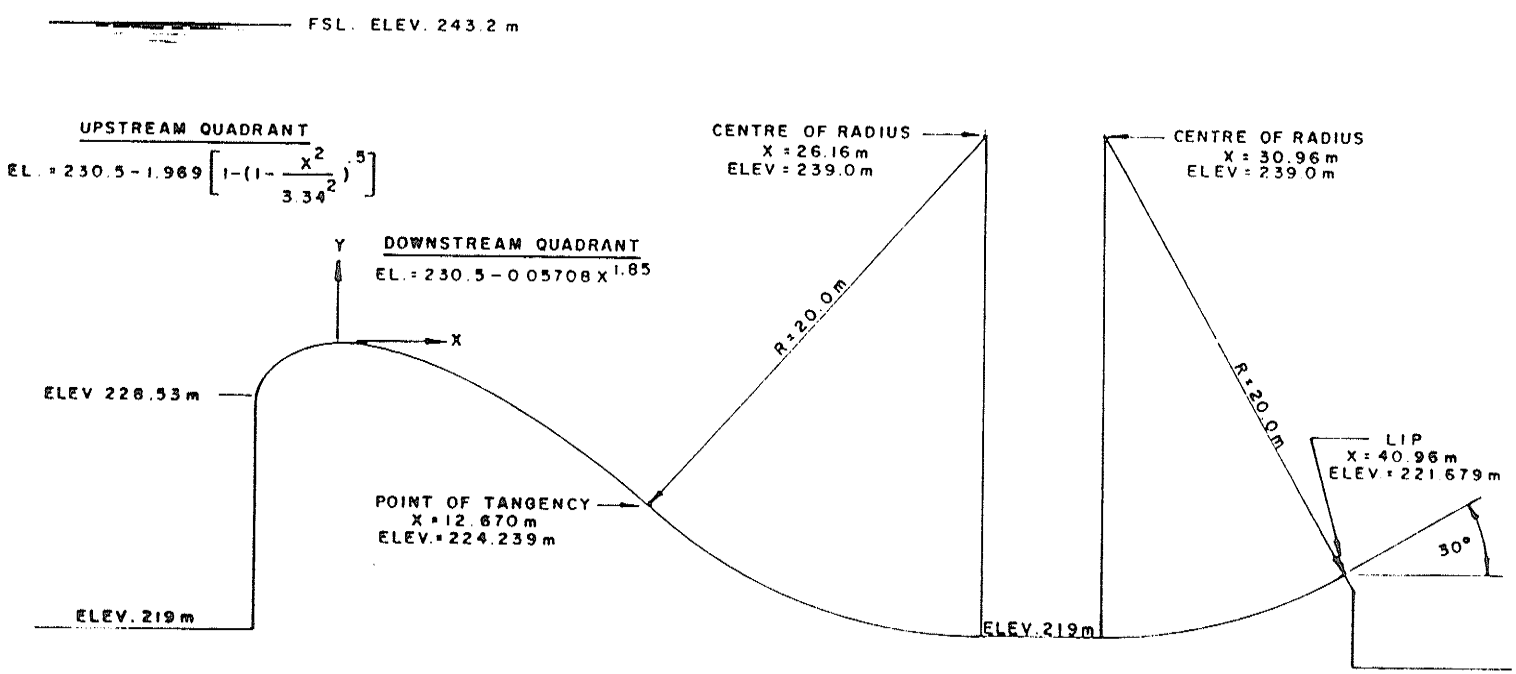

Figure 4.3 Rollway Design for the Overflow Spillway

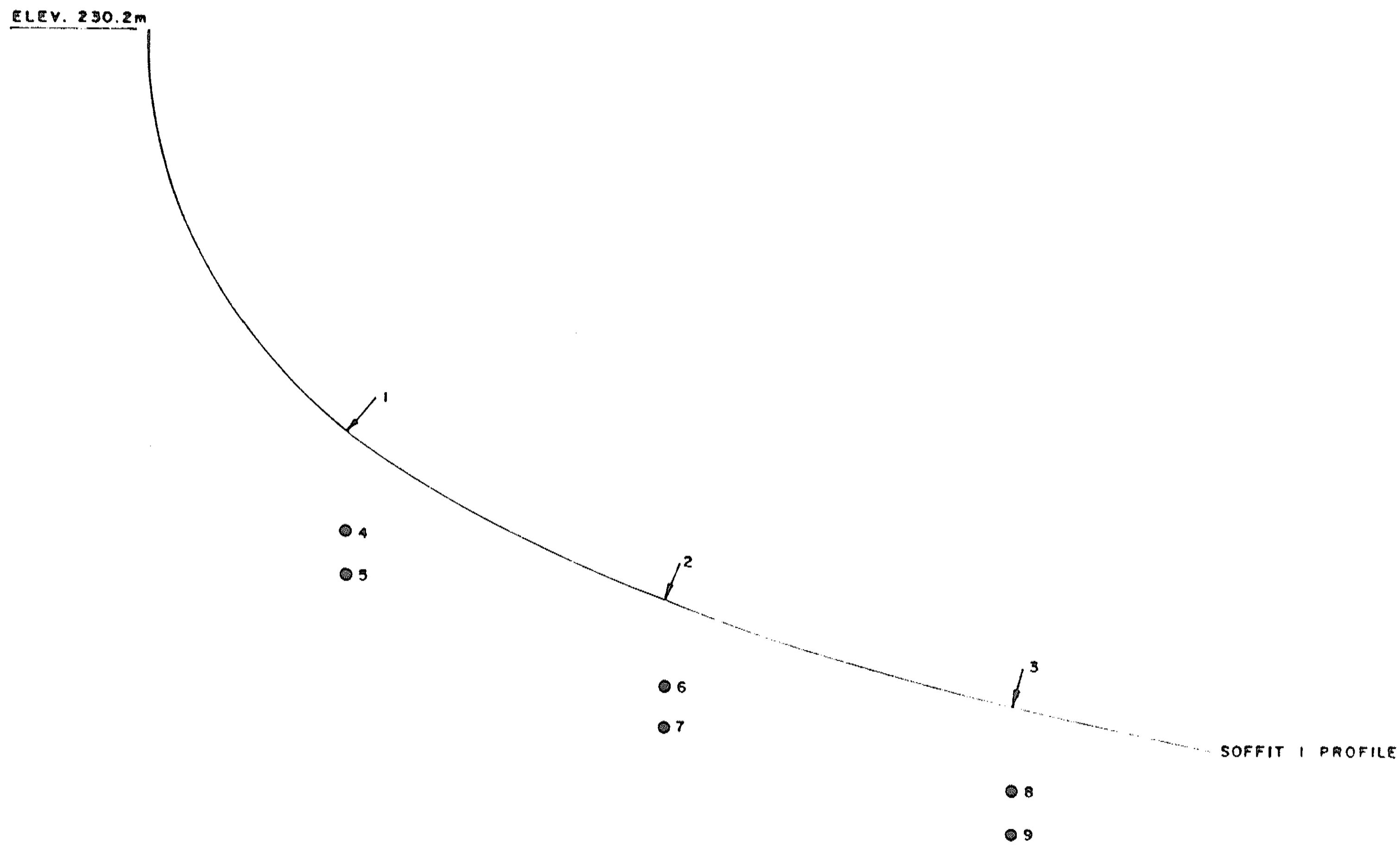

Figure 4.4 Piezometric Pressure Taps used for the Orifice Spillway Soffit Design Evaluations

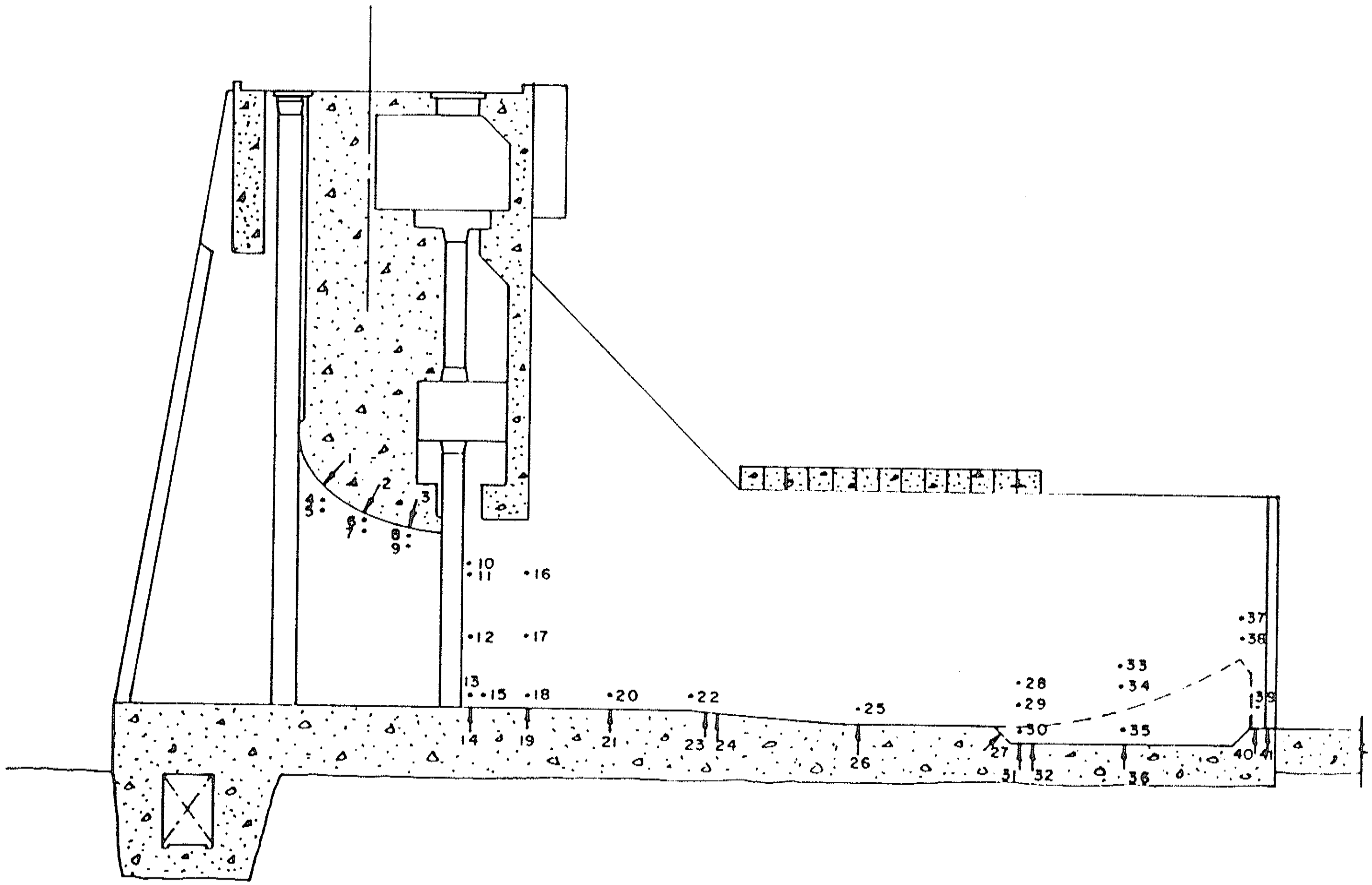

Figure 4.5 Piezometric Pressure Taps for the Diversion and Completed Configurations of the Orifice Spillway

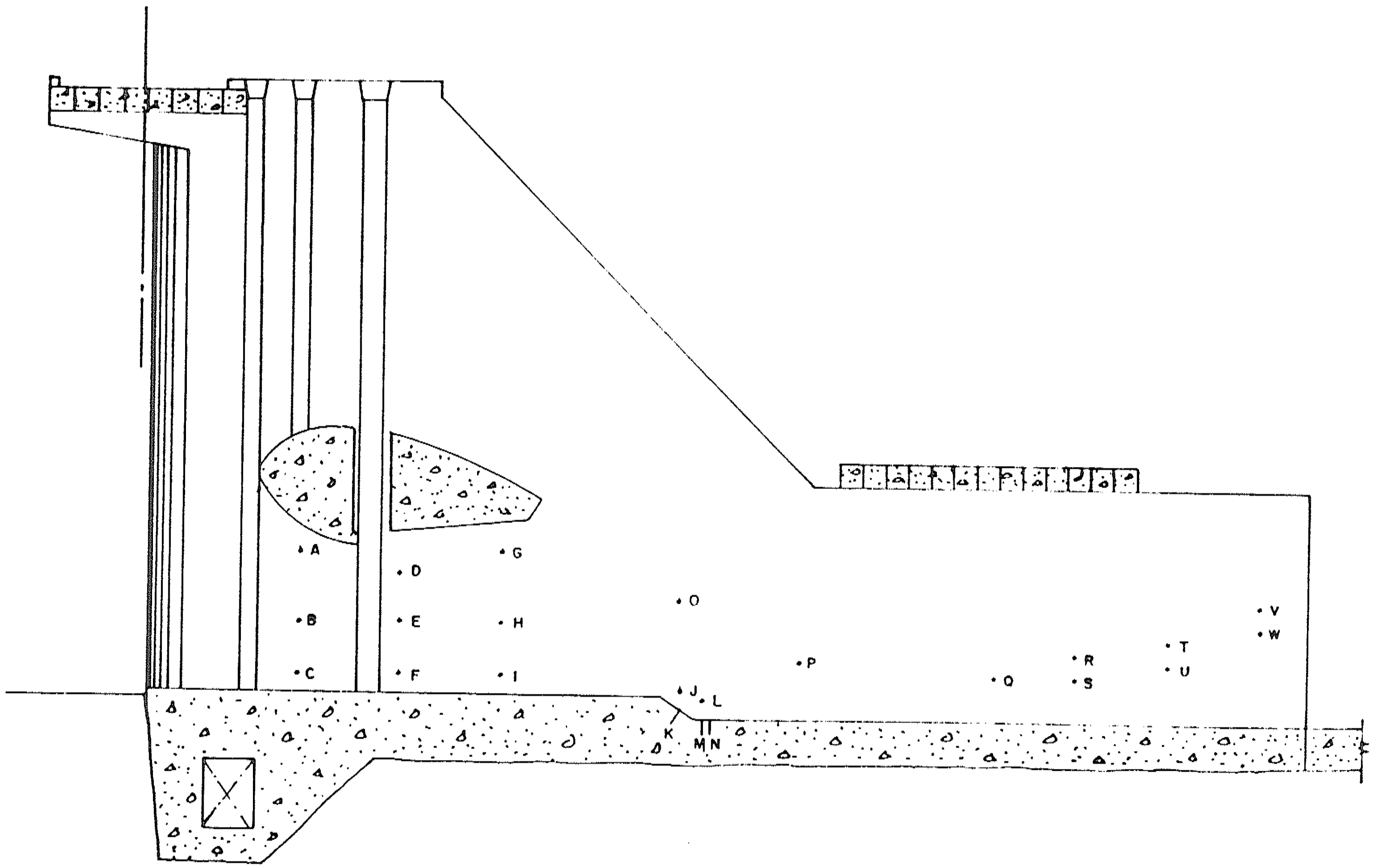

Figure 4.6 Piezometric Pressure Taps for the Diversion Port Configuration of the Overflow Spillway

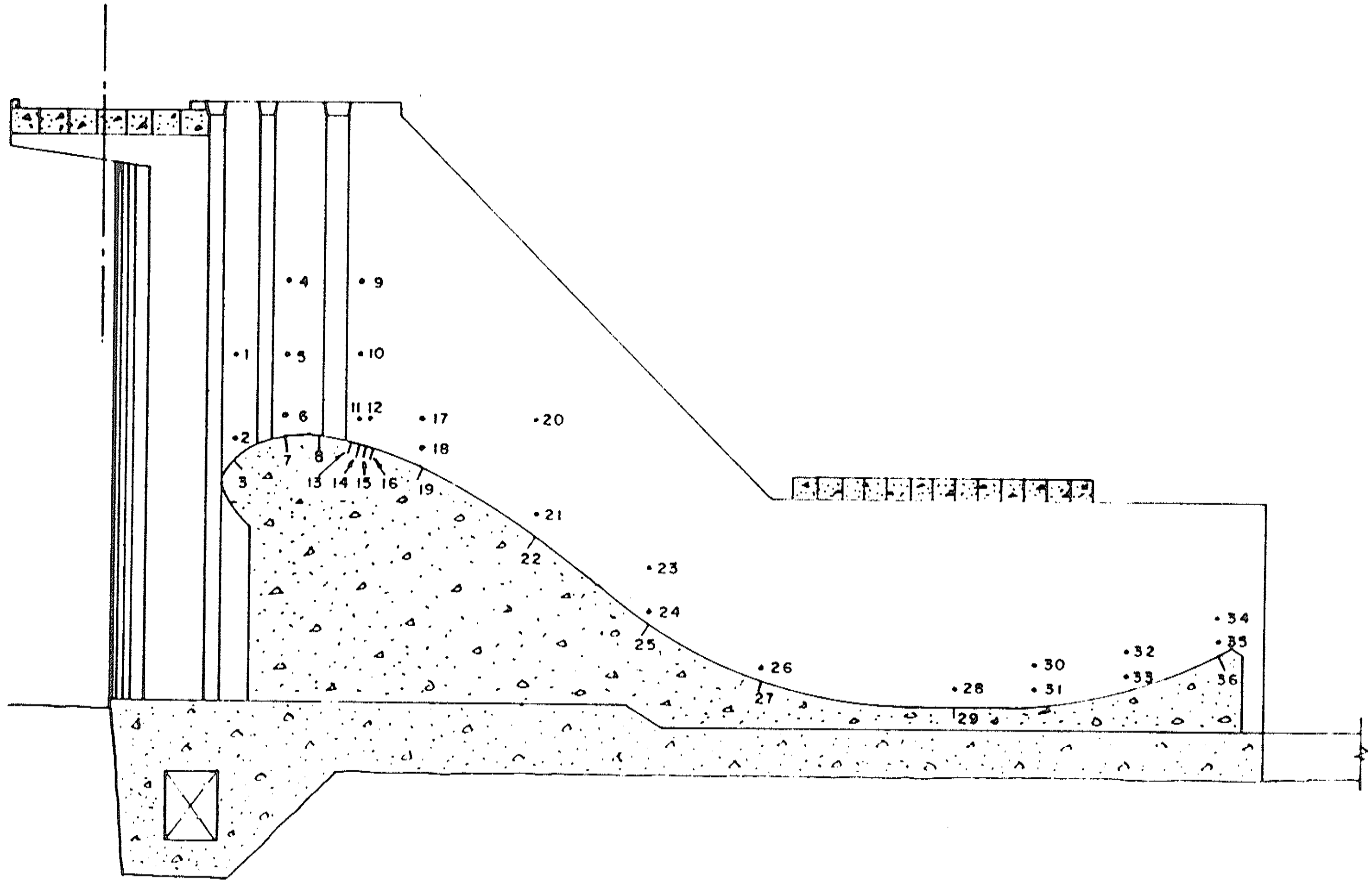

Figure 4.7 Piezometric Pressure Taps for the Completed Configuration of the Overflow Spillway

