PROVINCE OF MANITOBA
DEPARTMENT OF AGRICULTURE AND CONSERVATION
WATER CONTROL AND CONSERVATION BRANCH

Red River Floodway

REPORT

on

HYDRAULIC MODEL TESTS OF THE SUBMERSIBLE GATES AND THE INLET CONTROL STRUCTURE

H.G. ACRES & COMPANY LIMITED
Consulting Engineers
Niagara Falls, Canada

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1 - Summary

The results of hydraulic model tests carried out on the submersible gates and the inlet control structure for the Red River Floodway are presented in this report. Analysis of test results showed that:

(a) - Nonaerated submersible sector gates which are hinged at their upstream edges provide an efficient means of controlling the rate of diversion of flood waters into the floodway channel.

(b) - Two gates 112.5 feet wide are adequate to pass the maximum probable flood.

(c) - Passage of small craft through the structure will not be adversely affected except during abnormal summer or fall floods when velocities up to 10 feet per second may be experienced. During the navigation period when tailwater is maintained at or above elevation 734 by operation of St. Andrews control dam, a minimum draft of 6 feet will be available. There will be no restriction on height or width for small craft.

(d) - Normal operation of the gates results in a maximum uplift averaging 2.7 feet of water acting over the upstream skin plate. (This uplift pressure will be reversed to a downthrust of about 6 feet of water whenever the gates are unwatered during the summer navigation season, for maintenance.)

(e) - The downstream skin plate of the gates will be subject to a maximum local differential pressure of 24.1 feet of water and ±3.0 feet of water pressure due to turbulent fluctuations. The downstream skin plate will be subject to a maximum negative pressure of 8 feet of water. If the downstream skin plate is constructed as a continuous smooth surface, with local protuberances not exceeding 0.1-inch, cavitation is not anticipated.

(f) - A simple flip bucket downstream from the gate provides adequate energy dissipation.

(g) - The most economical structural arrangement of the upstream and downstream wing wall shapes tested provides adequate protection for the bordering dykes without loss of hydraulic efficiency.

(h) - Riprap protection is required upstream and downstream from the structure in the channel and on the slopes of the adjacent dykes.

The recommended arrangement of the structure and the requirements for riverbed and slope protection are shown on Plate 1. Operating curves and design information for the gates are presented on Plates 7, 8 and 9.

2 - Statement of the Problem

Excess flood flows from the Red River will be diverted into the floodway channel with control of the rate of diversion being maintained through operation of the inlet control structure located approximately 3,000 feet downstream from the inlet to the channel.

The confluence of the Assiniboine River and the Red River occurs 19 miles downstream from the control structure in the centre of the City of Winnipeg, as shown on Plate 2. As the time of the peak flood discharges from the Assiniboine River does not necessarily coincide with the flood peaks on the Red River, the discharge characteristics of the Red River at the inlet control structure vary over a considerable range due to backwater effects from the Assiniboine River.

The basic requirements affecting the design of the inlet control structure are:

(a) - The control works are to be designed to effect the desired diversion of river flow through the floodway channel while maintaining upstream water levels at those which would occur under natural conditions up to design flood level of 770.25. Alternative provisions for operation at flows exceeding the design flood are to be provided as specified under (2) and (3) below.

(b) - The discharge capacity of the structure should be sufficient to pass the 1,000-year flood with the floodway in operation and without causing an increase in upstream water levels over those which would have been obtained under natural conditions with a minimum contribution to the flood peak from the Assiniboine River.

(c) - The works should be capable of restricting the discharge passing downstream through the City of Winnipeg so that water levels at Redwood Bridge will not exceed elevation 755.5 under conditions of maximum Assiniboine River contribution up to the stage at which upstream water levels have been increased to elevation 778.05, and the discharge through the structure is 55,800 cfs.

(d) - Provision is to be made for passage of small boats during the summer navigation season during which time water levels through the city are controlled at elevation 734 by the dam at Lockport. The following dimensions have been specified as desirable minimum requirements: draft, 6 feet; width of channel, 15 feet; overhead clearance, 15 feet.

(e) - The range of conditions to be considered has been defined as those combinations of flood flows in the Red River and Assiniboine River which may be expected to occur 90 per cent of the time with all flood control works in operation. These works include the Shellmouth Reservoir and Portage Diversion, in addition to the Red River Floodway.

Plate 3 shows the discharge rating curves for the inlet control structure which satisfy the above basic requirements. From consideration of the foregoing requirements, preliminary studies led to the conclusion that submersible sector gates would afford the best means of control. Several of the important considerations influencing the choice and design of these gates were:

(a) - The Red River is one of the few rivers in Canada which flows from south to north, and it is not unusual for the spring breakup of river ice to occur in the headwaters before it does in the downstream area. As a result, the ice breakup can be very abrupt and large volumes of ice would have to be passed through the structure in a short period of time. It was considered, therefore, that submersible gates would be the most satisfactory means of providing flow regulation and avoiding any tendency to create ice jams upstream from the structure.

(b) - Because of the large range of flows likely to occur during the spring runoff period, submersible sector gates hinged at their upstream edge would greatly reduce the possibility of ice damage to the downstream skin plates.

(c) - An upstream skin plate is desirable to prevent debris from entering the gate chamber in the substructure when the gates are down, as they will be most of the time.

As the flow conditions which governed the operation of the inlet control structure are extremely complex, and, as design information for submersible sector gates hinged at their upstream edges is negligible, two hydraulic models were constructed and tested to evaluate performance and obtain design data.

A pilot model of the submersible sector gate was examined to determine:

(a) - Basic hydraulic information on discharge characteristics of the gate and the required shape of stilling bucket.

(b) - Preliminary structural design information for the gates so that modifications to the main model could be kept to a minimum.

A model of the control structure was then tested to determine:

(a) - The most economical and hydraulically efficient arrangement of the control structure.

(b) - Discharge rating curves for the gates.

(c) - Final hydraulic design information for the gates.

(d) - Flow conditions through the control structure at which discharges ice may be moving in the river.

(e) - Flow conditions through the control structure during the summer navigation season.

(f) - The extent of riprap required upstream and downstream from the control structure.

3 - Description of the Models

3.1 - Pilot Model of the Gate - The design conditions for the gates are:

(a) - The structure must be capable of passing the maximum probably flood of 141,000 cfs with a headwater elevation of 774.6, a tailwater elevation of 770.2, and with the sill of the structure at elevation 728.0.

(b) - The gates must be capable of restricting flow through the structure, under emergency operating conditions, to 55,800 cfs with a headwater elevation of 778.05 and a tailwater elevation of 762.70.

Analytical studies showed that two gates 112.5 feet wide would be required to pass the maximum probable flood. Although pier and wing wall contraction effects were not tested in the pilot model, the foregoing gate width was used to determine the test discharges per foot width. In addition, preliminary studies indicated that construction costs for a sector gate could be minimized if the contained angle were 60 degrees. To satisfy hydraulic requirements and the foregoing structural considerations, it was found that the gate should have a radius of 42 feet.