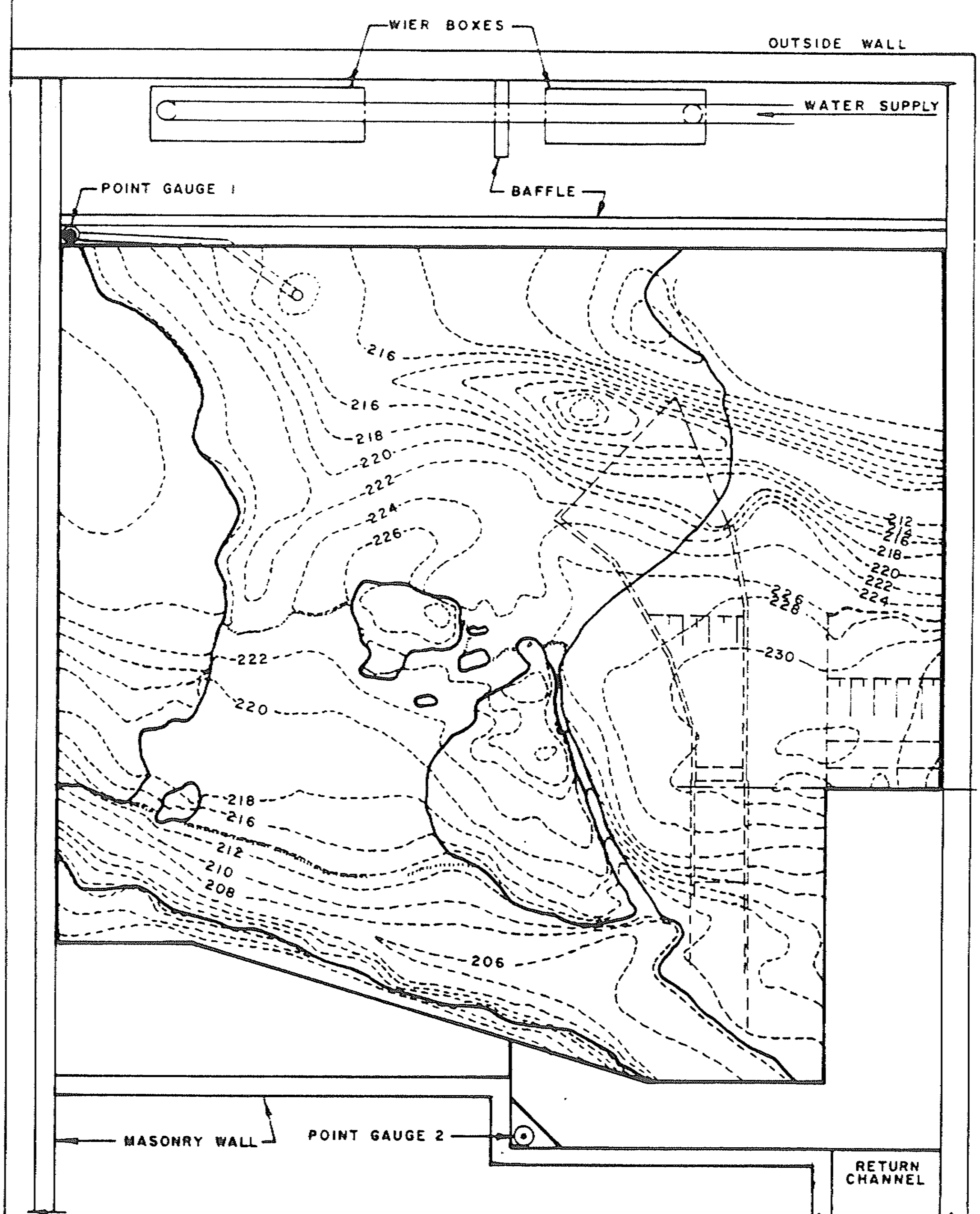

Figure 4.8 Layout of the Comprehensive Model Without the Principal Structures

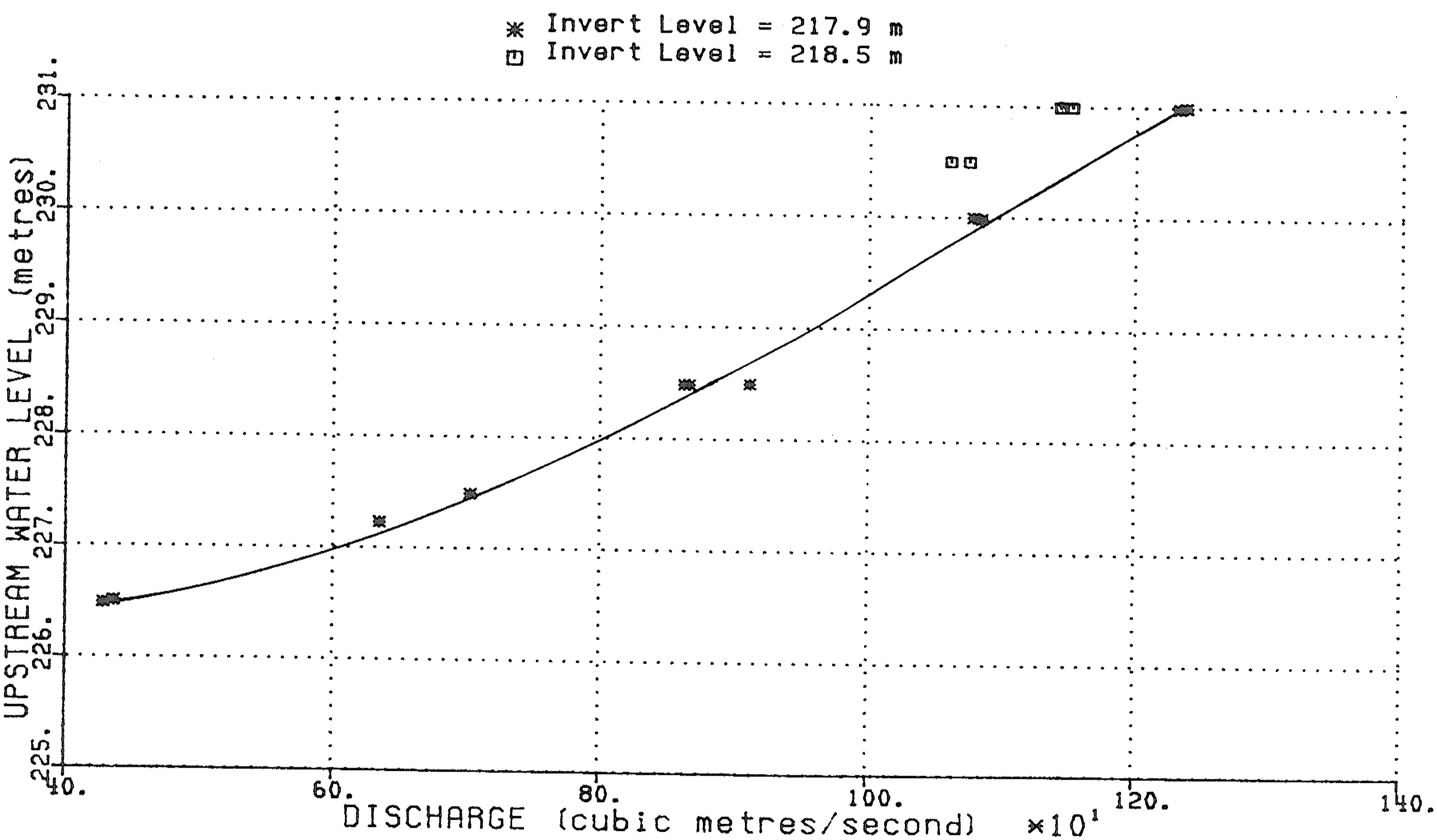

Figure 5.1 Discharge Rating Curve for the Diversion Configuration of the Orifice Spillway With an Invert Level of 217.9 m and Rating Data With an Invert Level of 218.5 m

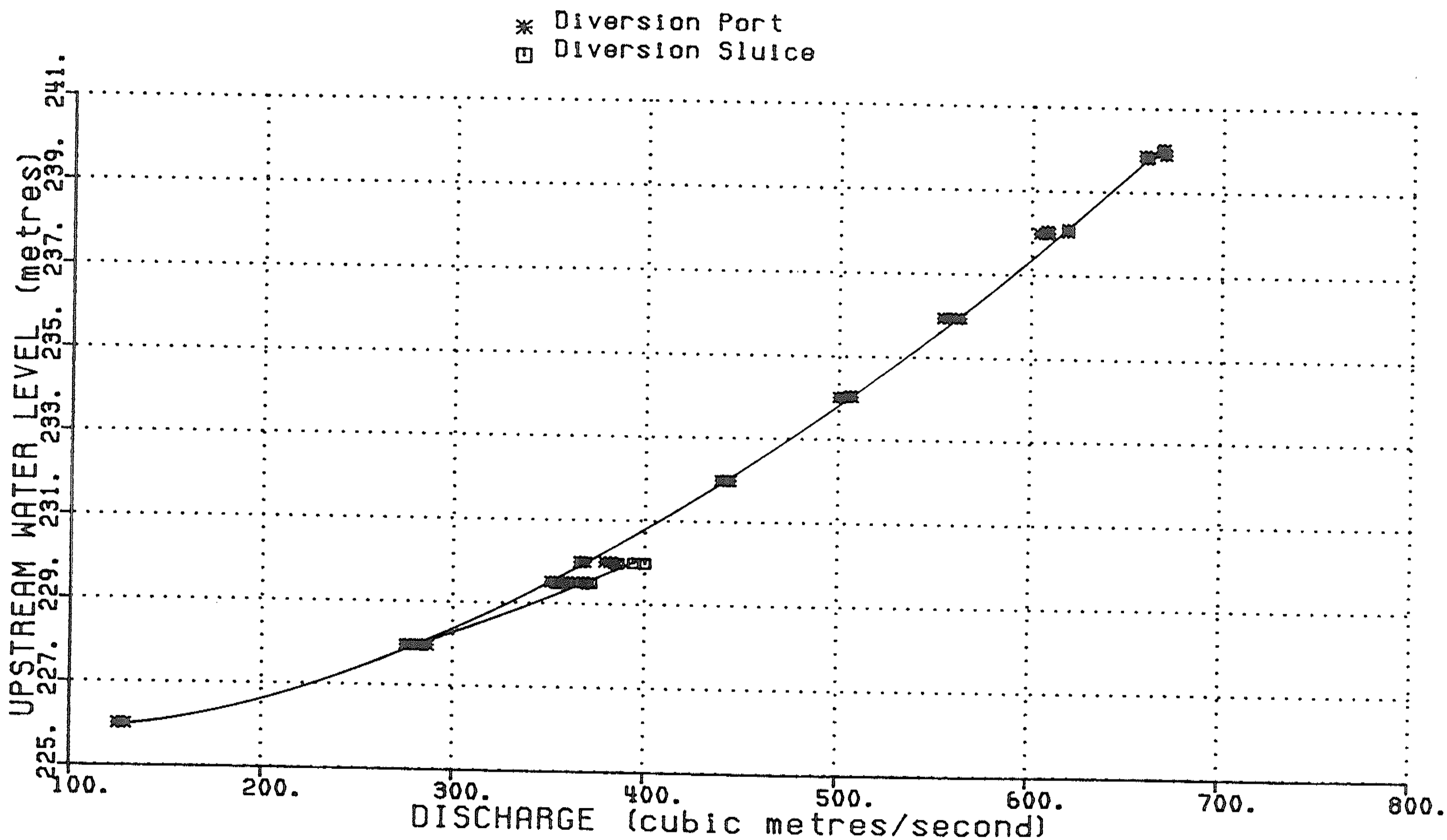

Figure 5.2 Discharge Rating Curves for the Diversion Ports and Sluice of the Overflow Spillway With an Invert Level of 219.0 m

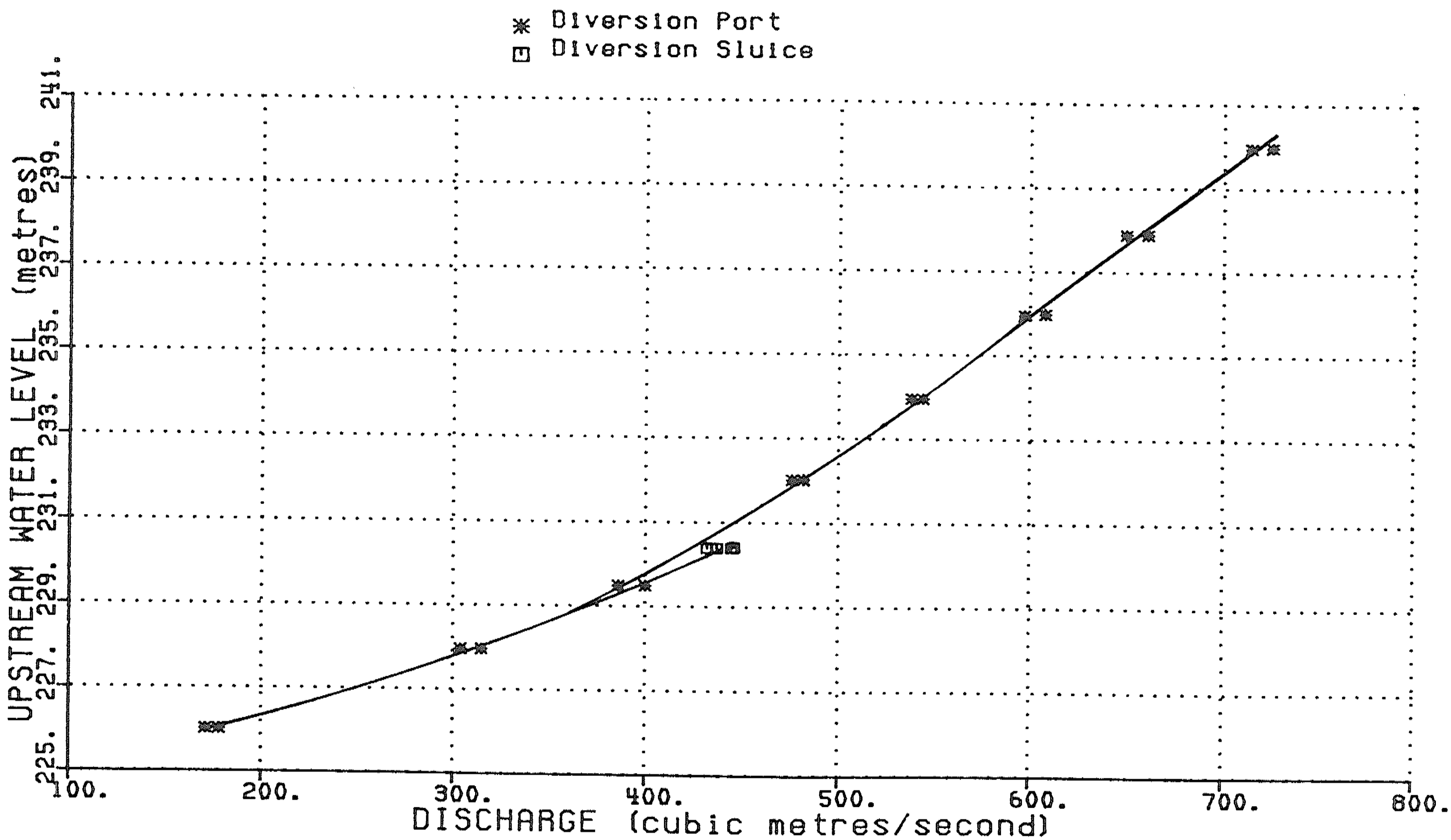

Figure 5.3 Discharge Rating Curve for the Diversion Ports and Sluice of the Overflow Spillway With an Invert Level of 218.5 m

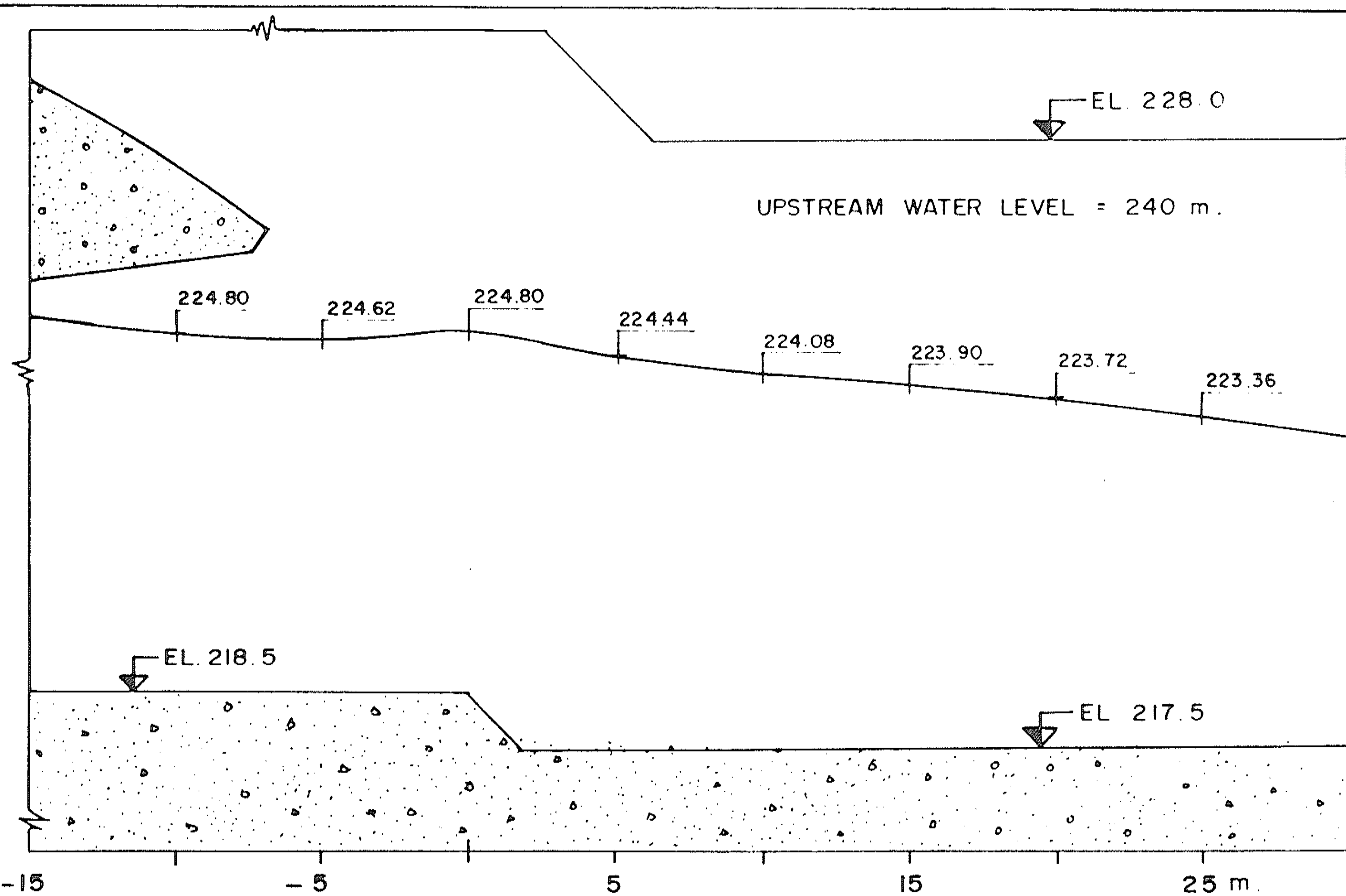

Figure 5.4 Water Surface Profile along the Left Pier of the Diversion Port of the Overflow Spillway With an Upstream Water Elevation of 240.0 m