A section of a 42-foot radius gate was constructed to a scale of 1:60 in a one-foot wide flume. As shown on Plate 4, the modelled area represented a 60-foot wide section through the control structure extending 350 feet upstream and 500 feet downstream.

A plexiglass panel built into the side of the test flume facilitated the observation of the various flow conditions. Ten piezometers were positioned on the centreline of the plexiglass gate along the upstream and downstream skin plates and connected to individual stilling wells located on the outside of the flume. To measure pressure fluctuations along the downstream skin plate, pressure transducers inserted into piezometer tapping points were connected to a two-channel Sanborn Recorder having a signal response capacity of 60 cycles per second.

The moveable bed material was 3/4-inch stone. This size of rock was not chosen to represent a particular prototype material but because preliminary tests showed it produced stable scour patterns over the downstream length of stilling bucket. A stilling bucket developed from these studies was constructed of paraffin wax so that further minor modifications could be readily accomplished.

Water entered the model from a supply line which emptied into a headbox containing two sets of rock baffles to minimize surges. From the headbox, water was led to the structure through a bell-mouthed sheet metal channel to minimize entrance losses. After passing over the gate section, the water left the model over an overflow tailwater control gate and entered the sump for measurement and recirculation.

A point gauge railway serviced all areas of the model for a distance of 150 feet upstream and 180 feet downstream from the gate hinge.

3.2 - Model of the Control Structure - A model of the control structure, also constructed to a linear scale of 1:60, included one and one-quarter gates and a dyke section 540 feet long. The modelled area represented a reach of the Red River extending 400 feet upstream and 500 feet downstream from the control structure. This length was chosen to provide adequate overlap with the moveable bed model of the inlet works area, tested at the University of Manitoba, so that riprap requirements could be finalized[1].

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(1) - Report on Hydraulic Model Tests of the Inlet Control Works, H.G. Acres & Company Limited, January 1963.

As shown on Plate 5, the topography was modelled either as a moveable bed using sand or as a fixed bed using a cement mortar laid over a well-compacted sand base. The moveable bed sections were moulded using surface templates spaced at 120-foot intervals. The fixed bed areas were constructed using templates which accurately defined the ground surface. Flood plain roughness was simulated using 1-1/2-inch stone chips at 2-1/2-inch centres placed above ground level.

A plywood bulkhead constructed along the dyke axis eliminated leakage through the dyke section.

All sections of the inlet control structure, except the stilling bucket, were constructed of plexiglass. Fifteen piezometers were built into the full gate; ten were positioned on the upstream skin plate (five adjacent to the pier and five along the gate centreline), and five were located along the downstream skin plate centreline. All piezometer tubes were led outside the model to individual stilling wells.

The stilling bucket was constructed out of laminated wood and lacquered to prevent swelling.

Ice tests were carried out using pans of paraffin wax to simulate ice flows. The specific gravity of the wax was between 0.88 and 0.92, and pans were cast to represent ice floes which varied in size from 20 feet square and 2 feet thick to 100 feet by 50 feet and 2 feet thick.

Water entered the model from a supply line which emptied into a headbox containing two sets of rock baffles to minimize surges, flowed through the structure, and passed over a sectional overflow tailwater control gate before entering the sump for measurement and recirculation. To provide a discharge rating curve for one gate, water passing over the quarter-gate was led out of the model by a metal trough and deposited in the sump downstream from the measuring weirs. A point gauge railway serviced all major areas of the model.

The ratios for the transfer of model data to the prototype, by Froude similitude, are as follows:

Length	1:60	= 1:60
Volume	$1:60^3$	= 1:216,000
Discharge	$1:60^{5/2}$	= 1:27,890
Velocity	$1:60^{1/2}$	= 1:7.746
Time	$1:60^{1/2}$	= 1:7.746
Unit Pressure	1:60	= 1:60

4 - Test Program and Test Procedure

4.1 - Pilot Model of the Gate - Various gate arrangements were tested to provide the following information:

(a) - The gate height required to meet the design conditions of discharge and headwater and tailwater levels when the overflow is aerated and nonaerated.

(b) - The magnitude of the negative pressures on the downstream skin plate.

(c) - The magnitude and frequency of the pressure fluctuations on the downstream skin plate.

(d) - The magnitude of the differential pressures on the downstream skin plate.

(e) - The required shape of the stilling bucket.

The various gate arrangements tested are shown on Plate 6. The discharges used during the pilot model tests are listed in Table 1. For each of the above arrangements the following measurements were made:

(a) - Discharge through the control structure with headwater and tailwater levels adjusted in accordance with Plate 3.

(b) - Water surface levels along the centre-line of the model and the shape of the lower nappe, as established from dye traces.

(c) - Gate height and angle.

(d) - Upstream and downstream skin plate pressures.

(e) - Magnitude and frequency of the pressure fluctuations on the downstream skin plate.

(f) - Profiles of the eroded bed.

<u>4.2 - Model of the Control Structure</u> - Using the gate configuration developed in the pilot model, the main model was tested to evaluate:

(a) - The width of gates required to pass the maximum probable flood.

(b) - The effect of raising the upstream wing walls on the discharge through the structure.

(c) - The effect on downstream scour patterns caused by using downstream wing walls placed at 90 degrees and 45 degrees to the centreline of the structure.

(d) - The suitability of the stilling bucket shape, as determined from the pilot model.

(e) - The final hydraulic design data for the gates.

The various discharges used in the testing of the model of the control structure are listed in Table 2. For each of these tests the following measurements were made:

(a) - Discharge through the control structure with headwater and tailwater levels adjusted in accordance with Plate 3.

(b) - Water surface elevations.

(c) - Gate height.

(d) - Upstream and downstream piezometer levels.

(e) - Point velocity traverses, to define bottom velocities, using both a Bentzel tube and Pitot tube.

5 - Analysis of Results and Recommendations

A summary of all tests completed on the pilot model of the gate is contained in Table 3, and a summary of all tests completed on the model of the control structure is presented in

Table 4. For convenience, the test results are discussed hereunder in the several sections which, although interdependent, were subject to separate modification throughout the course of the test program.

5.1 - The Submersible Sector Gates

5.1.1 - Gate Arrangement - The requirement for passage of the maximum probable flood dictates the width of the gates for the control structure. Under this discharge condition, the gates would be fully down. For all other discharges, however, the gates would be raised sufficiently to effect the necessary diversion of flood waters into the floodway channel.

A number of gate arrangements, as shown on Plate 6, were tested to determine the arrangement which would provide a reliable and economic means of satisfying the complex operating requirements of the inlet control structure.

Gate arrangements Nos. 1 to 4 inclusive were designed to assess the effect of changes to the shape of the upstream skin plate on the required gate height. The pertinent test results are summarized in Table 5, and from these it is readily seen that arrangement No. 2 would permit a saving in gate height of approximately 0.5-foot. However, this saving in gate height would be offset by increased construction cost for the tip. Since arrangements Nos. 3 and 4 were not as efficient as arrangement No. 2, they were not considered further.