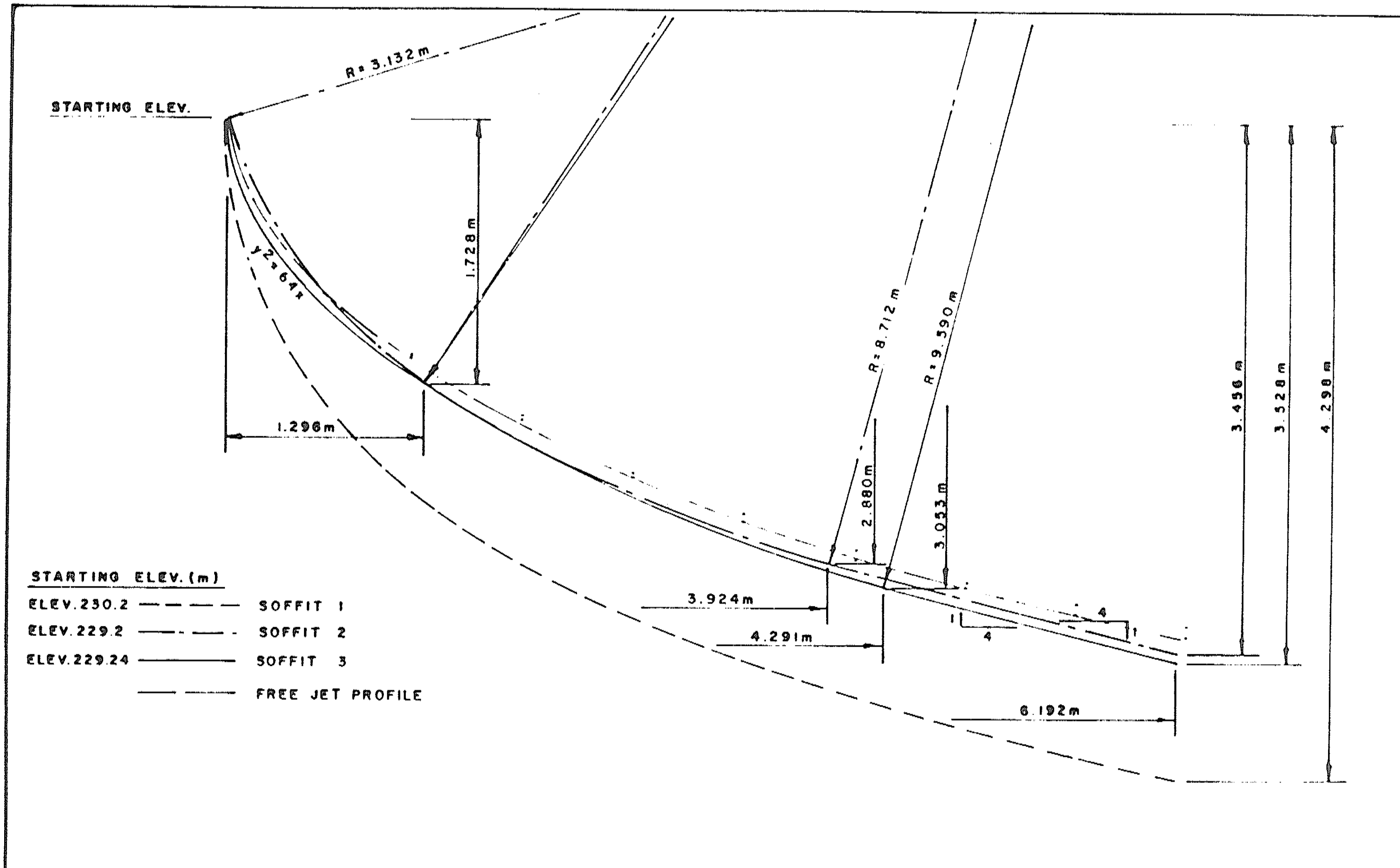

Figure 5.5 Designs of the Soffit Alternatives and the Profile of a Free Jet

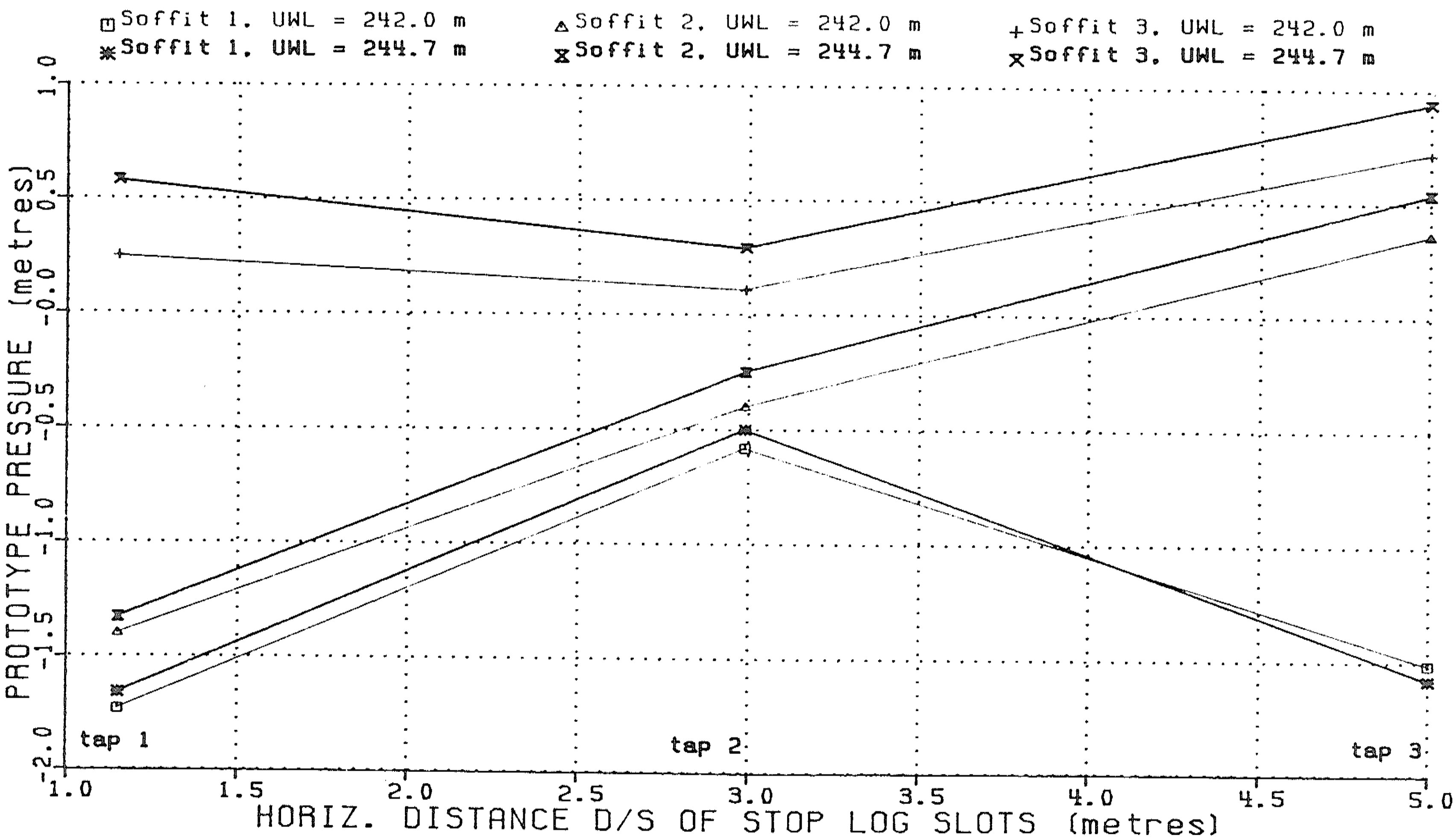

Figure 5.6 Prototype Pressures Measured Along the Centerlines of the Three Soffits

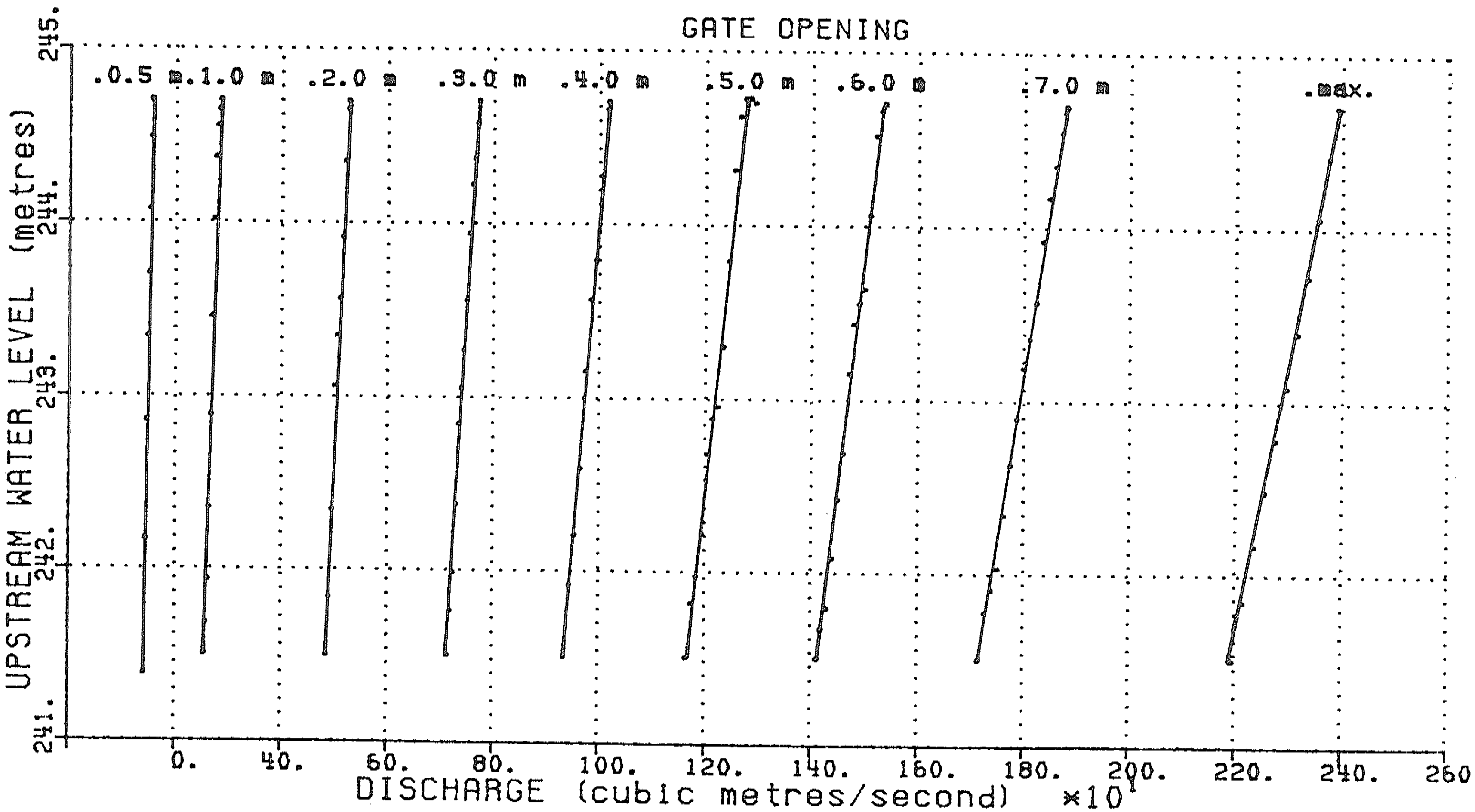

Figure 5.7 Discharge Rating Curves of the Orifice Spillway for Several Gate Openings

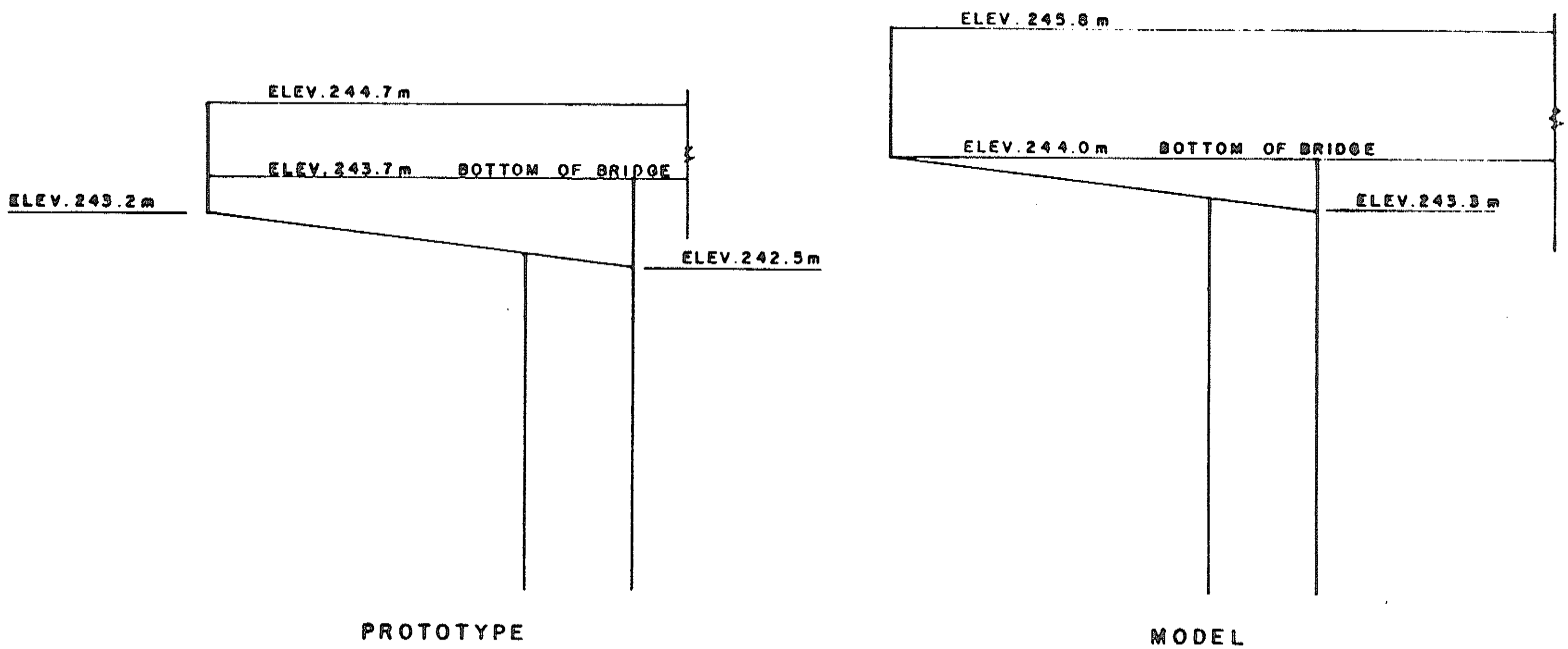

Figure 5.8 Overflow Spillway Pier Tops as Shown in the Preliminary Drawing and as Constructed on the Model

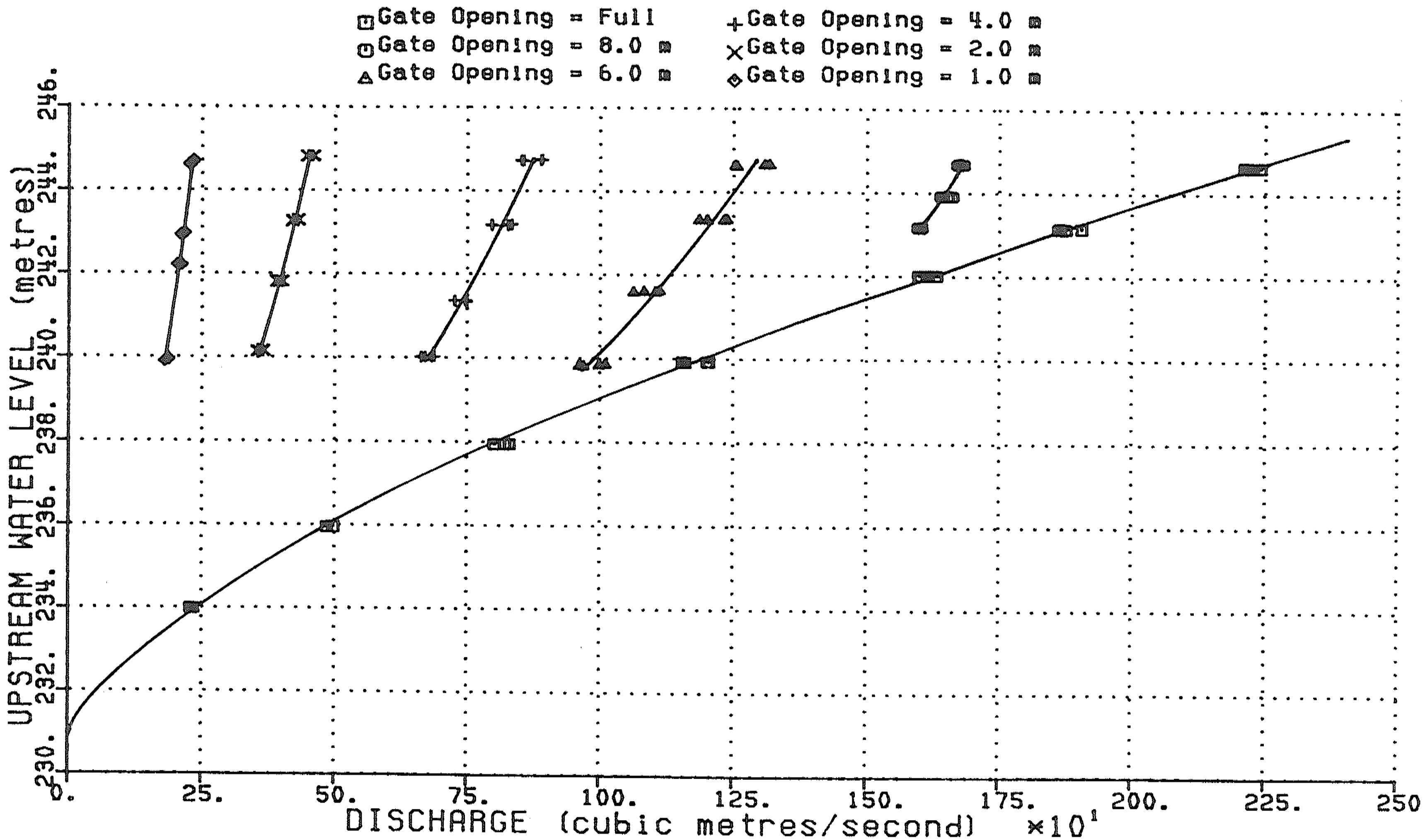

Figure 5.9 Discharge Rating Curves of the Overflow Spillway for Several Gate Openings

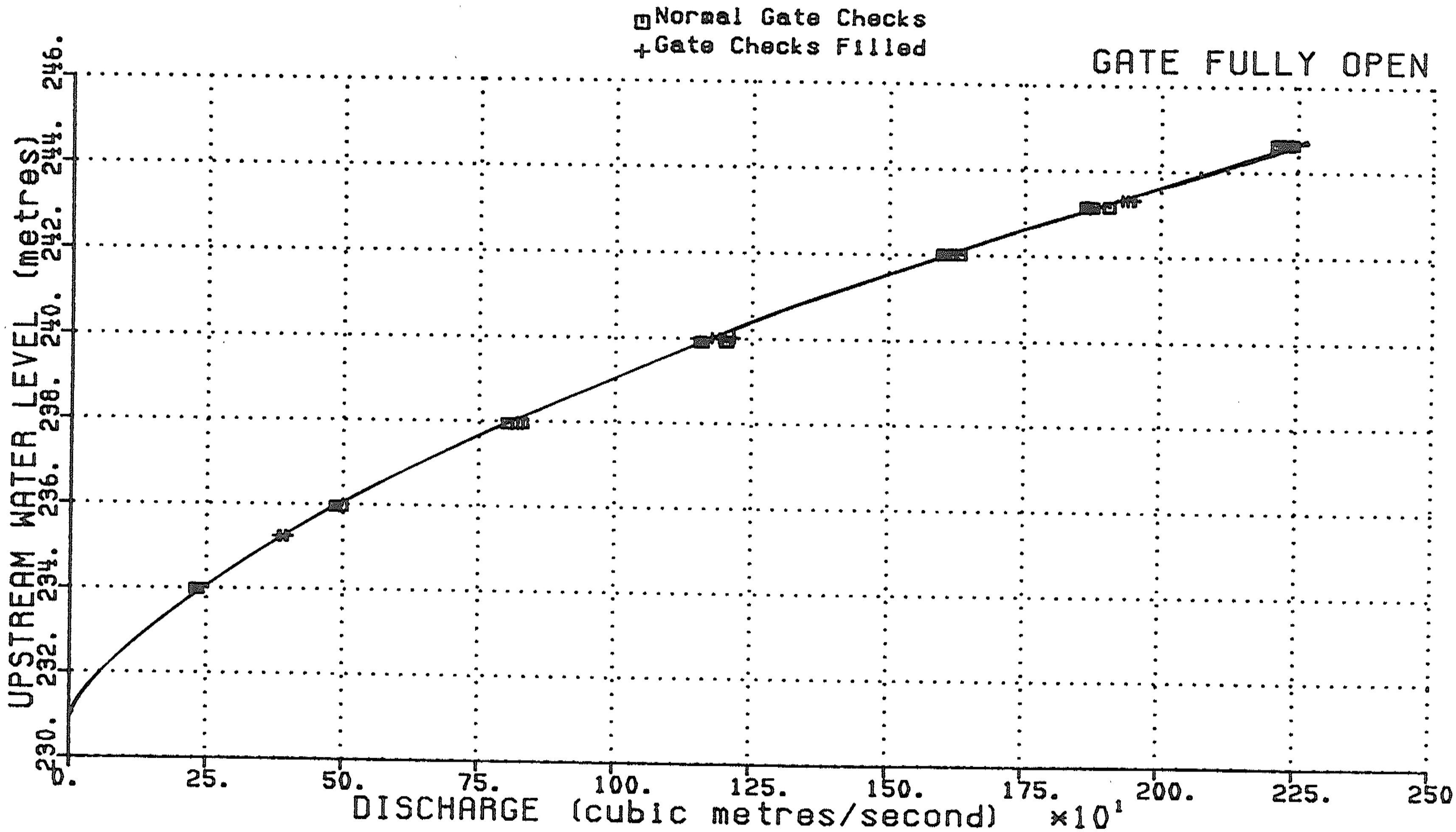

Figure 5.10 Discharge Rating Data for the Overflow Spillway With and Without Gate Checks