In all cases, aeration of the nappe resulted in a maximum saving in gate height of 1.0 foot. However, since aeration of the overflow nappe was possible for only a very limited range of flows and did not result in appreciable reduction in gate height, and therefore gate costs, compared to the cost of aeration facilities, further gate arrangements were tested for the nonaerated condition only.

Although arrangement No. 1 appeared to be acceptable, it was considered worthwhile to attempt to reduce the magnitude of the pressure fluctuations on the downstream skin plate before making a final selection. The maximum pressure fluctuations recorded on the downstream skin plate for arrangement No. 1 were 6 feet of water and were entirely random with no well-defined frequencies. While there was, therefore, no reason for concern about hydro-elastic vibrations, it was decided to investigate the possibility of preventing separation of the flow at the tip and attempt to maintain streamline flow along the major portion of the downstream skin plate with the gate in the raised position.

Arrangements Nos. 5 and 6 were designed to reduce pressure fluctuations on the downstream skin plate. Arrangement No. 5 was tested to assess the practicability of reducing pressure fluctuations by forcing the overflow nappe away from the downstream skin plate with high-pressure water jets. A 2-1/2-foot projection was designed to provide tangential flow of high-pressure water from inside the gate along the downstream skin plate. From the test results, shown on Table 5, it may be seen that this overhang aggravated the situation.

Arrangement No. 6 was designed with its upper surface conforming to the shape of the nonaerated nappe as determined using arrangement No. 1 and dye traces. Virtual streamline flow was achieved and the pressure fluctuations were reduced to a negligible magnitude. This arrangement was not considered acceptable, however, as even though it virtually eliminated pressure fluctuations on the downstream skin plate, the concave configuration of upstream skin plate would become a potential silt trap during those periods when the gates were fully down. With this wedge area completely filled with silt, the lifting capacity of the operating mechanism for the gates would be nearly double that otherwise required.

Arrangement No. 7 was designed to incorporate the crest shape of arrangement No. 6 but to eliminate the potential silt trap. Test results verified that the same advantages existed as for arrangement No. 6 with respect to the pressure fluctuations. Even though this arrangement appeared to meet all requirements, an electrical analogue was used to evaluate skin plate pressures in those areas of the gate not equipped with piezometer connections. Results of this study showed that negative pressures in the order of 24 feet of water would develop near the tip of the gate. Negative pressures of this magnitude would almost certainly result in local areas of cavitation in the prototype. Even though provision to offset the effects of cavitation could be designed for, sealing of the downstream skin plate would demand an extensive increase in the length of the downstream skin plate; that is, when the gate is fully down, the 3.5-foot radius tip would require the downstream seal to be positioned at elevation 724.5, whereas gates with a downstream skin plate shaped as a continuous arc could be sealed at elevation 728.0. The added cost of construction to cope with possible cavitation, in addition to an increase in length of the downstream skin plate of 7 feet, far outweighed any advantage there might be in reducing the downstream skin plate pressure fluctuations from 6 feet of water to 2 feet of water.

Before abandoning the possibility of reducing downstream pressure fluctuations, arrangement No. 1 was retested with a simple horizontal slot positioned near the crest. High-pressure water discharging from the slot resulted in a slight reduction in pressure fluctuations. However, a reduction in the magnitude of the pressure fluctuations of from 6 feet of water to 5 feet of water was not considered sufficient to warrant the cost of ductwork to lead the water from the upstream face, through the gate, to the downstream face.

From the above assessment of the seven gate arrangements tested, it is readily seen that the gate shape which best meets both hydraulic and economic considerations is arrangement No. 1, the basic 42-foot radius sector gate. Although the downstream skin plate in arrangement No. 1 will be subject to pressure fluctuations of 6 feet of water, structural means to cope with these provide a more economic solution than would be realized with a gate shaped to eliminate pressure fluctuations.

Photograph 1 illustrates the water surface profile and stilling action which will develop during periods of maximum restriction of flow through the structure with gate arrangement No. 1. Photograph 2 illustrates the design flood condition with average Assiniboine contribution.

Final hydraulic design data for the gates were obtained from the model of the control structure.

<u>5.1.2 - Gate Width</u> - Analytical studies had indicated that two gates 112.5 feet wide would be required to pass the maximum probable flood of 141,000 cfs. As shown on Plate 3,

headwater and tailwater levels for this discharge were fixed at elevation 774.6 and elevation 770.2 respectively, and the sill crest was set at elevation 728 by navigation requirements.

Test results showed that, with the gates fully down, the upstream water level was 1.2 feet below criteria. This in effect provides a factor of safety in the structure width of approximately 4 per cent. However, as this percentage is of the same order as that of the accuracy of the experimental test results, the gate width was considered acceptable without further modification.

The results of tests conducted using summer discharges showed that the structure will not constrict the water passage sufficiently to create navigational problems for even small pleasure craft. Surface velocities through the structure for river discharges of up to 5,000 cfs will not exceed 5 feet per second. In the case of a summer or fall flood of 12,000 cfs, velocities up to 10 feet per second will be experienced. However, as such floods are relatively infrequent and of short duration, it is anticipated that such velocities will be acceptable.

5.1.3 - Design Criteria - Plates 7, 8 and 9 illustrate respectively, the operating curves for the gates, the distribution of the differential pressures likely to develop along the upstream skin plate, and the distribution of the differential pressures likely to develop on the downstream skin plate as developed from the larger model. The maximum negative pressure recorded on the downstream skin plate during these final gate tests was 8 feet of water. Negative pressures of this magnitude are not sufficient to cause general cavitation in the prototype. Local cavitation is also not anticipated if the downstream skin plate is constructed as a relatively smooth continuous surface with local protuberances not exceeding 0.1-inch.

It will be noted from Table 5 and Plate 7 that the required gate height of 35.5 feet, as indicated by the tests of the pilot model, has been reduced to 34.8 feet. This change is due to upstream wing wall and pier contraction effects as evaluated in the main model.

Plate 10 illustrates typical water surface profiles through the inlet control structure. Photograph 3 is a general view of the model of the control structure and Photograph 4 illustrates the configuration of the water surface during the maximum probable flood.

<u>5.1.4 - Operating Requirements</u> - Tests were carried out in the model of the control structure for a range of flows covering the period when ice runs could be expected in the river. The modelled ice pans simulated the size and mass of the floes expected in the prototype but exceeded their flexural

strength by a factor of sixty. Results showed that all but the largest pans tested passed smoothly through the structure. The largest pans showed a tendency to arch from the gate crest to the bucket. This condition could exist only momentarily in the prototype as the ice would immediately break up upon striking the bucket. However, should this or any unforeseen condition cause a serious overload, relief valves in the operating mechanism would open and automatically allow the gates to lower, thereby relieving the excess loading. As soon as the ice cleared, the gates would return to their regulated position.