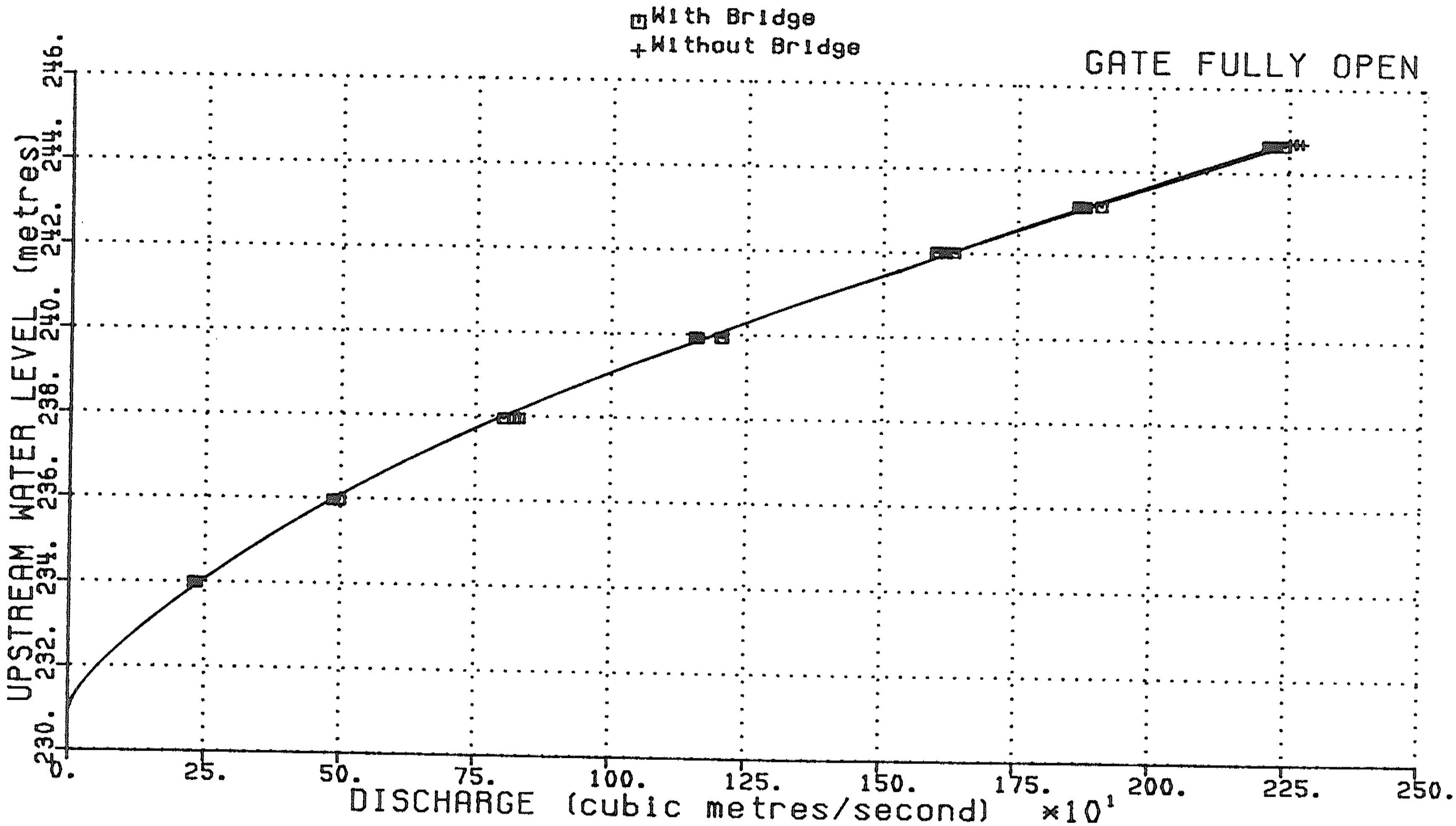

Figure 5.11 Discharge Rating Data for the Overflow Spillway With and Without the Permanent Bridge

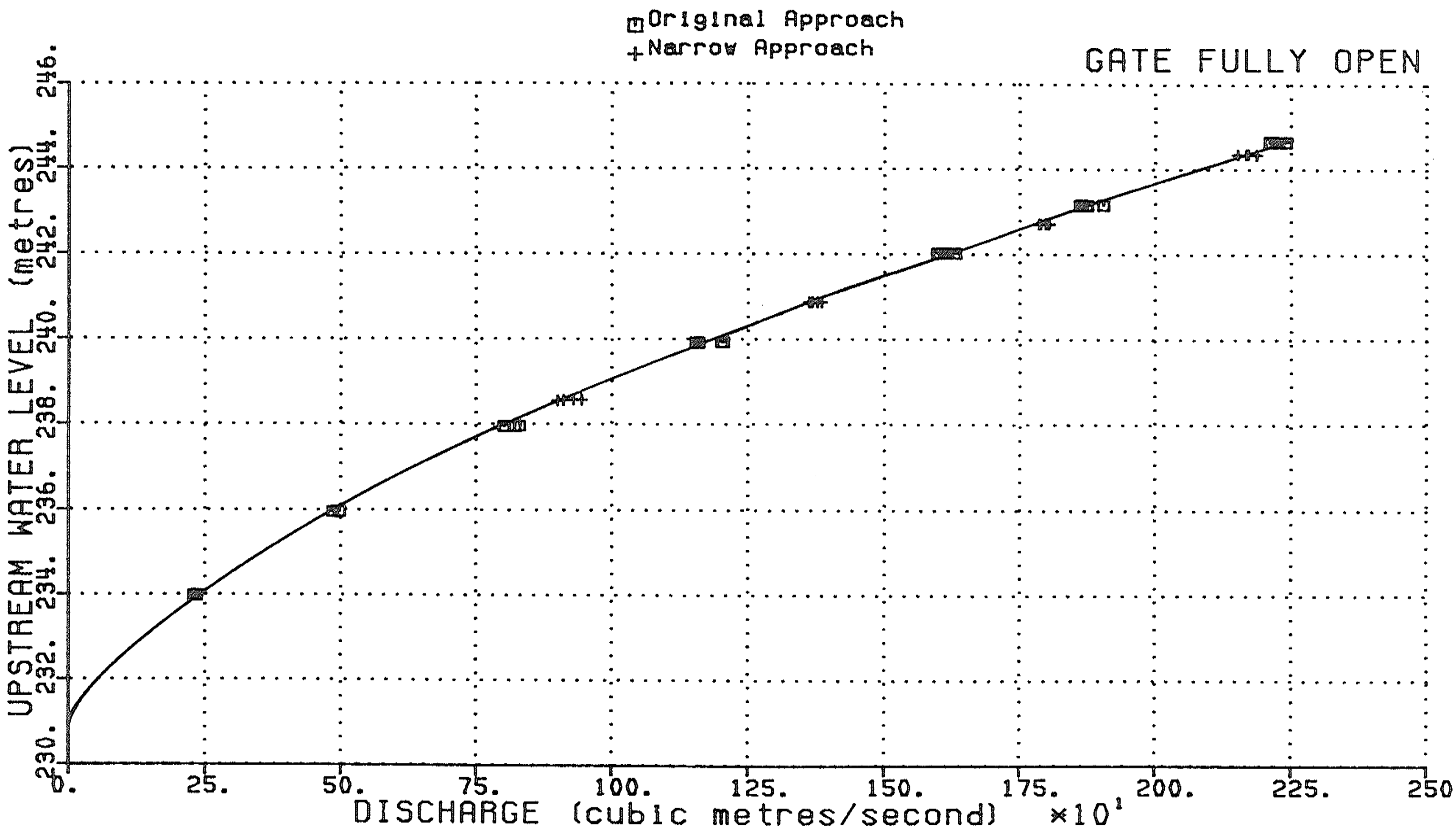

Figure 5.12 Discharge Rating Data for the Overflow Spillway With the Original Approach Conditions and With a Narrow Approach

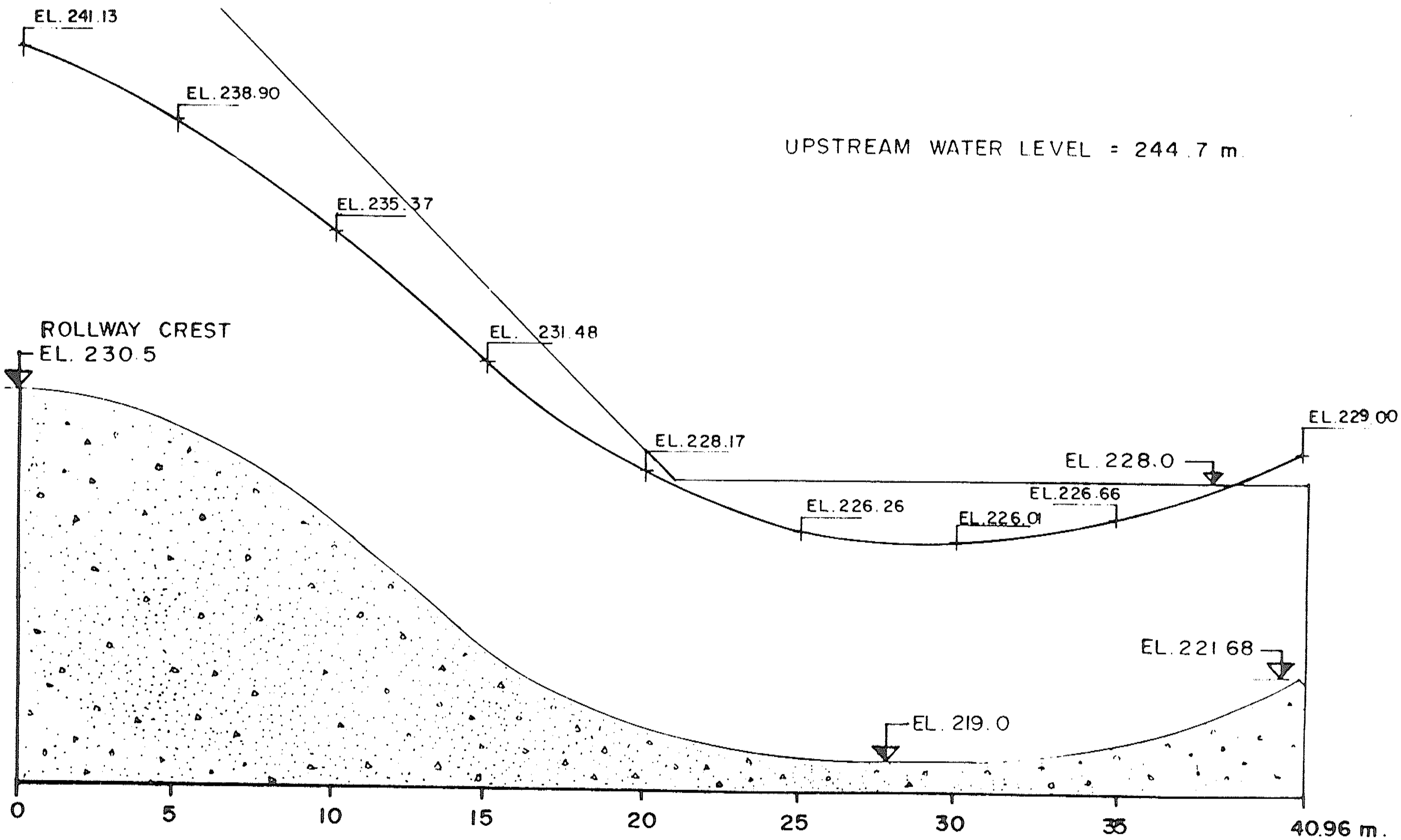

Figure 5.13 Water Surface Profile Along the Left Pier of the Overflow Spillway With an Upstream Water Elevation of 244.7 m

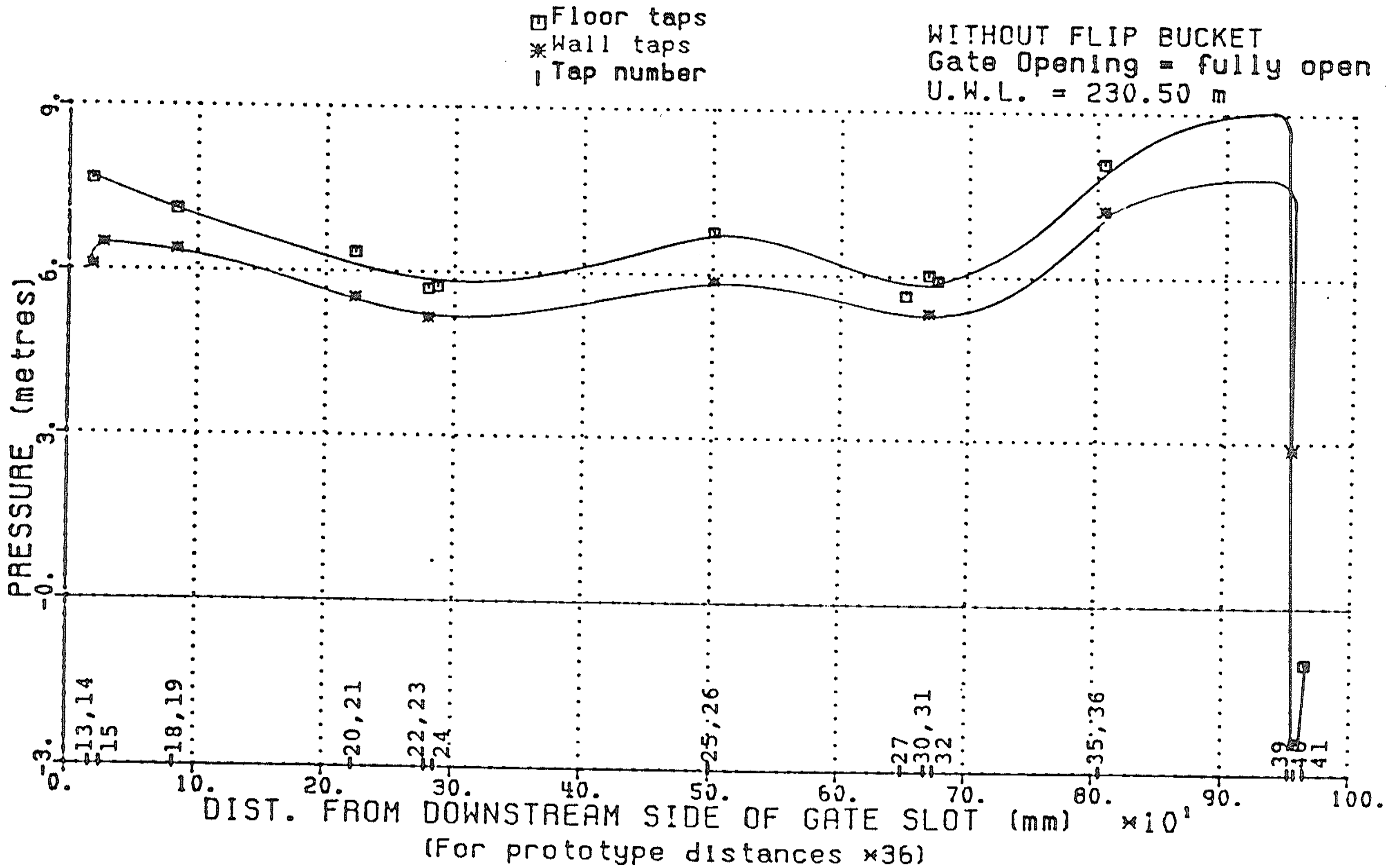

Figure 5.14 Pressure Distributions Along the Diversion Configuration of the Orifice Spillway With an Upstream Water Elevation of 230.50 m

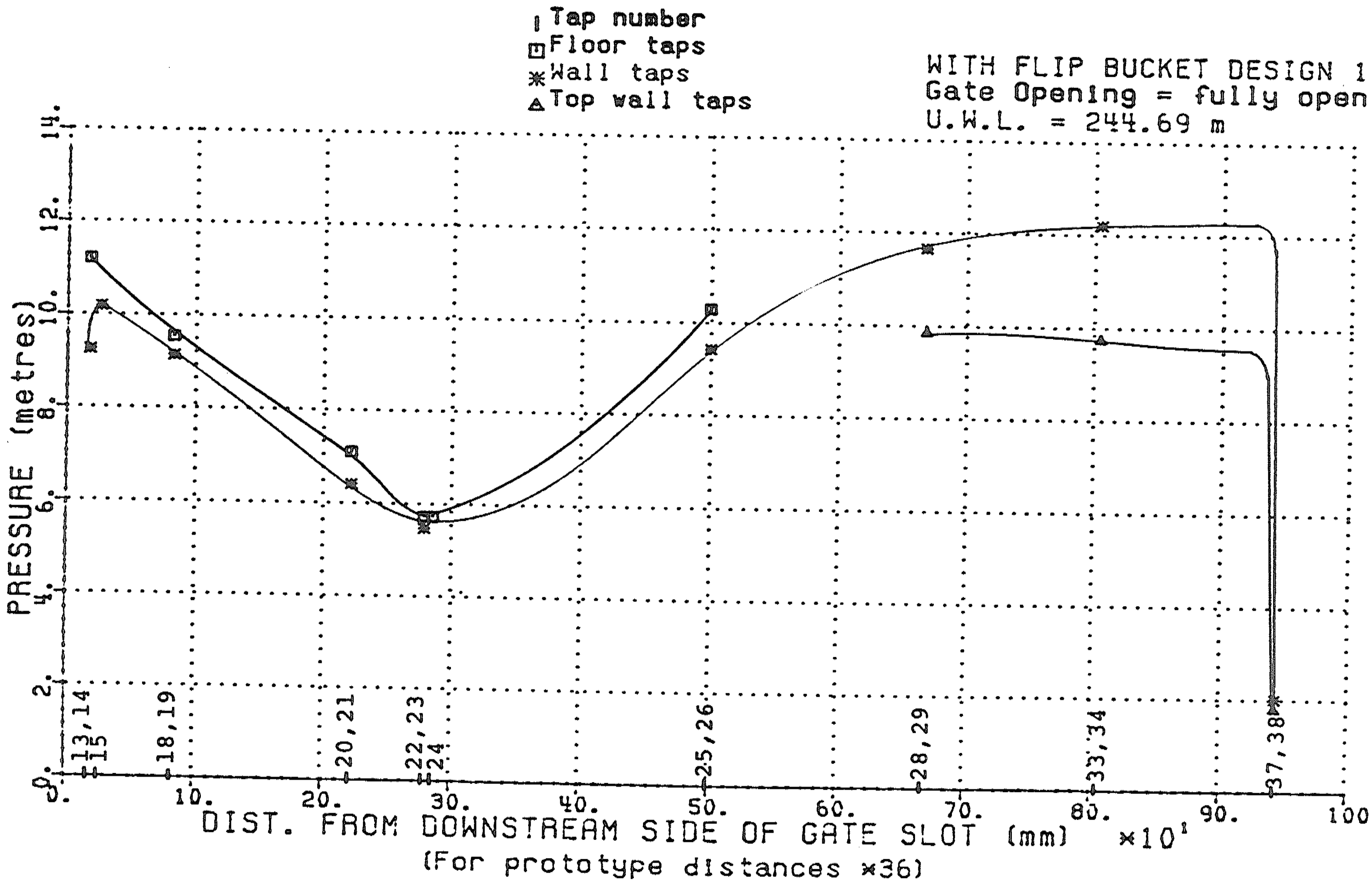

Figure 5.15 Orifice Spillway Pressure Distributions With the Forebay Near its Maximum Level and the Gate Fully Open

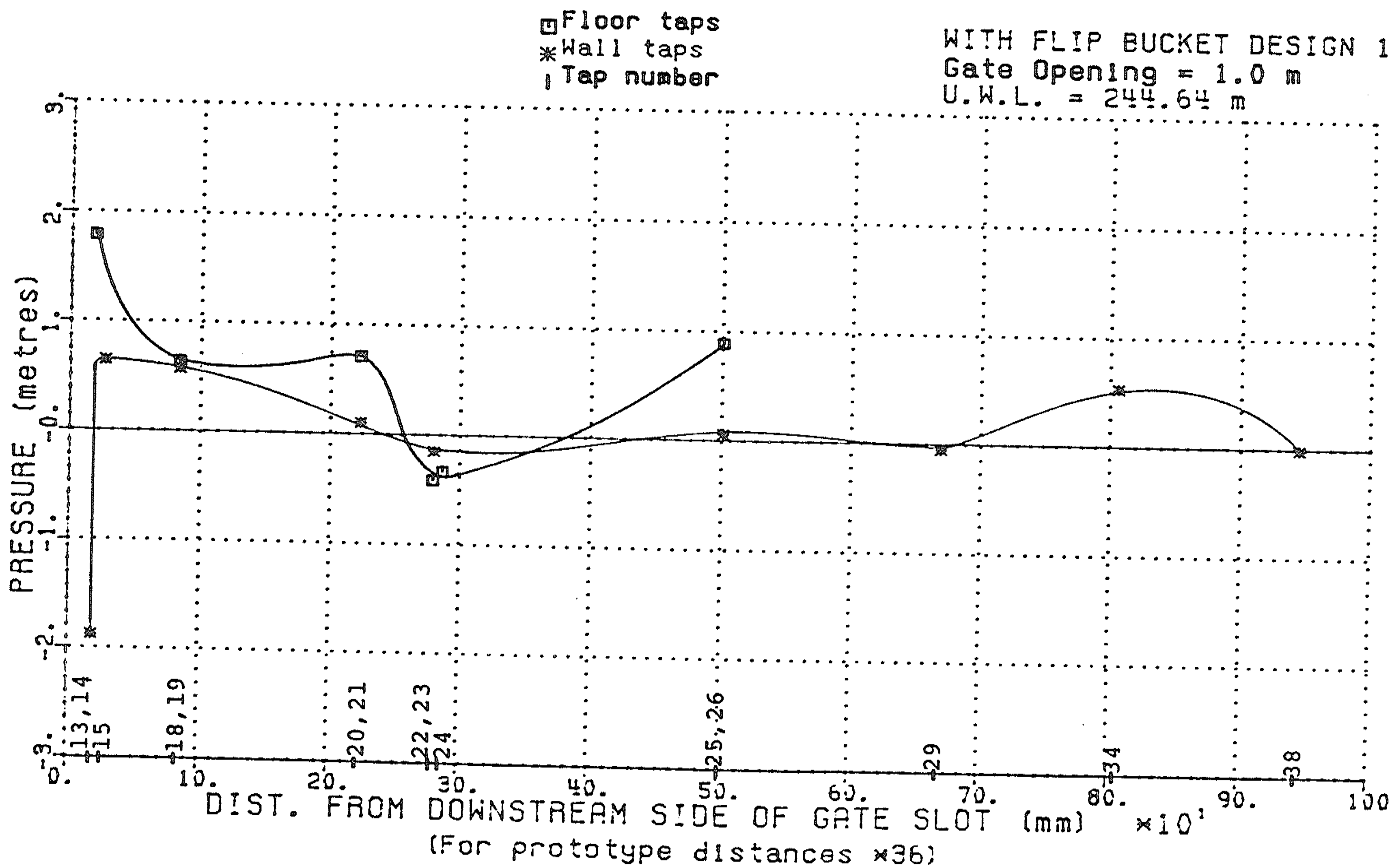

Figure 5.16 Orifice Spillway Pressure Distributions With the Forebay Near its Maximum Level and a 1.0 m Gate Opening

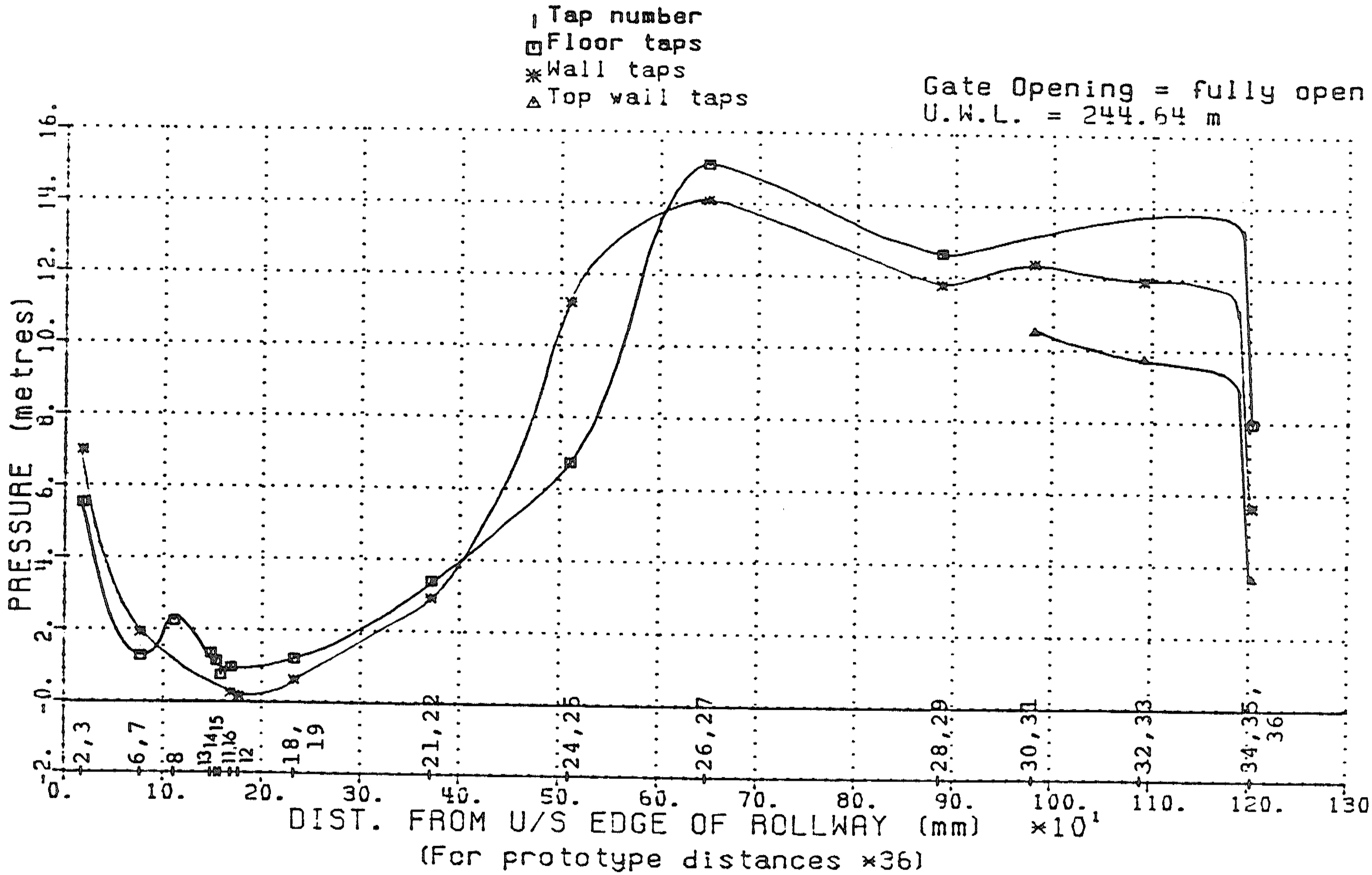

Figure 5.17 Overflow Spillway Pressure Distributions With the Forebay Near its Maximum Level and the Gate Fully Open

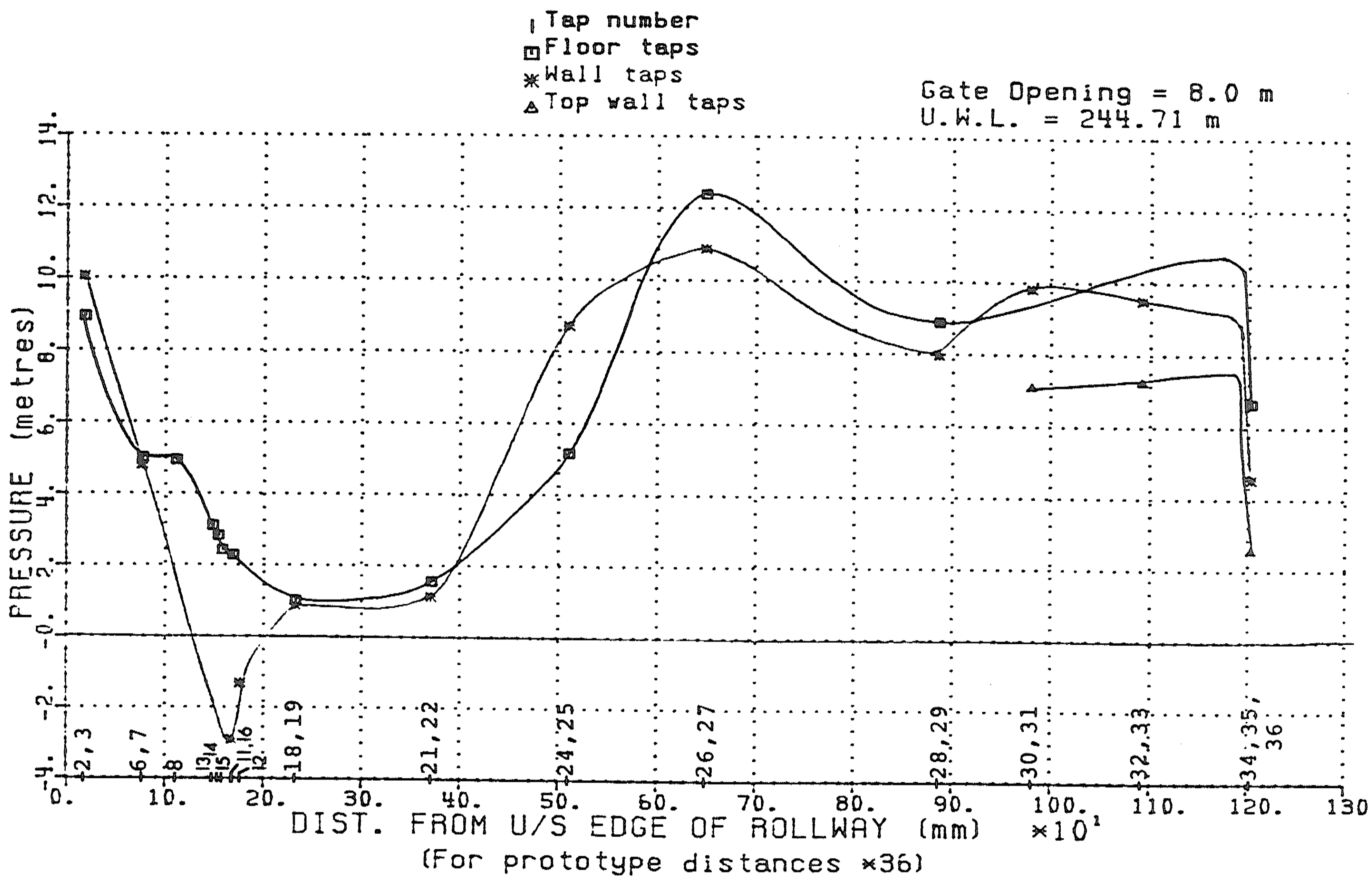

Figure 5.18 Overflow Spillway Pressure Distributions With the Forebay Near its Maximum Level and an 8.0 m Gate Opening

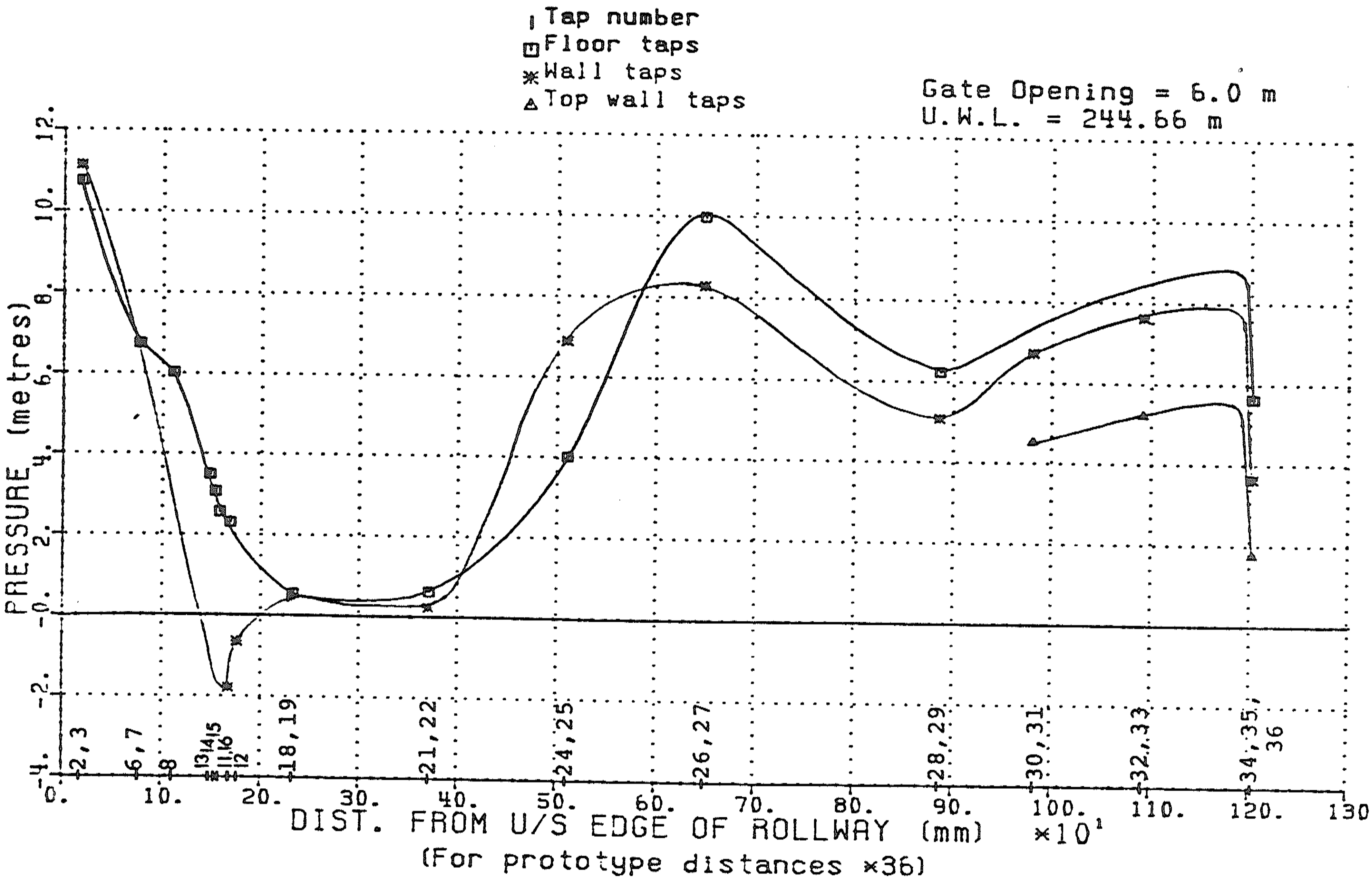

Figure 5.19 Overflow Spillway Pressure Distributions With the Forebay Near its Maximum Level and a 6.0 m Gate Opening

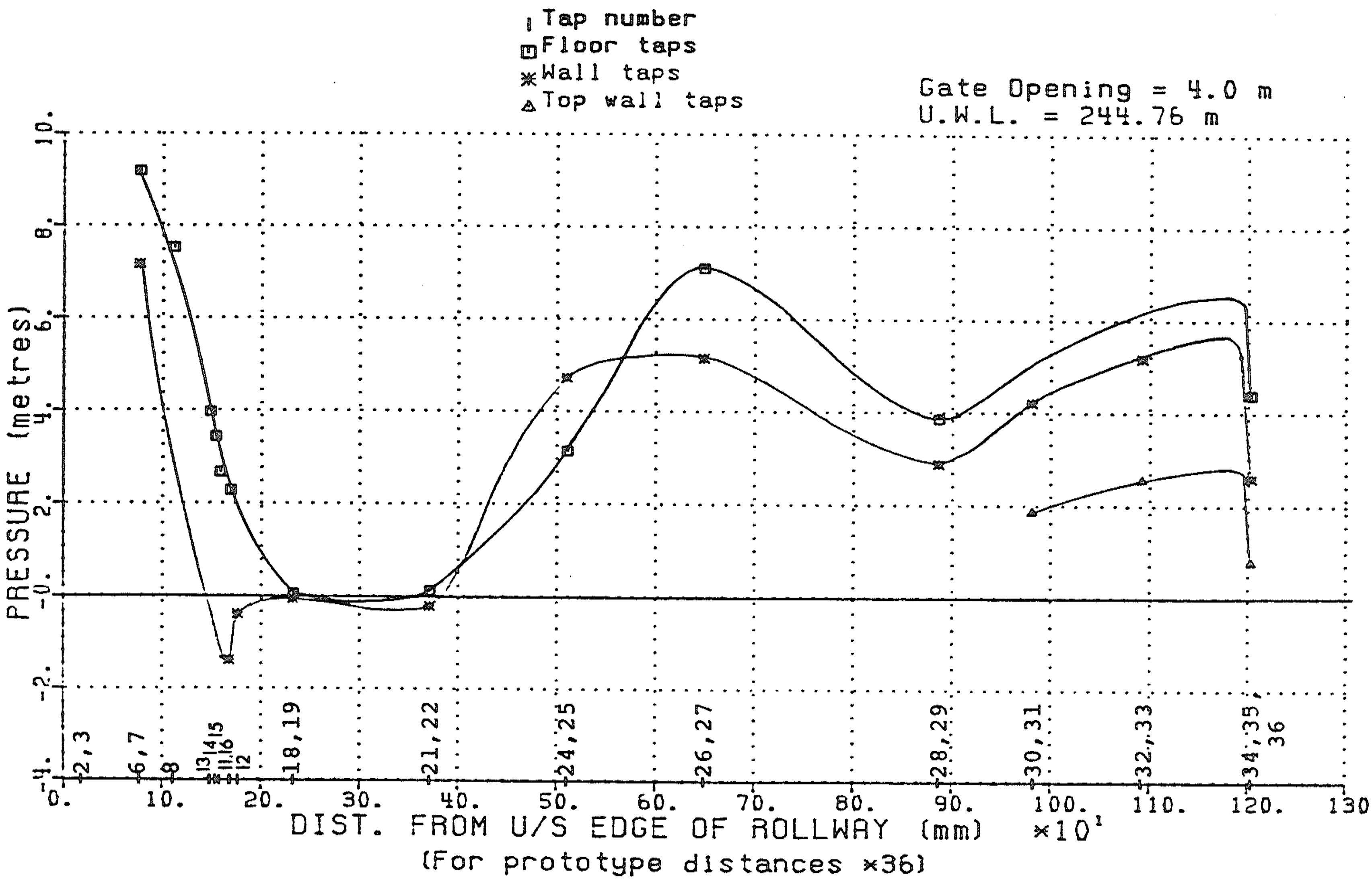

Figure 5.20 Overflow Spillway Pressure Distributions With the Forebay Near its Maximum Level and a 4.0 m Gate Opening