The only damage which could result to the gates would occur at the gate crests due to thicker ice pans grounding. Although it is very unlikely that these conditions would develop in the prototype, the possibility could be eliminated by operating the gates, during periods that ice is in the river, so that the differential between headwater and tailwater levels does not exceed 8 feet. This provision could readily be incorporated in the operating rule curves for the gates, as ice runs are not anticipated at discharges in excess of 77,000 cfs, at which stage the flow through the control structure would be 39,000 cfs with a water differential of 14.0 feet, the small variation required would not appear to be unacceptable. Nevertheless, it is recommended that the gate crest and the stilling bucket be reinforced to take into account possible impact loads.

<u>5.2 - The Stilling Bucket</u> - Due to the relatively high tailwater levels and the extreme range of flows encountered at the control structure a flip bucket is considered to be the most suitable means of ensuring the effective dissipation of destructive kinetic energy released by the falling water.

Plate 11 shows the results of scour tests from the pilot model of the gate and the chosen bucket shape. It will be noted that certain combinations of discharge and tailwater level result in the overflow jet plunging at such an angle that it misses the stilling bucket. For the very limited range of discharges at which the jet will plunge beyond the bucket, tests have shown that adequate protection can be provided by riprap more economically than through extension of the bucket.

<u>5.3 - The Upstream Wing Walls</u> - Only two arrangements of the upstream wing walls were tested. These were:

(a) - With the crest of the wing walls raised to elevation 771.0 and thereby protecting the dykes in the immediate area of the structure for all flow conditions up to the design flood level of 770.25.

(b) - With the crest of the wing walls raised to elevation 779.0 and thereby protecting the dykes in the immediate area of the structure for all headpond elevations.

Test results showed that the hydraulic performance of the structure was not improved by raising the top of the wing walls. Since the upstream dyke slopes will require riprap protection to guard against the destructive force of wave action, the additional cost was not warranted.

5.4 - The Downstream Wing Walls - Tests were performed on two configurations of the downstream wing walls as follows:

(a) - With the wing walls aligned at right angles to the structure centreline and raised to elevation 770.0 thereby protecting the dykes in the immediate area of the structure for all flow conditions.

(b) - With the wing walls aligned at 45 degrees to the structure centreline and sloping down at 1:1 from elevation 770.0

Dye traces and point velocity readings completed in the area of the downstream wing walls showed that neither arrangement affected the shape of the high-velocity jet leaving the stilling bucket nor the velocities in the immediate area of the structure to such a degree as to permit a choice to be made on the basis of hydraulic efficiency. Since wing walls aligned at right angles to the structure centreline are more economical, they are therefore recommended as shown on the final arrangement of Plate 1.

5.5 - The Riprap Protection - Plates 12 and 13 show the velocity distributions recorded at various sections of the Red River upstream and downstream from the control structure respectively.

Upstream riprap requirements are dictated by the necessity for passing of the maximum probable flood with the gates fully down. For all other discharge conditions the gates are raised and bottom velocities are greatly reduced. The suggested riprap sizes, as determined from these traverses, are listed in Table 6, and shown on Plate 1.

Downstream riprap protection is dictated neither by the maximum probable flood conditions nor the design flood conditions, but from other combinations of discharge and water level defined by the envelope curve shown on Plate 13. The recommended riprap sizes are listed in Table 7 and shown on Plate 1.

It may be readily seen by a comparison of Plate 1 of this report with Plate 1 of the model report on the inlet control works model(1), that the above riprap recommendations differ somewhat from those previously suggested. The recommendations made herein, however, result from a more detailed investigation of velocity distribution.

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(1) Op. Cit., Page 8

Protection of the riverbed and banks for a distance of 800 feet downstream from the control structure will be required to ensure the safety of the works under all discharge conditions.

Although not required to ensure the safety of the works, riprap laid from elevation 730 on the east bank to elevation 755 on the west bank for a distance of from 800 feet to 2,300 feet downstream from the structure would be required for protection against scouring under conditions of the maximum probable flood. This additional riprap requirement on the west bank is due to the fact that, while the riverbed veers gradually to the east, the high-velocity jet from the structure continues straight downstream and impinges against the west bank. In view of the infrequent occurrence of such an unusual flood, the riprap protection of the west bank downstream to the Sale River does not appear warranted.

The riprap zones shown on Plate 1 result from an analysis of all test results, and therefore represent the theoretical limits of each size. It should be noted, however, that practical considerations may dictate some adjustment in the final divisions specified for the prototype to facilitate construction.

Riprap is also recommended for the upstream dyke slopes to protect against wave action. These requirements were not developed from the model tests, but are the result of analytical studies of potential wave action which could develop on the impounded reservoir during emergency operation of the gates.

LIST OF TABLES

TABLE 1

DISCHARGES USED IN TESTING OF THE PILOT MODEL OF THE GATE

Control Structure Discharge (cfs)	Headwater Level	Tailwater Level	Assiniboine Contribution	Operating Conditions
32,000	766.20	748.65	Maximum	Normal
37,000	767.10	750.60	"	"
40,000	767.60	751.90	"	"
45,000	768.30	754.10	"	"
50,000	768.90	756.25	"	"
55,000	769.45	758.80	"	"
71,000	770.85	763.85	"	"
90,000	772.25	767.60	"	"
55,800	778.05	762.70	Maximum	Emergency
57,000	778.05	762.75	"	"
65,000	778.05	763.60	"	"
35,000	757.70	749.45	Average	Normal
40,000	765.70	751.40	"	"
45,000	767.35	753.30	"	"
50,000	768.20	755.05	"	"
70,000	770.15	761.35	"	"
90,000	771.65	766.30	"	"
120,000	773.80	769.40	"	"
40,000	754.65	751.25	Minimum	Normal
45,000	763.00	753.15	"	"
50,000	766.85	754.85	"	"
60,000	768.60	758.10	"	"
90,000	771.10	765.45	"	"
141,000	774.60	770.20	"	"

TABLE 2

DISCHARGES USED IN TESTING OF THE MODEL OF THE CONTROL STRUCTURE

Control Structure Discharge (cfs)	Headwater Level	Tailwater Level	Assiniboine Contribution	Operating Conditions	Remarks
25,500	750.00	744.80	Maximum	Normal	
30,000	761.70	747.45	"	"	
32,500	766.20	748.65	"	"	
39,000	767.40	751.50	"	"	
40,000	767.60	751.90	"	"	
45,000	768.30	754.05	"	"	
55,000	769.45	758.80	"	"	
63,500	770.25	761.55	"	"	Design
71,000	770.85	763.85	"	"	
82,400	771.70	766.70	"	"	
100,000	772.95	768.60	"	"	
120,000	774.55	770.15	"	"	
65,000	773.80	763.20	Maximum	Emergency	
60,000	776.15	762.85	"	"	
55,800	778.05	762.70	"	"	Determines gate height
65,000	778.05	763.60	"	"	
80,000	778.05	767.40	"	"	
100,000	778.05	769.25	"	"	
32,500	752.10	748.40	Average	Normal	
40,000	765.70	751.40	"	"	
45,000	767.40	753.30	"	"	
55,000	768.85	756.70	"	"	
70,700	770.25	761.60	"	"	Design
81,200	771.00	764.75	"	"	
83,000	771.15	765.10	"	"	
100,000	772.40	767.60	"	"	
120,000	773.80	769.40	"	"	
75,000	775.10	764.15	Average	Emergency	
69,000	778.05	763.60	"	"	
80,000	778.05	765.75	"	"	
100,000	778.05	768.30	"	"	
120,000	778.05	769.80	"	"	

Table 2 - 2

Control Structure Discharge (cfs)	Headwater Level	Tailwater Level	Assiniboine Contribution	Operating Conditions	Remarks
35,200	750.00	749.15	Minimum	Normal	
40,000	754.65	751.25	"	"	
45,000	762.65	753.10	"	"	
47,500	765.50	754.00	"	"	
65,000	769.10	759.50	"	"	
77,600	770.25	762.70	"	"	Design
82,400	770.60	763.75	"	"	
90,000	771.10	765.40	"	"	
96,500	771.55	766.35	"	"	
100,000	771.80	766.80	"	"	
120,000	773.20	768.75	"	"	
141,000	774.60	770.25	"	"	Determines structure width
82,400	778.05	765.05	Minimum	Emergency	
100,000	778.05	767.50	"	"	

TABLE 3

SUMMARY OF TESTS COMPLETED ON THE PILOT MODEL OF THE GATE

TEST NO.	DISCHARGE THROUGH THE CONTROL STRUCTURE (cfs)	HEADWATER LEVEL (ELEVATION)	TAILWATER LEVEL (ELEVATION)	ASSINIBOINE CONTRIBUTION	OPERATING CONDITION	GATE HEIGHT (ELEVATION)	GATE ARRANGEMENT	REMARKS
1	35,000	757.80	749.40	AVERAGE	NORMAL	746.7	NO. 1	NON-AERATED NAPPE
2	40,000	765.70	751.40	"	"	753.5	"	AERATED NAPPE
3	50,000	768.20	755.00	"	"	754.2	"	NON-AERATED NAPPE
4	70,000	770.20	761.40	"	"	751.8	"	NON-AERATED NAPPE
5	90,000	771.65	766.30	"	"	748.2	"	NON-AERATED NAPPE
6	120,000	773.30	769.40	"	"	745.7	"	NON-AERATED NAPPE
7	32,000	765.60	748.40	MAXIMUM	"	756.4	"	NON-AERATED NAPPE
8	57,000	777.40	762.75	"	EMERGENCY	762.9	"	NON-AERATED NAPPE
9	90,000	772.30	767.60	"	NORMAL	740.0	"	NON-AERATED NAPPE
10	40,000	754.65	751.25	MINIMUM	"	738.5	"	NON-AERATED NAPPE
11	60,000	768.60	758.10	"	"	752.1	"	NON-AERATED NAPPE
12	90,000	771.10	765.45	"	"	747.8	"	NON-AERATED NAPPE
13	141,000	774.60	770.20	"	"	743.9	"	NON-AERATED NAPPE
14	40,000	767.60	751.90	MAXIMUM	"	755.3	"	AERATED NAPPE
15	40,000	767.60	751.90	"	"	756.9	"	NON-AERATED NAPPE
16	50,000	768.90	756.20	"	"	755.3	"	NON-AERATED NAPPE
17	50,000	766.85	754.85	MINIMUM	"	752.7	"	NON-AERATED NAPPE
18	45,000	763.00	753.10	"	"	743.6	"	NON-AERATED NAPPE
19	45,000	767.40	753.30	AVERAGE	"	754.6	"	NON-AERATED NAPPE
20	45,000	767.40	753.30	"	"	754.2	"	AERATED NAPPE
21	45,000	768.30	754.05	MAXIMUM	"	755.8	"	NON-AERATED NAPPE
22	45,000	768.30	754.05	"	"	755.4	"	AERATED NAPPE
23	32,000	765.60	748.40	"	"	755.8	"	AERATED NAPPE
24	32,750	766.20	748.60	"	"	756.3	NO. 2	NON-AERATED NAPPE
25	40,000	767.60	751.90	"	"	756.3	"	NON-AERATED NAPPE
26	45,000	768.30	754.05	"	"	755.25	"	NON-AERATED NAPPE
27	55,800	778.05	762.70	"	EMERGENCY	763.1	"	NON-AERATED NAPPE
28	90,000	772.30	767.60	"	NORMAL	746.9	"	NON-AERATED NAPPE
29	32,750	766.20	748.60	"	"	755.4	"	AERATED NAPPE
30	40,000	767.60	751.90	"	"	755.4	"	AERATED NAPPE
31	40,000	767.60	751.90	"	"	756.6	NO. 3	NON-AERATED NAPPE
32	40,000	767.60	751.90	"	"	755.5	"	AERATED NAPPE
33	37,000	767.15	750.60	"	"	756.6	"	NON-AERATED NAPPE
34	45,000	768.30	754.05	"	"	755.3	"	NON-AERATED NAPPE
35	40,000	767.60	751.90	"	"	756.7	NO. 4	NON-AERATED NAPPE
36	57,200	777.40	763.75	"	EMERGENCY	761.6	NO. 5	NON-AERATED NAPPE
37	71,000	770.80	763.80	"	NORMAL	750.3	NO. 6	NON-AERATED NAPPE
38	40,000	767.60	751.90	"	"	755.5	"	NON-AERATED NAPPE
39	55,000	769.45	758.30	"	"	753.5	"	NON-AERATED NAPPE
40	65,000	773.80	763.20	"	EMERGENCY	756.1	"	NON-AERATED NAPPE
41	55,800	778.05	762.70	"	"	763.35	"	NON-AERATED NAPPE
42	57,000	777.40	763.75	"	"	762.2	"	NON-AERATED NAPPE
43	55,800	778.05	762.70	"	"	763.7	NO. 7	NON-AERATED NAPPE
44	57,000	777.40	763.75	"	"	762.9	"	NON-AERATED NAPPE
45	55,000	769.45	758.80	"	NORMAL	755.0	"	NON-AERATED NAPPE
46	40,000	767.60	751.90	"	"	756.6	"	NON-AERATED NAPPE
47	55,800	778.05	762.70	"	EMERGENCY	763.0	NO. 1	WATER SLOT NEAR GATE CREST
48	40,000	767.60	751.90	"	NORMAL	756.0	"	WATER SLOT NEAR GATE CREST
49	57,000	777.40	763.75	"	EMERGENCY	762.5	"	NON-AERATED NAPPE
50	55,800	778.05	762.70	"	"	763.15	"	NON-AERATED NAPPE
51	65,000	773.80	763.20	"	"	755.9	"	NON-AERATED NAPPE
52	55,000	769.45	758.80	"	NORMAL	754.0	"	NON-AERATED NAPPE
53	40,000	767.60	751.90	"	"	756.35	"	NON-AERATED NAPPE
54	71,000	770.80	763.80	"	"	751.3	"	NON-AERATED NAPPE