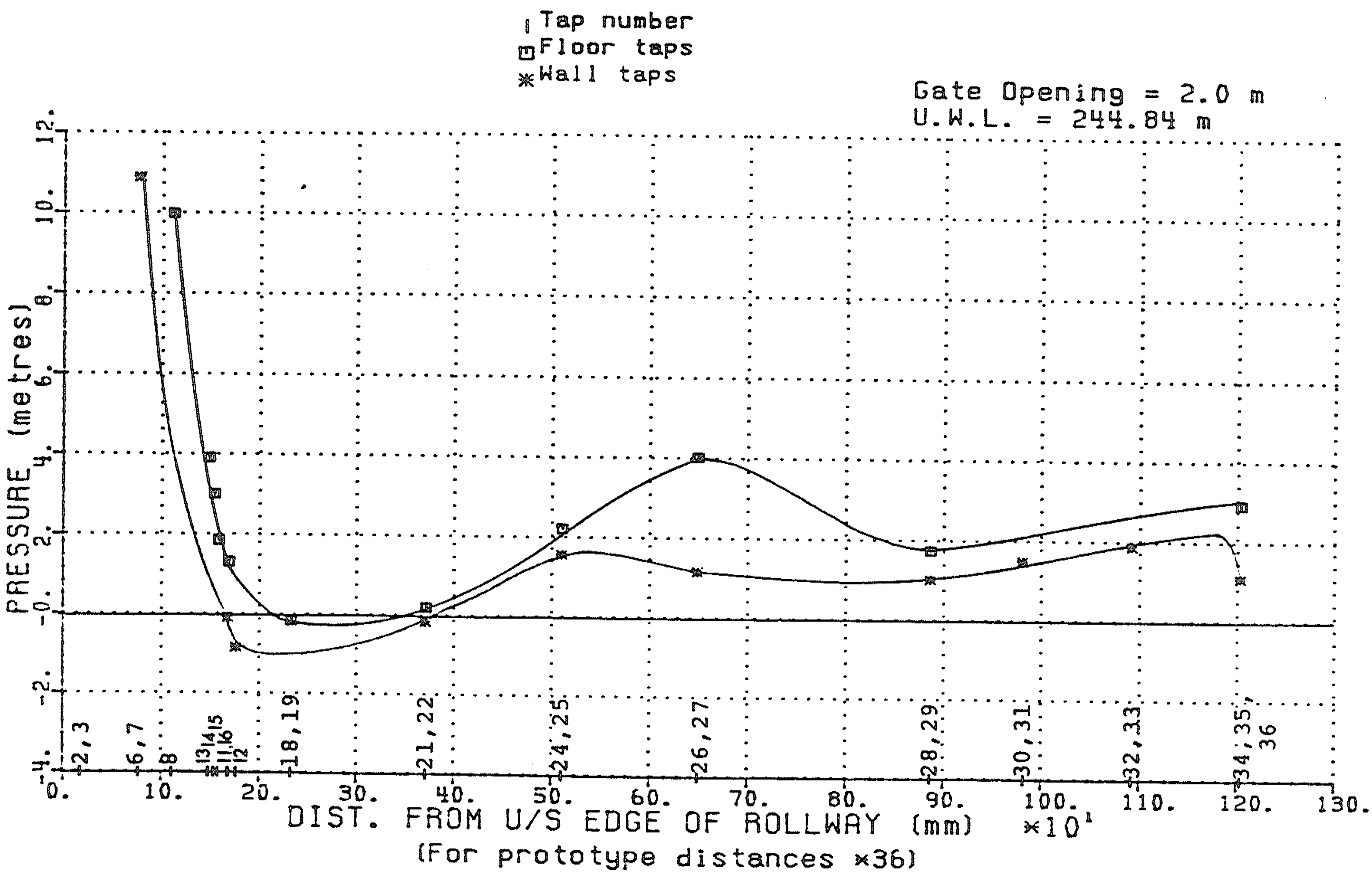

Figure 5.21 Overflow Spillway Pressure Distributions With the Forebay Near its Maximum Level and a 2.0 m Gate Opening

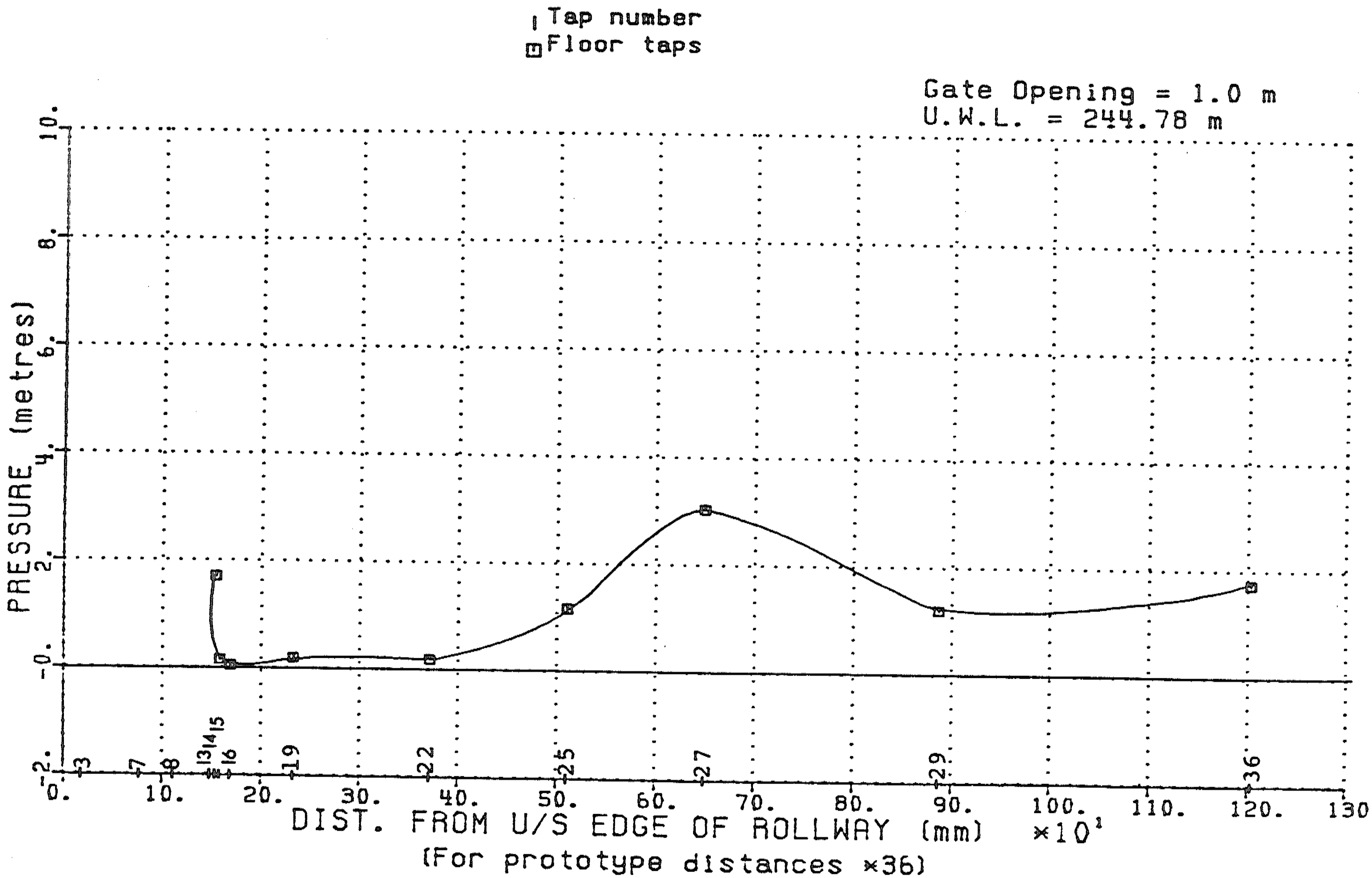

Figure 5.22 Overflow Spillway Pressure Distributions With the Forebay Near its Maximum Level and a 1.0 m Gate Opening

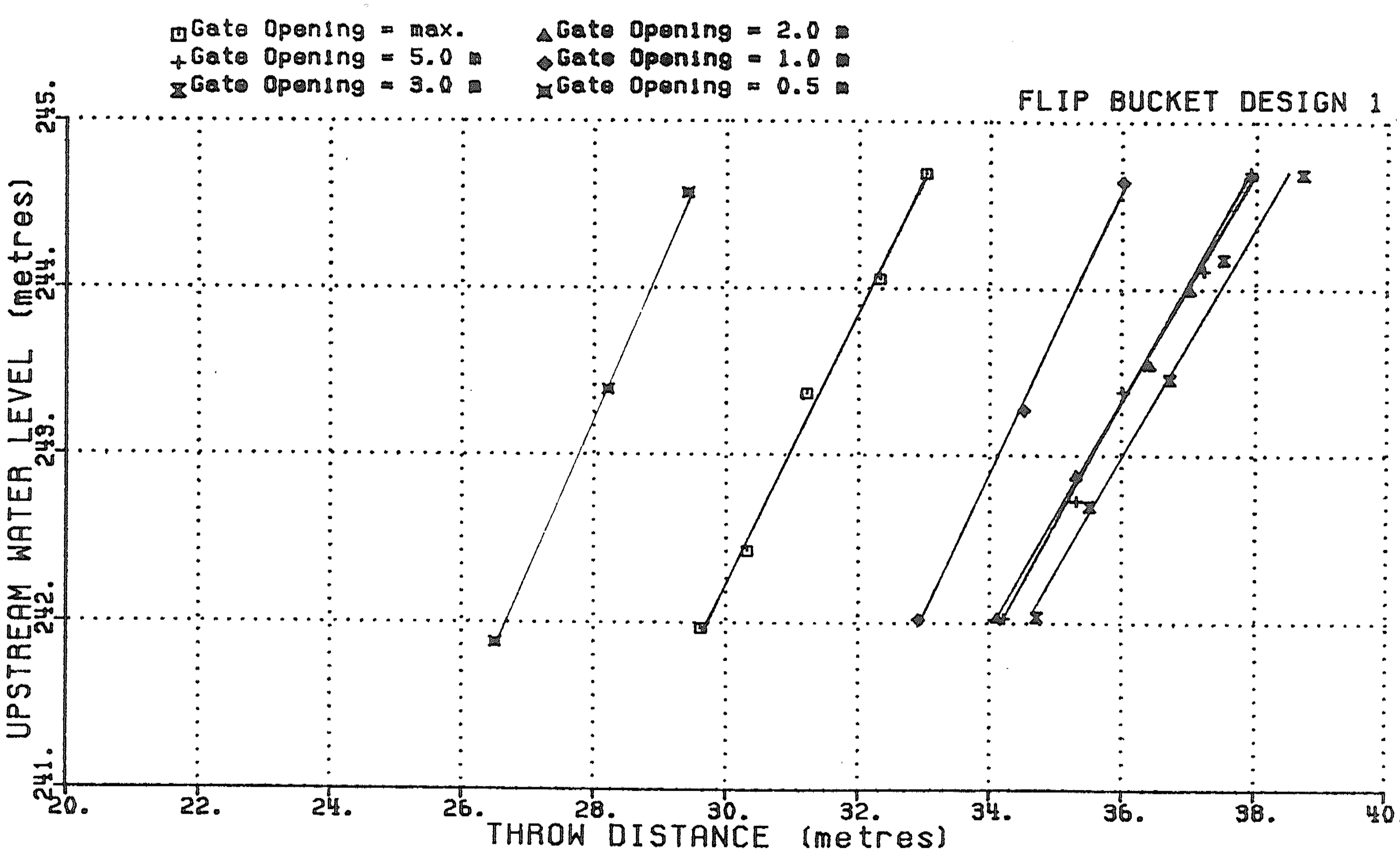

Figure 5.23 Orifice Spillway Flip Bucket Throw Distances for Various Gate Openings With Bucket 1

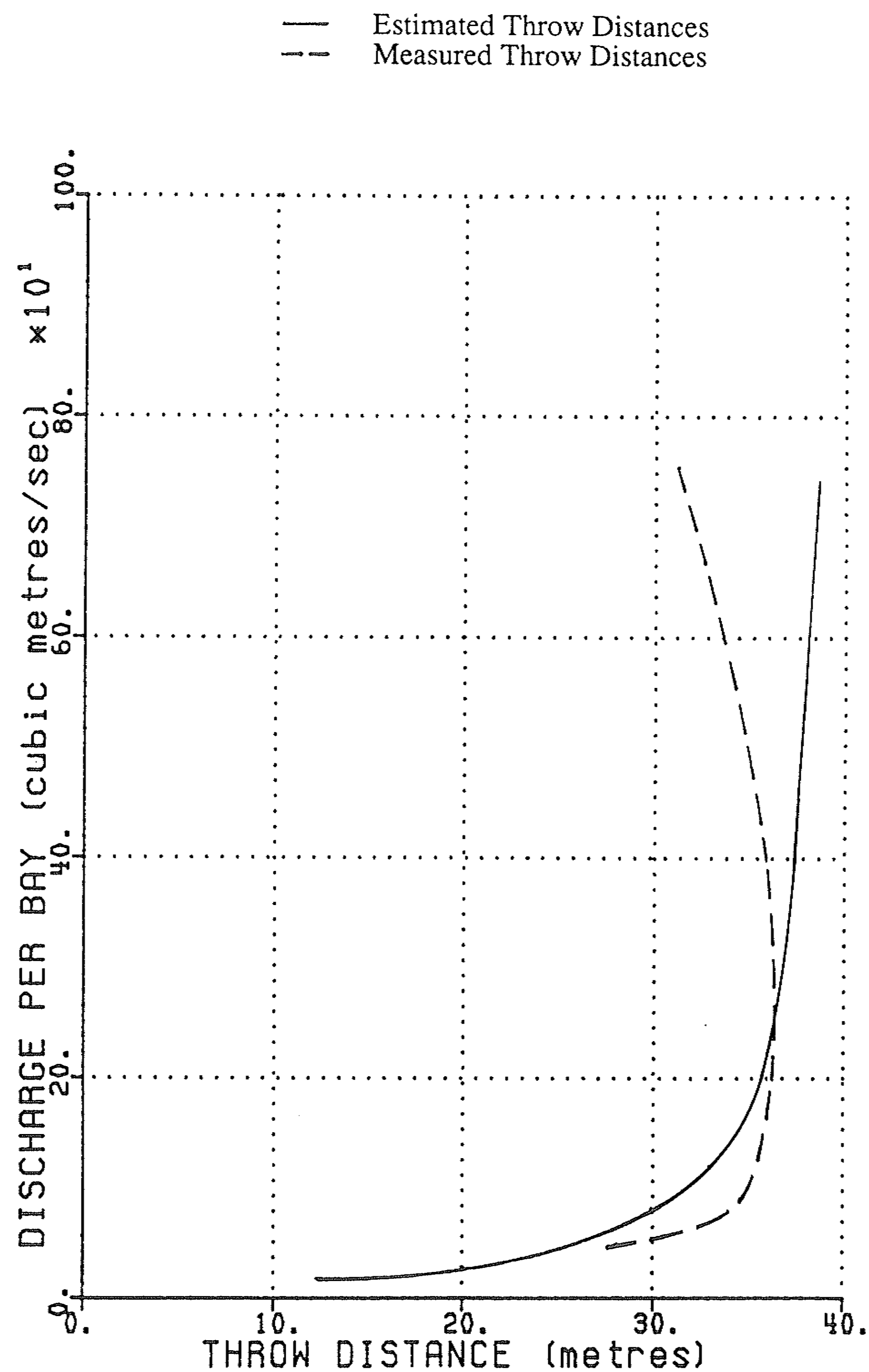

Figure 5.24 Comparison of Estimated and Measured Throw Distances for Bucket 1 of the Orifice Spillway

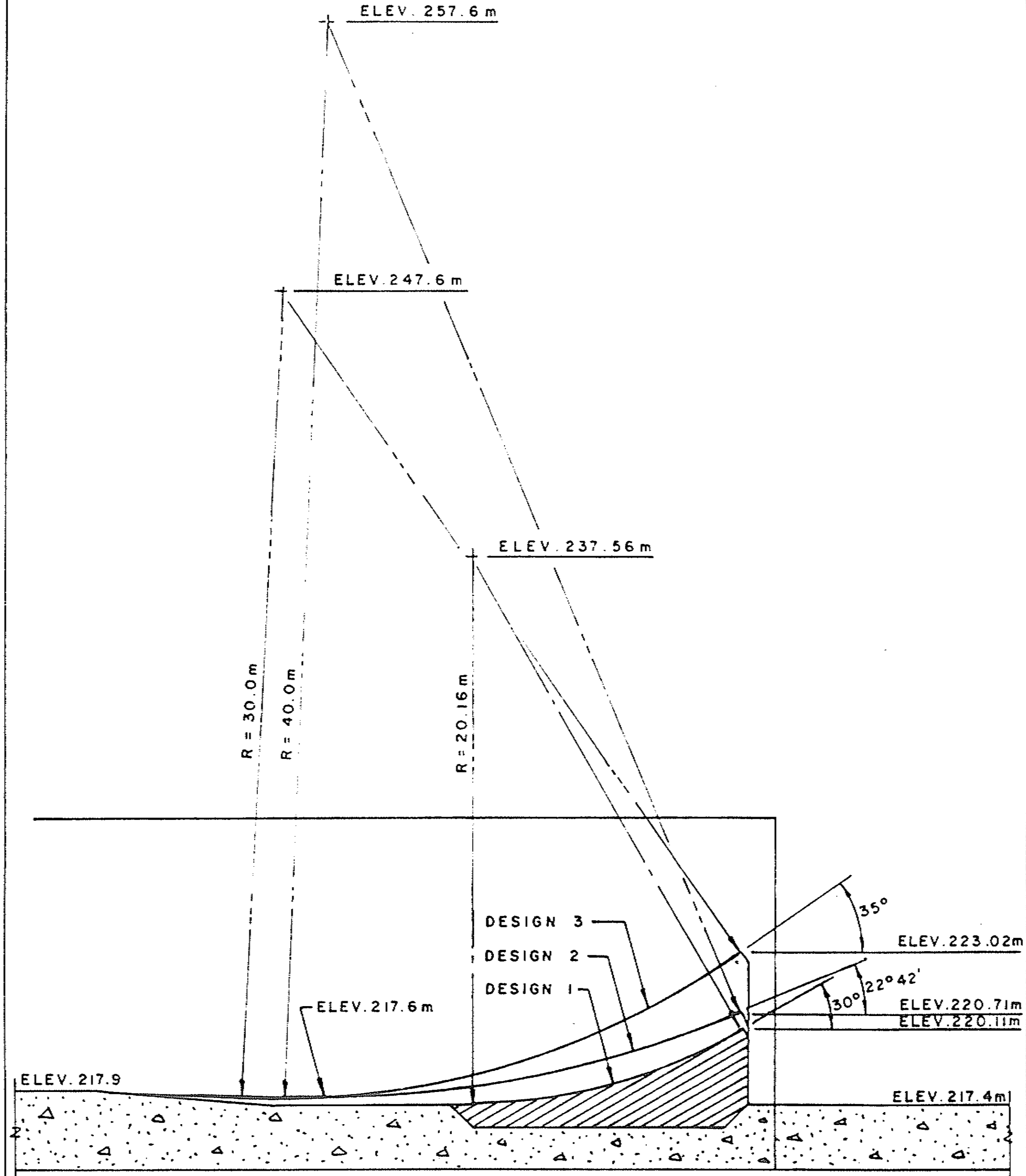

Figure 5.25 Orifice Spillway Flip Bucket Design Alternatives

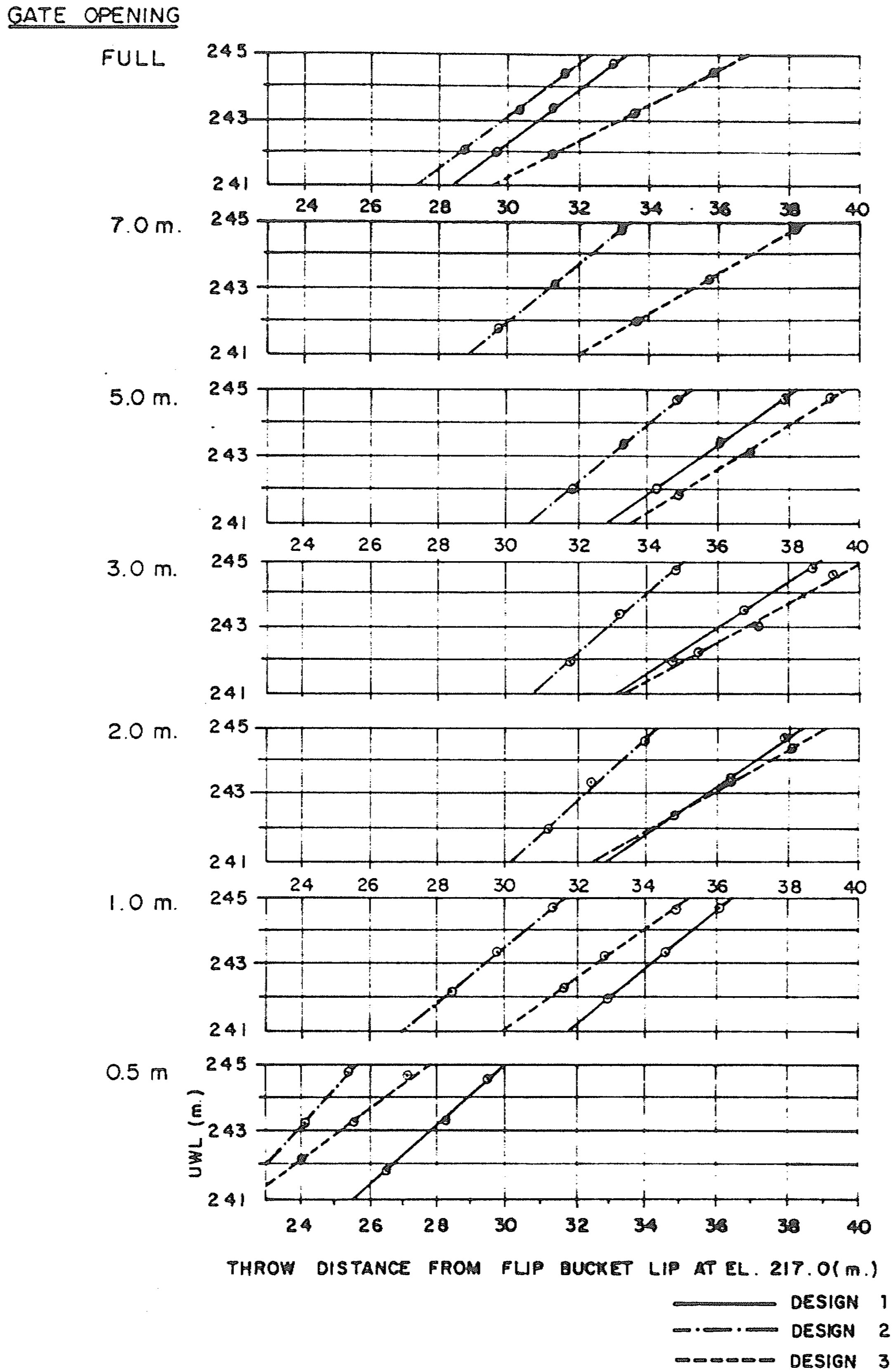

Figure 5.26 Comparison of Throw Distances for the Three Flip Bucket Alternatives of the Orifice Spillway