TABLE 4

SUMMARY OF TESTS COMPLETED ON THE MODEL OF THE CONTROL STRUCTURE

TEST NO.	DISCHARGE THROUGH THE CONTROL STRUCTURE (cfs)	HEADWATER LEVEL (ELEVATION)	TAILWATER LEVEL (ELEVATION)	ASSINIBOINE CONTRIBUTION	OPERATING CONDITION	CREST OF U/S WING WALLS (ELEVATION)	D/S WING WALL ARRANGEMENT (ELEVATION)	GATE HEIGHT (ELEVATION)	REMARKS
1	141,000	774.60	770.20	MINIMUM	NORMAL	771.00	770.0 AT 90°	737.3	MAXIMUM PROBABLE FLOOD.
2	141,000	773.40	"	"	"	"	"	728.0	ASSESSMENT OF U/S
3	141,000	774.60	"	"	"	779.0	"	737.3	WING WALL SHAPE AND
4	141,000	773.40	"	"	"	"	"	728.0	REQUIRED STRUCTURE WIDTH.
5	77,600	770.25	762.70	"	"	771.0	"	748.2	ASSESSMENT OF D/S
6	141,000	774.60	770.20	"	"	"	770.0 AT 45° SLOPED AT 1:1	737.3	WING WALL ARRANGEMENT.
7	77,600	770.25	762.70	"	"	"	"	748.1	
8	32,500	766.20	743.65	MAXIMUM	"	"	770.0 AT 90°	755.8	MAXIMUM DIFFERENTIAL
9	55,800	778.05	762.70	"	EMERGENCY	"	"	762.8	MAXIMUM GATE HEIGHT.
10	65,000	769.10	759.50	MINIMUM	NORMAL	"	"	750.3	ASSESSMENT OF STILLING
11	90,000	771.10	765.40	"	EMERGENCY	"	"	745.5	BUCKET SHAPE.
12	5,000	734.30	734.00			"	"	728.0	ASSESSMENT OF STRUCTURE
13	7,500	734.60	"			"	"	"	WIDTH ON SUMMER FLOWS.
14	10,000	735.00	"			"	"	"	
15	12,500	735.60	"			"	"	"	
16	25,500	750.00	744.80	MAXIMUM	NORMAL	"	"	739.8	DEVELOPMENT OF OPERATING
17	30,000	761.70	747.45	"	"	"	"	751.7	CURVES FOR THE GATES
18	39,000	767.40	751.50	"	"	"	"	755.1	AND FINAL HYDRAULIC
19	40,000	767.60	751.90	"	"	"	"	755.0	DESIGN CRITERIA.
20	45,000	768.30	754.05	"	"	"	"	754.6	
21	55,000	769.45	753.30	"	"	"	"	753.1	
22	63,500	770.25	761.55	"	"	"	"	751.0	
23	71,000	770.35	763.85	"	"	"	"	749.8	
24	82,400	771.70	766.70	"	"	"	"	746.0	
25	100,000	772.95	768.60	"	"	"	"	742.6	
26	120,000	774.55	770.15	"	"	"	"	740.3	
27	65,000	773.80	763.20	"	EMERGENCY	"	"	754.7	
28	60,000	776.15	762.85	"	"	"	"	758.9	
29	65,000	778.05	763.60	"	"	"	"	759.8	
30	80,000	778.05	767.40	"	"	"	"	756.2	
31	100,000	778.05	769.25	"	"	"	"	752.0	
32	32,500	752.10	748.40	AVERAGE	NORMAL	"	"	739.5	
33	40,000	765.70	751.40	"	"	"	"	752.9	
34	45,000	767.40	753.30	"	"	"	"	753.6	
35	55,000	768.85	756.70	"	"	"	"	752.5	
36	70,700	770.25	761.60	"	"	"	"	750.0	
37	81,200	771.00	764.75	"	"	"	"	747.1	
38	83,000	771.15	765.10	"	"	"	"	746.3	
39	100,000	772.40	767.60	"	"	"	"	743.0	
40	120,000	773.80	769.40	"	"	"	"	740.3	
41	75,000	775.10	764.15	"	EMERGENCY	"	"	754.5	
42	69,000	778.05	763.60	"	"	"	"	758.7	
43	80,000	778.05	765.75	"	"	"	"	756.1	
44	100,000	778.05	768.30	"	"	"	"	752.0	
45	120,000	778.05	769.80	"	"	"	"	748.2	
46	35,200	750.00	749.15	MINIMUM	NORMAL	"	"	730.4	

TABLE 4

SUMMARY OF TESTS COMPLETED ON THE MODEL OF THE CONTROL STRUCTURE

TEST NO.	DISCHARGE THROUGH THE CONTROL STRUCTURE (cfs)	HEADWATER LEVEL (ELEVATION)	TAILWATER LEVEL (ELEVATION)	ASSINIBOINE CONTRIBUTION	OPERATING CONDITION	CREST OF U/S WING WALLS (ELEVATION)	D/S WING WALL ARRANGEMENT (ELEVATION)	GATE HEIGHT (ELEVATION)	REMARKS
47	40,000	754.65	751.25	MINIMUM	NORMAL	771.00	770.0 AT 90°	733.5	DEVELOPMENT OF OPERATING CURVES FOR THE GATES
48	45,000	762.65	753.10	"	"	"	"	748.2	
49	47,500	765.50	754.00	"	"	"	"	751.0	
50	82,400	770.60	763.75	"	"	"	"	747.2	
51	96,500	771.55	766.35	"	"	"	"	743.5	
52	100,000	771.80	766.80	"	"	"	"	742.5	
53	120,000	773.20	768.75	"	"	"	"	740.25	
54	32,400	778.05	765.05	"	EMERGENCY	"	"	756.2	
55	100,000	778.05	767.50	"	"	"	"	752.2	
56	35,600	758.80	749.70	AVERAGE	NORMAL	"	"	747.0	ASSESSMENT OF ICE MOVEMENT THROUGH THE STRUCTURE. TEST 32 ALSO.
57	39,000	765.00	751.00	"	"	"	"	752.5	

TABLE 5

SUMMARY OF TEST RESULTS ON THE PILOT MODEL OF THE GATE

Gate Arrangement	Maximum Gate Height: Normal Operation (El)	Maximum Gate Height: Emergency Operation (El)	Maximum Negative Pressure (Ft of Water)	Maximum Pressure Fluctuation (Ft of Water)	Remarks
No. 1	756.4	763.3	8.3	±3.0	Nonaerated nappe
	755.7	762.4			Aerated nappe
No.1 with water slot near crest		763.3	7.5	±2.5	
No. 2	755.9				Nonaerated nappe
	755.0				Aerated nappe
No. 3	756.1				Nonaerated nappe
	755.1				Aerated nappe
No. 4	756.3				Nonaerated nappe
No. 5		763.4		±4.0	Water slot open
		763.2		±4.0	Water slot closed
No. 6	756.5	763.8	24.0	±1.0	Nonaerated nappe
		763.7	24.0	±1.0	Airfoil shape overhung 1 ft to form water slot
No. 7	756.5	763.7	24.0	±1.0	Nonaerated nappe

TABLE 6

RIPRAP REQUIREMENTS UPSTREAM FROM THE CONTROL STRUCTURE

Riprap Size	Location
30-inch cubes	On the upstream dyke slope from the training walls for a distance of 200 feet.
24-inch cubes	On the upstream dyke slope from 200 feet to 325 feet from the training walls.
18-inch cubes	On the upstream dyke slope from 325 feet from the training walls to the limits of the control structure contract.
	To elevation 725 from 0 feet to 35 feet upstream from the approach apron.
	On the wrap around dyke within the above limits from elevation 725 to elevation 770.
9-inch cubes	To elevation 725 from 35 feet to 70 feet upstream from the approach apron.
	On all sections of the wrap around dyke exclusive of that protected with 18-inch cubes.
6-inch cubes	To elevation 725 from 70 feet to 170 feet upstream from the approach apron.
	On the riverbanks within the above limits from elevation 725 to elevation 740.

TABLE 7

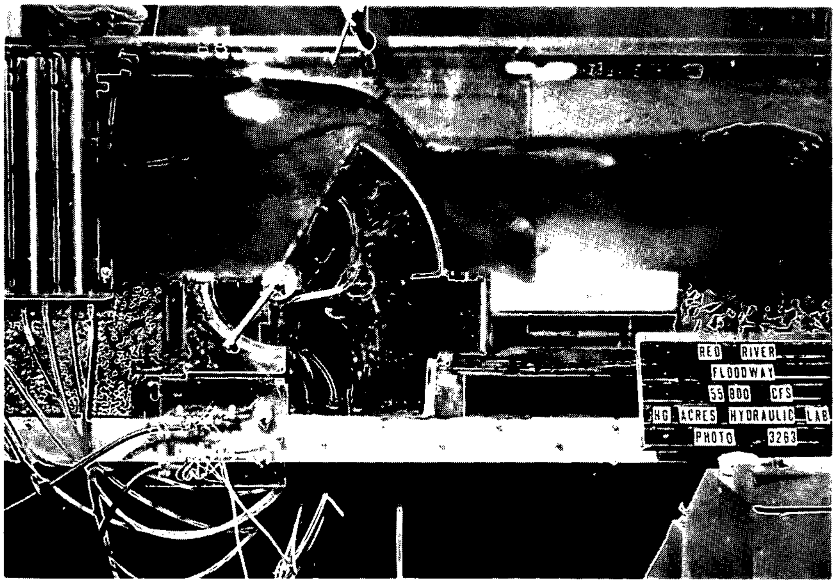
RIPRAP REQUIREMENTS DOWNSTREAM FROM THE CONTROL STRUCTURE

Riprap Size	Location
30-inch cubes	To elevation 725 from 0 feet to 80 feet downstream from the end of the stilling bucket.
18-inch cubes	To elevation 725 from 80 feet to 125 feet downstream from the stilling bucket.
	On the riverbanks to the limits of the wing walls from elevation 770 on the downstream dyke slope to 125 feet downstream from the bucket.
12-inch cubes	To elevation 725 from 125 feet to 400 feet downstream from the stilling bucket.
9-inch cubes	To elevation 725 from 400 feet to 800 feet downstream from the stilling bucket.
	From elevation 725 to elevation 730 from 125 feet to 400 feet downstream from the stilling bucket.
6-inch cubes	On the downstream dyke slope from the end of the wing walls for a distance of 50 feet to elevation 770.
	From elevation 730 to elevation 740 for a distance of 800 feet downstream from the stilling bucket.
	From elevation 725 to elevation 730 from 400 feet to 800 feet downstream from the stilling bucket.

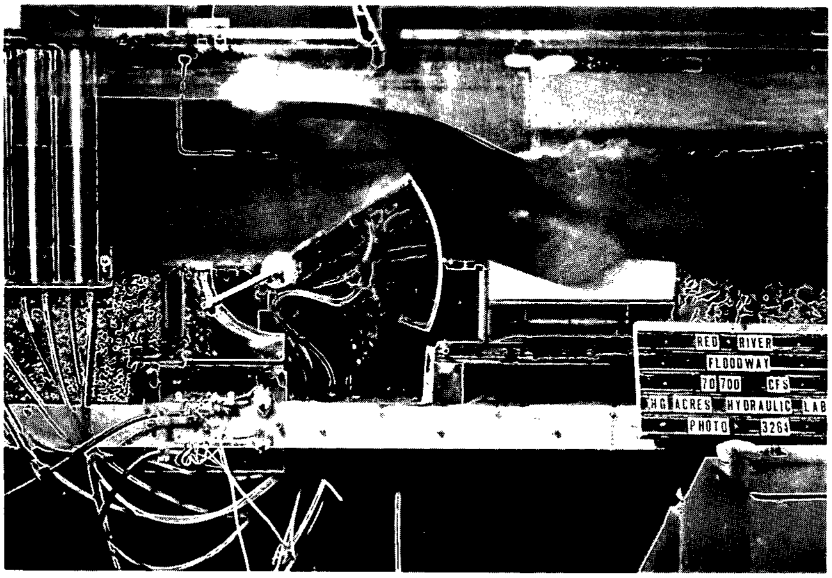
As determined from the inlet control works model tested at the University of Manitoba, 6-inch cubes will also be required from elevation 730 on the east bank to elevation 755 on the west bank from 800 feet to 2300 feet downstream from the stilling bucket if protection is required for the maximum probable flood condition.

LIST OF PHOTOGRAPHS

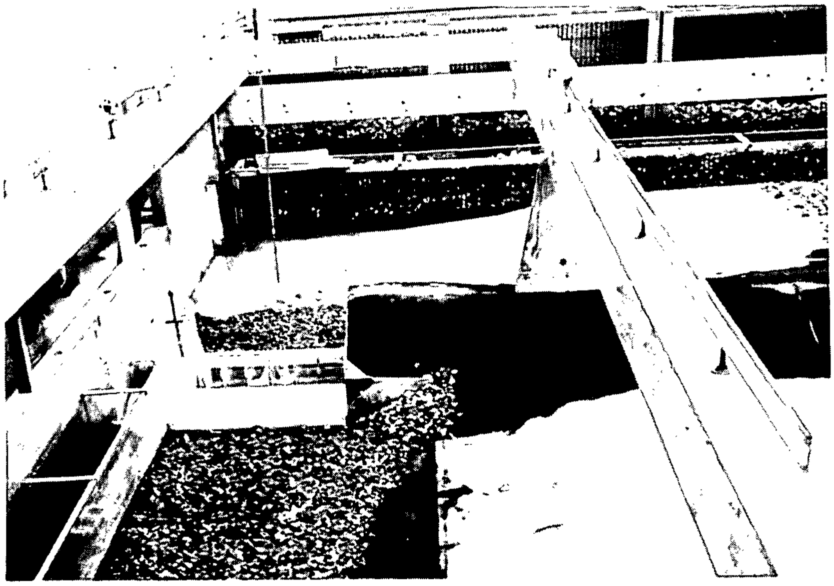
Number	Description
1	Final gate and stilling bucket arrangement. Emergency operation with maximum gate height. Control structure discharge = 55,800 cfs.
2	Final gate and stilling bucket arrangement. Normal operation at design flood. Control structure discharge = 70,700 cfs.
3	General view of the model of the control structure looking upstream.
4	Maximum probable flood discharging through the control structure looking downstream.