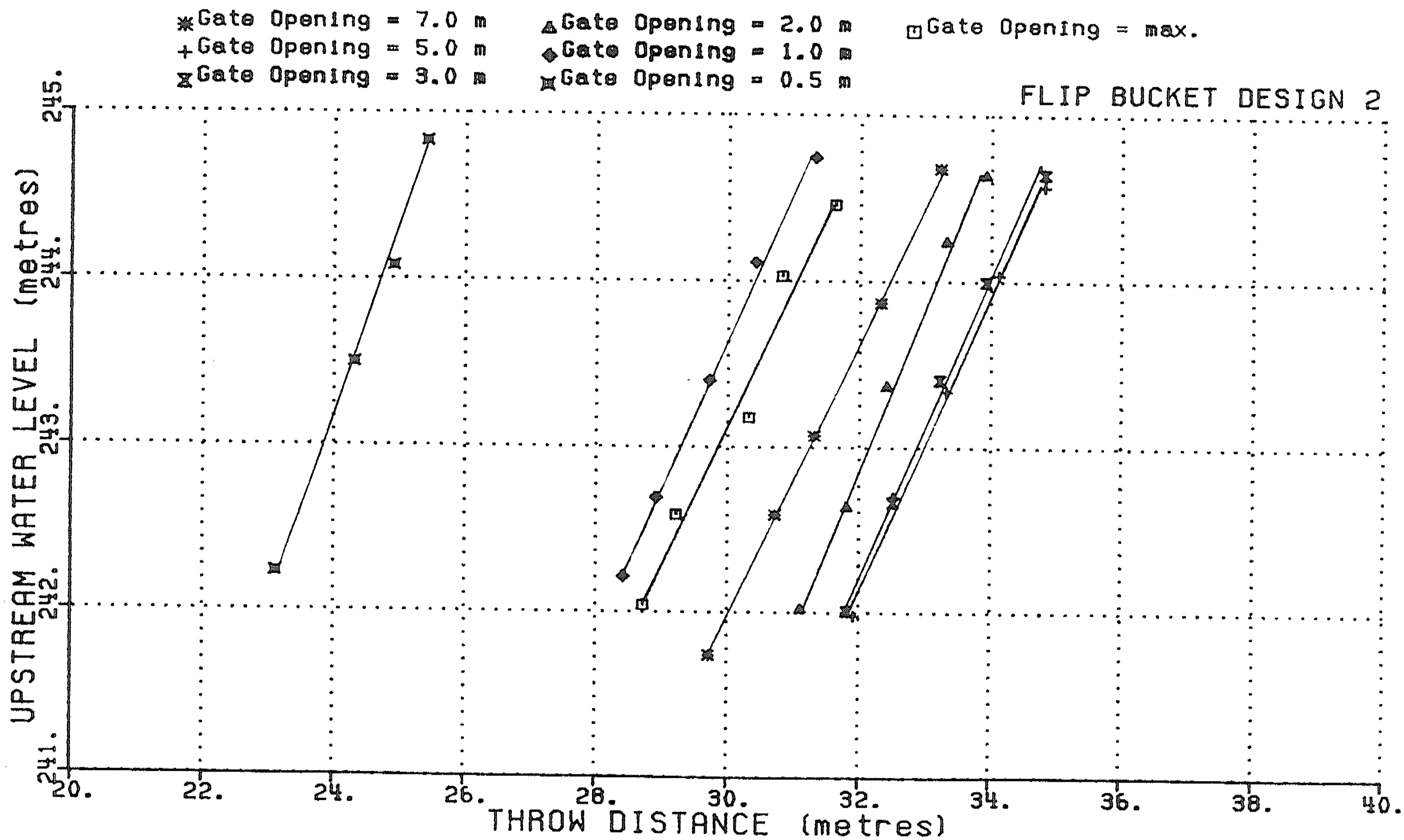

Figure 5.27 Orifice Spillway Flip Bucket Throw Distances for Various Gate Openings With Bucket 2

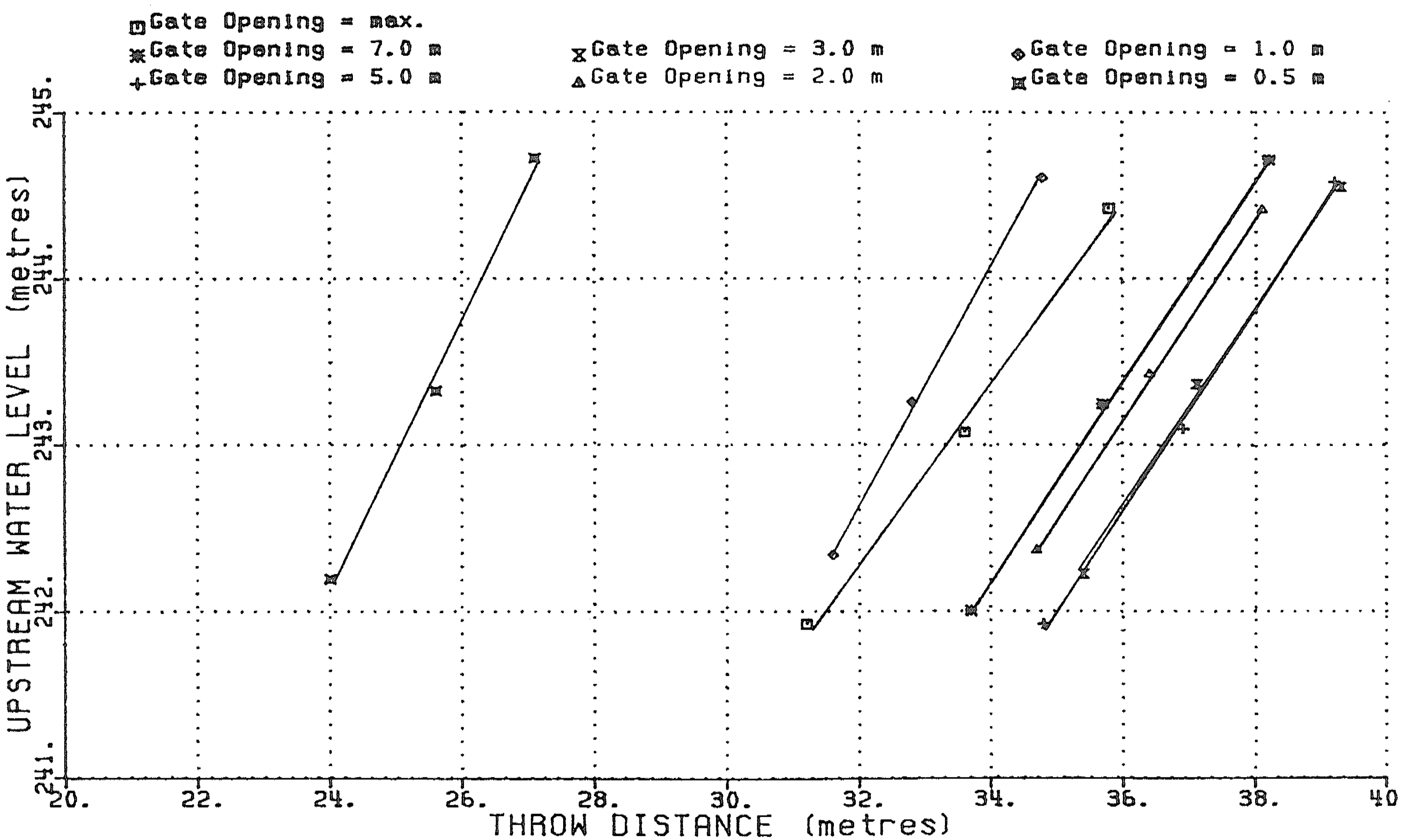

Figure 5.28 Orifice Spillway Flip Bucket Throw Distances for Various Gate Openings With Bucket 3

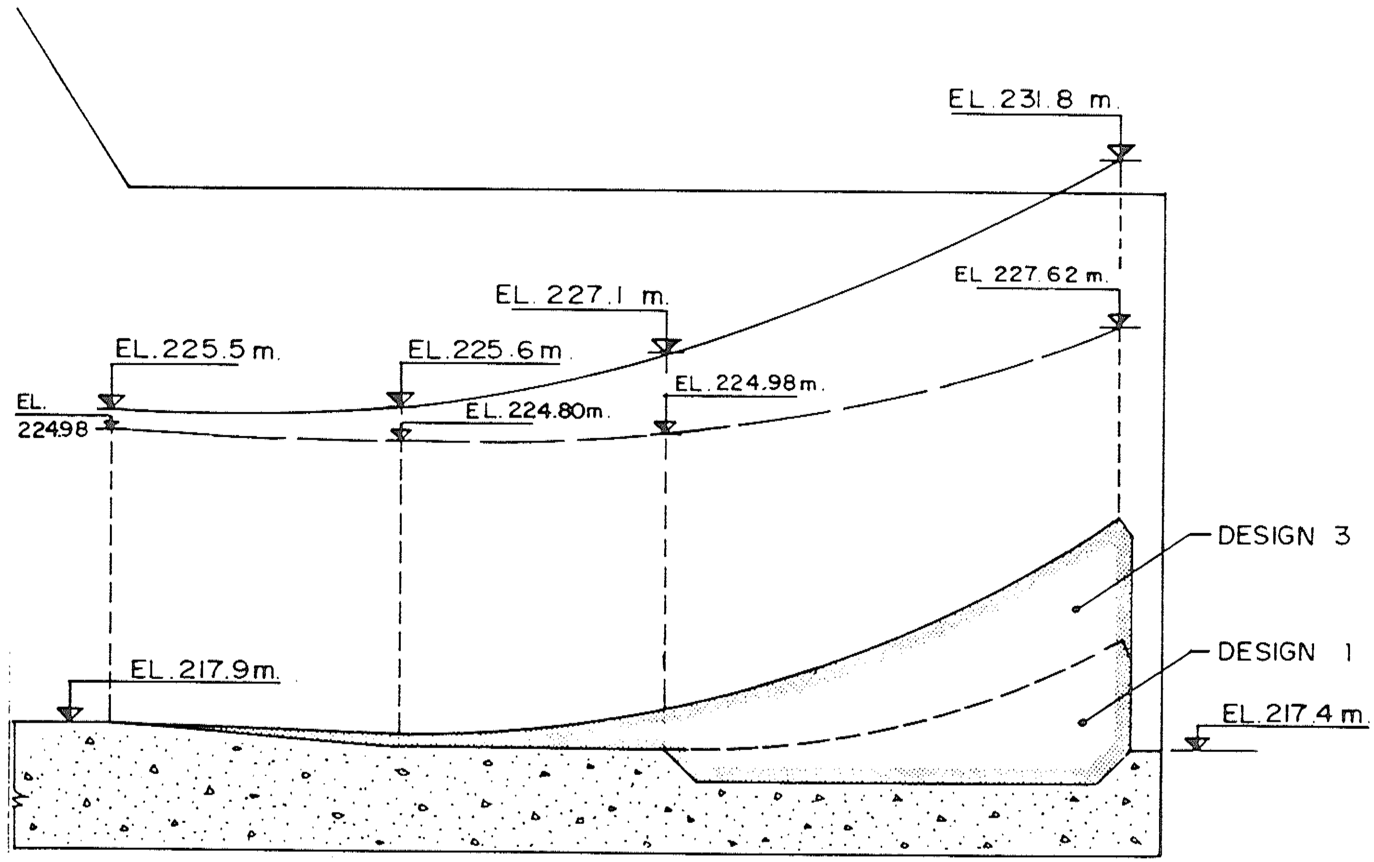

Figure 5.29 Water Surface Profile Along the Left Pier of the Orifice Spillway With an Upstream Water Elevation of 244.7 m

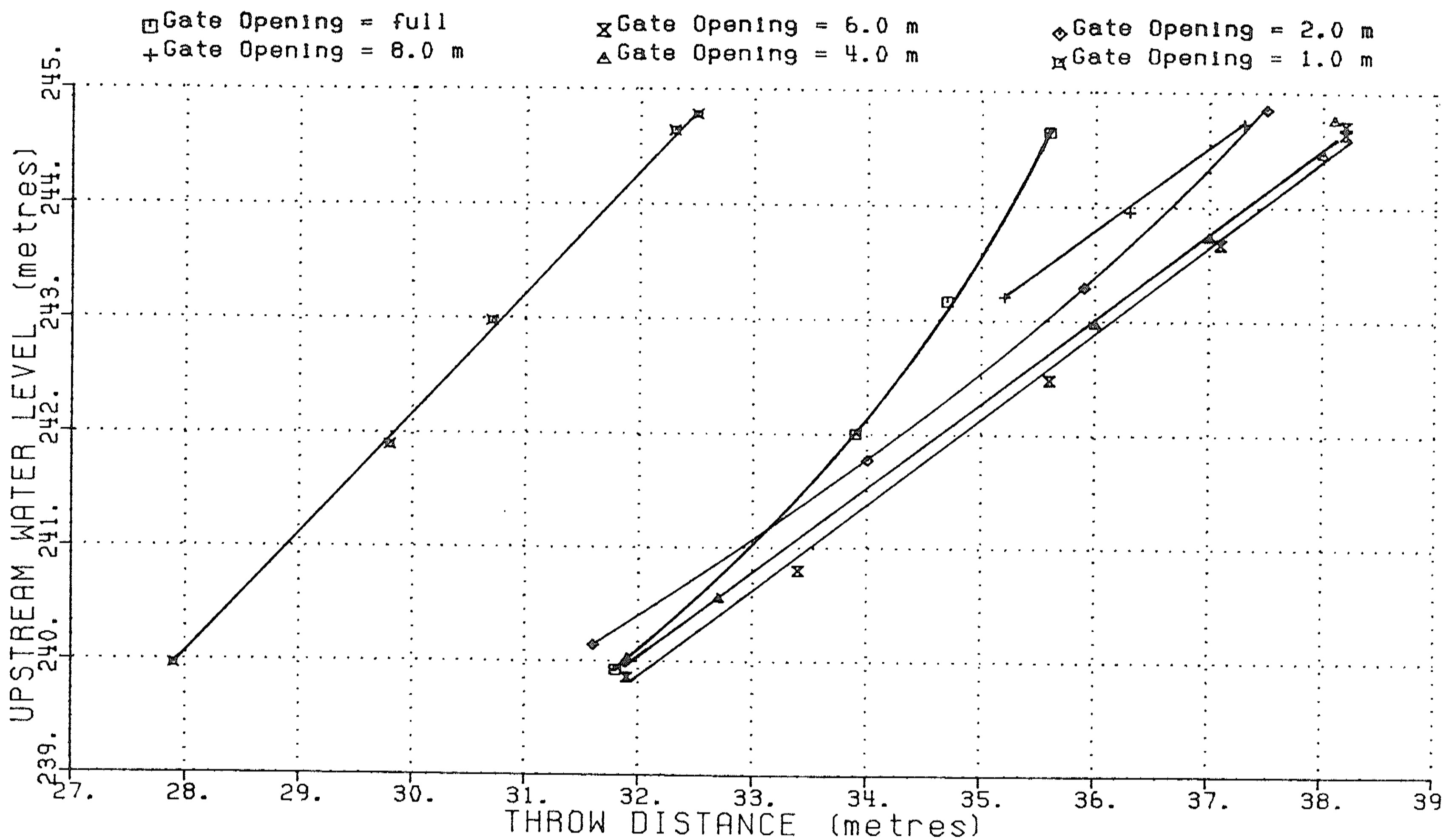

Figure 5.30 Overflow Spillway Flip Bucket Throw Distances for Various Gate Openings

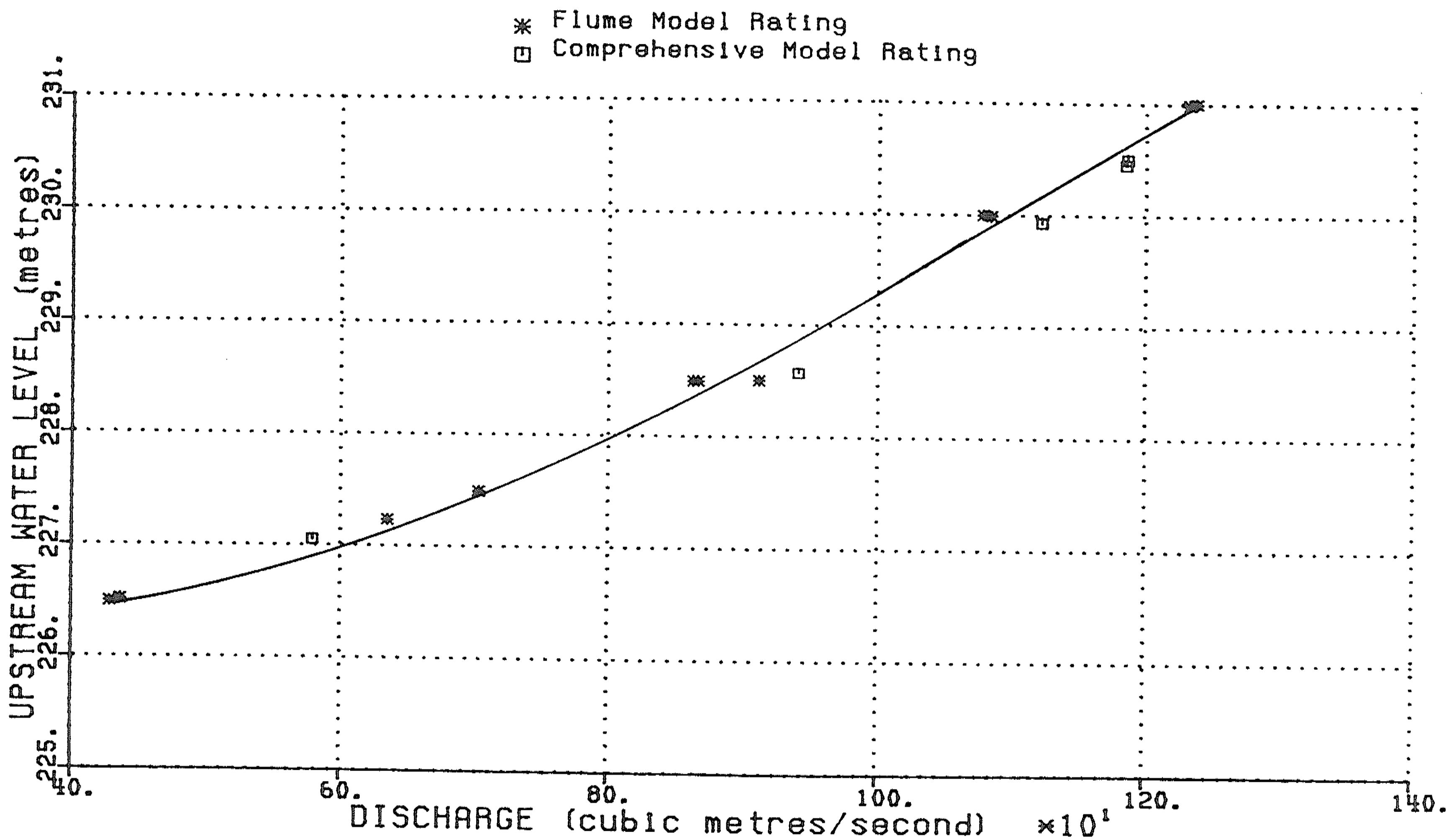

Figure 5.31 Orifice Spillway Comprehensive Model Rating Data Compared to the Flume Model Rating Curve for the Diversion Configuration

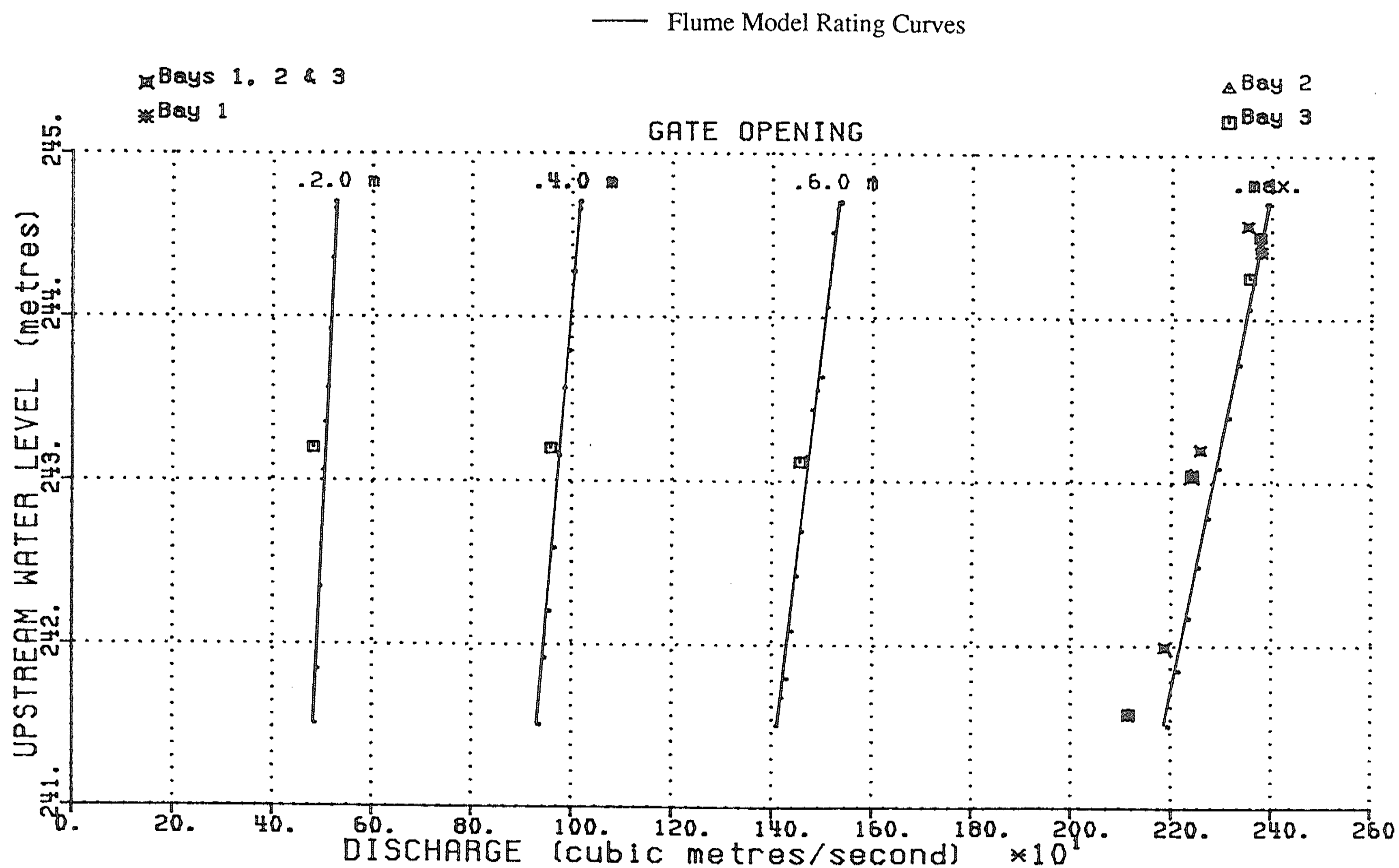

Figure 5.32 Orifice Spillway Comprehensive Model Rating Data Compared to the Flume Model Rating Curves

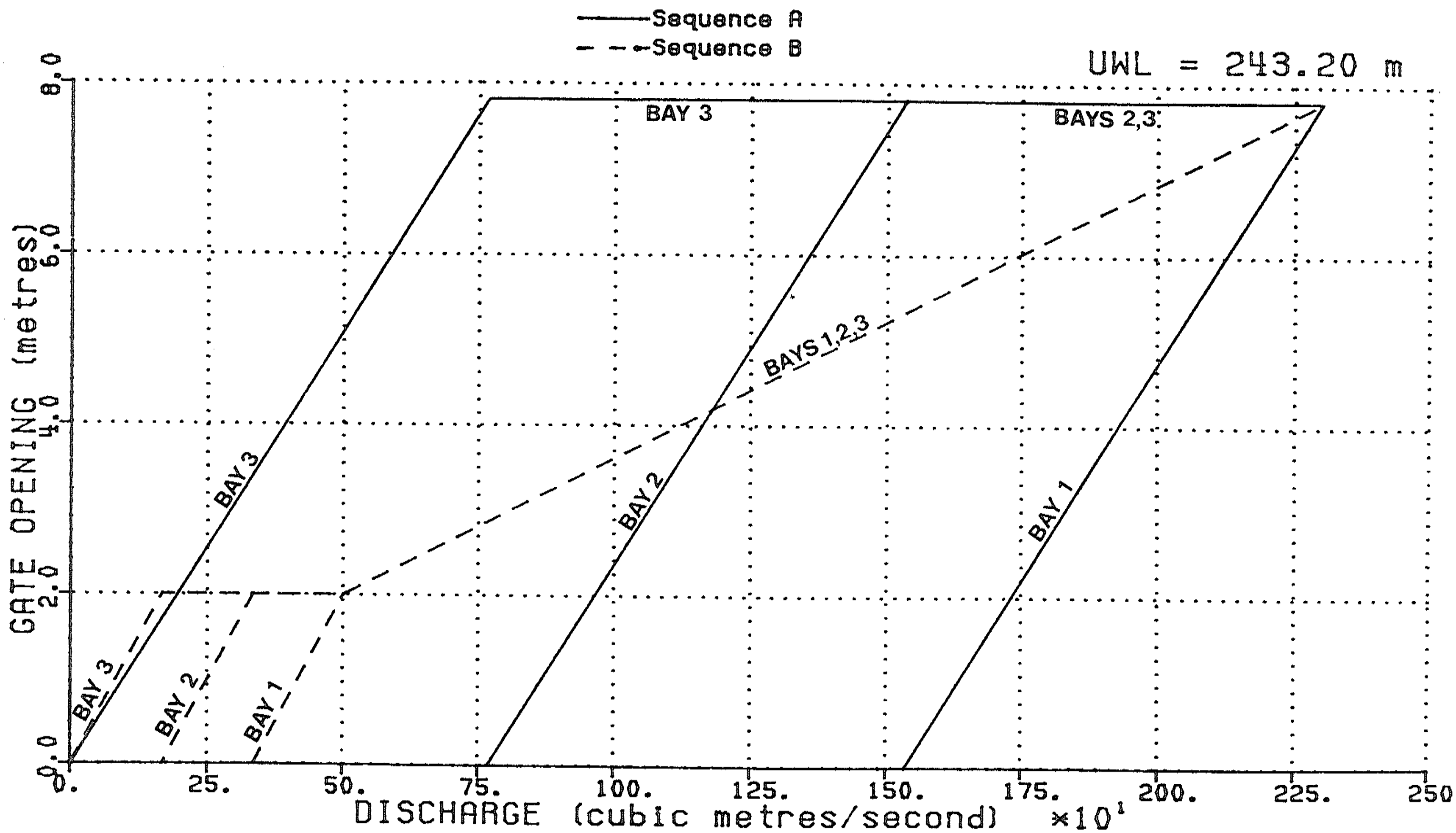

Figure 5.33 Investigated Gate Opening Sequences for the Orifice Spillway

Appendix C

Plates

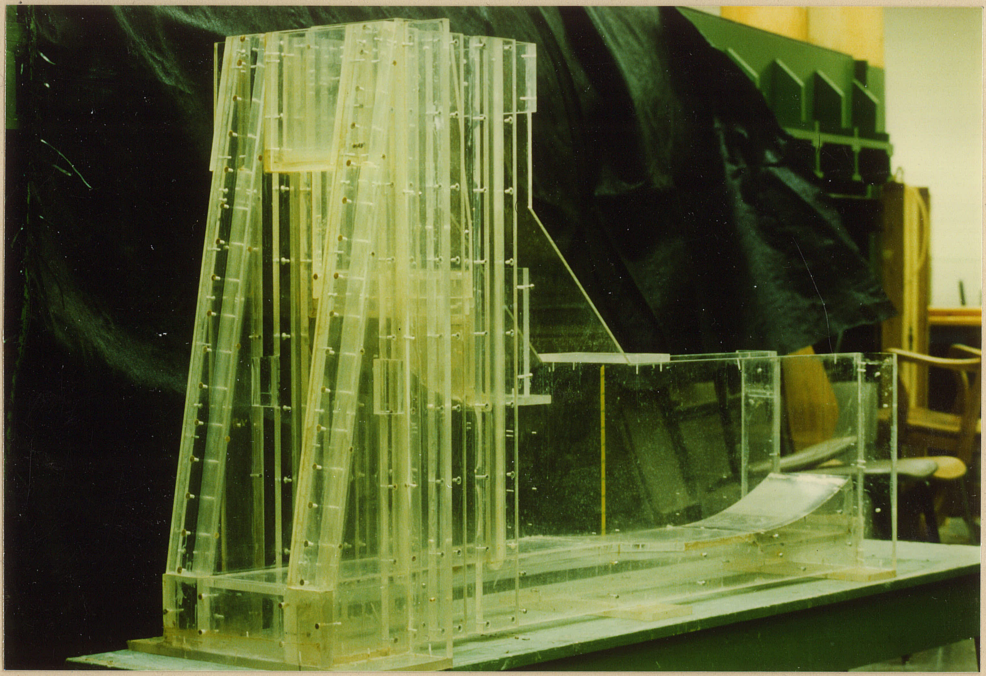

Plate 1   Orifice Spillway Flume Model

Plate 2   Overflow Spillway Flume Model - Diversion Configuration

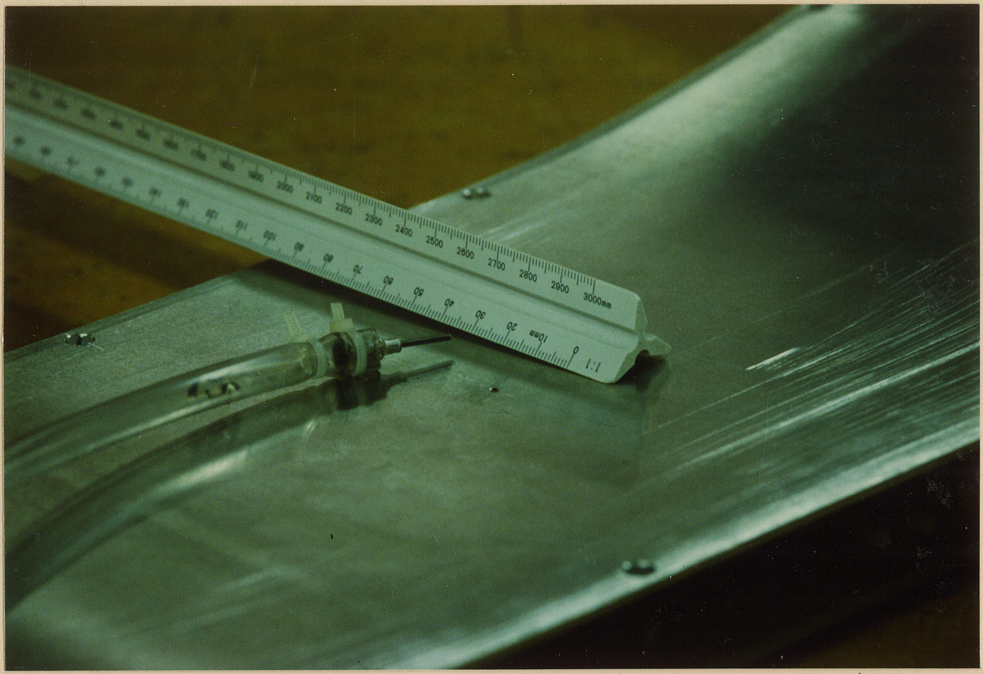

Plate 3   Piezometric Pressure Tap Apparatus and Rollway Installation

Plate 4   Comprehensive Site Model

Structures from left to right: main dam, south transition structure, spillway, trash sluice, central transition structure, and powerhouse.

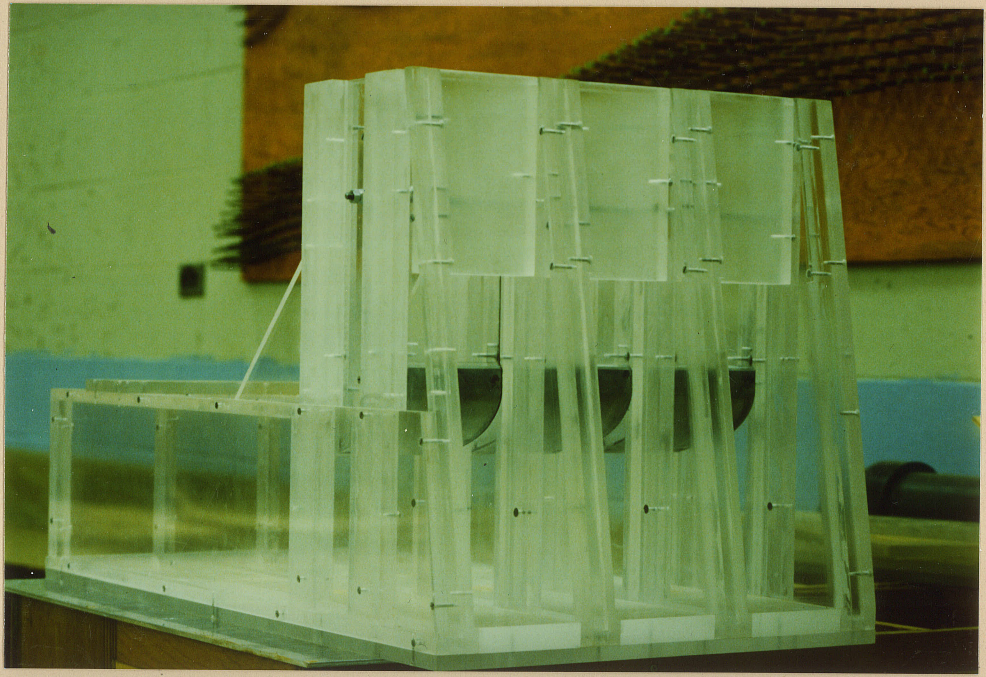

Plate 5   Orifice Spillway Structure for the Comprehensive Site Model - Diversion Configuration

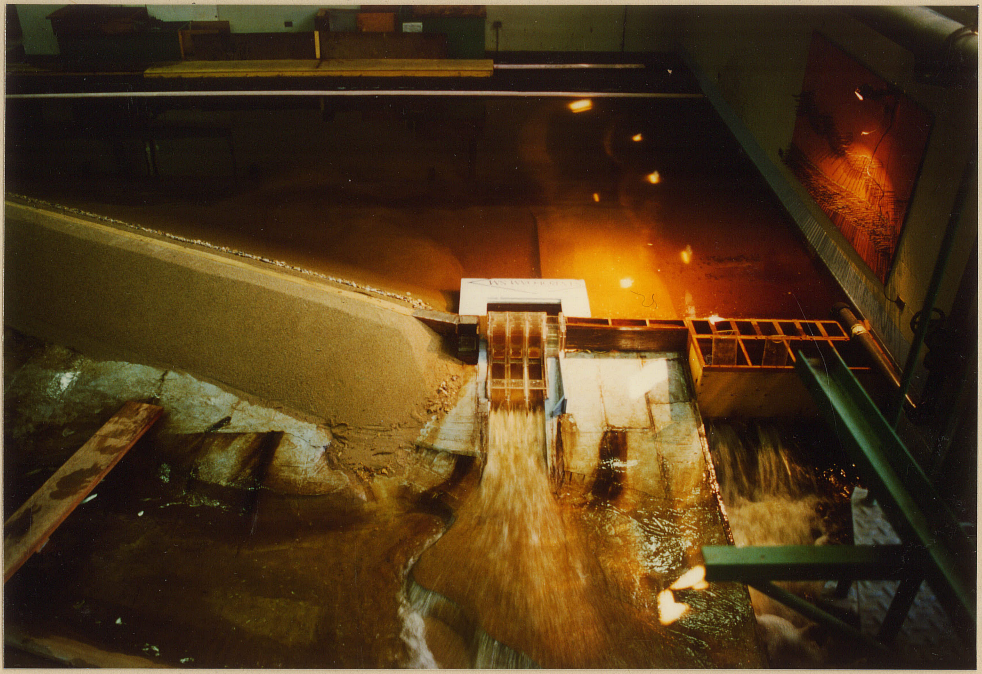

Plate 6 Comprehensive Model Ice Test Set-Up with Simulated Ice Cover Upstream of the Spillway

Plate 7 Dry Trash for the Comprehensive Model Trash Tests

Approximate prototype lengths: 9.6 metres, 6.4 metres, 4.5 metres

Plate 8 Orifice Spillway Flume Model Saturated Trash Tests - Before

Plate 9 Orifice Spillway Flume Model Saturated Trash Test - After

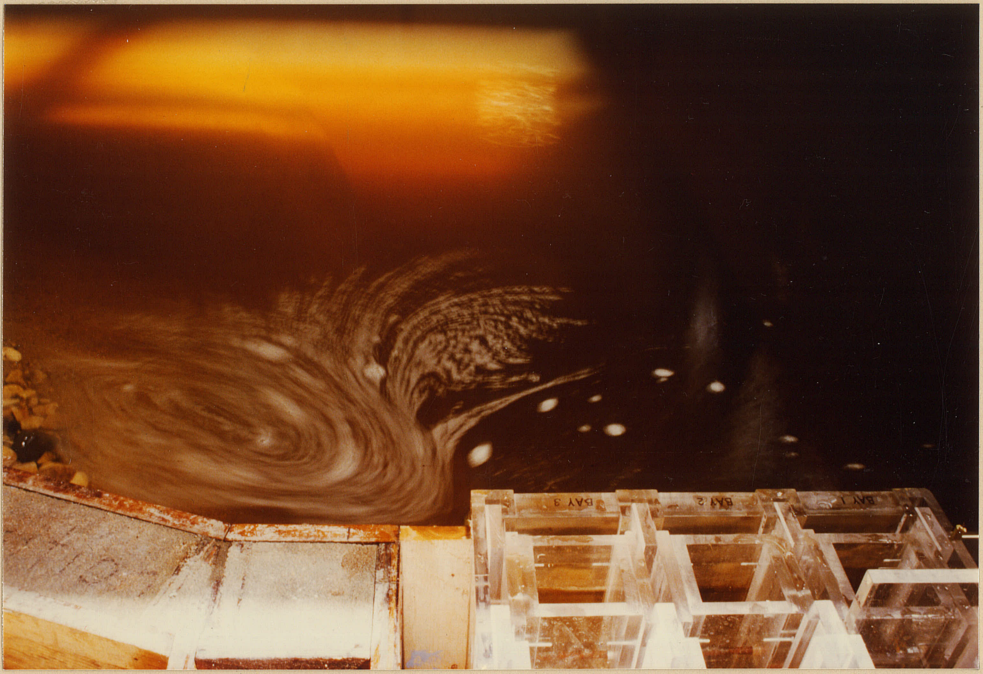

Plate 10 Clock-wise Rotational Flow Pattern in front of the South Transition Structure with Spillway Bays 2 & 3 Fully Open and Bay 1 Open 4.0 Metres.

Exposure Time = 1 second