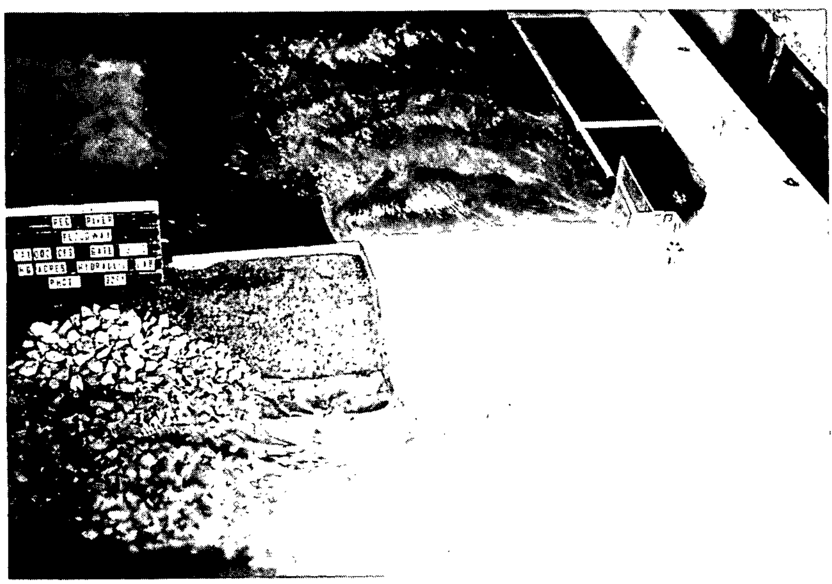
PHOTOGRAPH No. 1 — FINAL GATE AND STILLING BUCKET ARRANGEMENT EMERGENCY OPERATION WITH MAXIMUM GATE HEIGHT. CONTROL STRUCTURE DISCHARGE = 55,800 C.F.S.



PHOTOGRAPH No. 2 — FINAL GATE AND STILLING BUCKET ARRANGEMENT NORMAL OPERATION AT DESIGN FLOOD. CONTROL STRUCTURE DISCHARGE = 70,700 C.F.S.

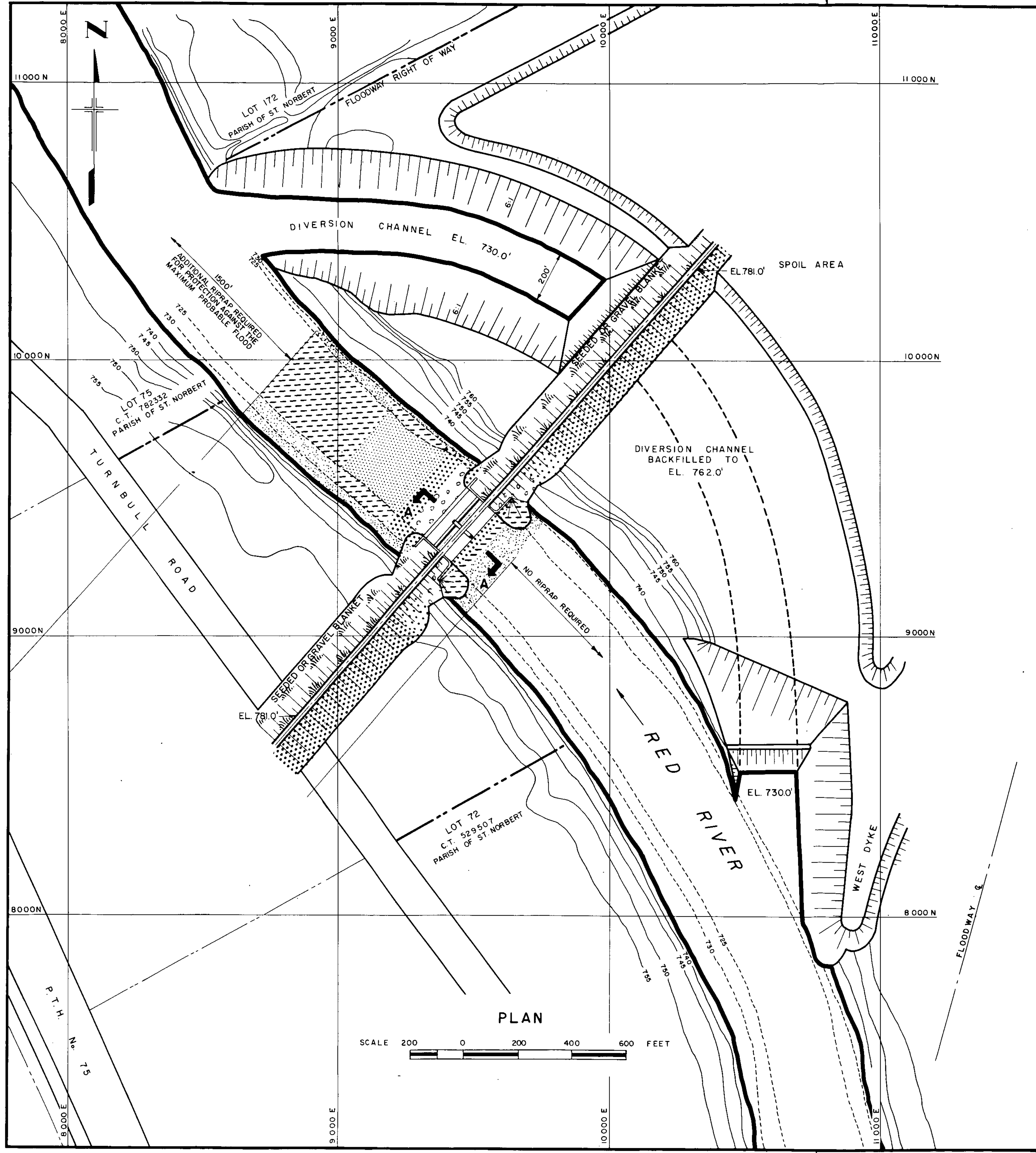


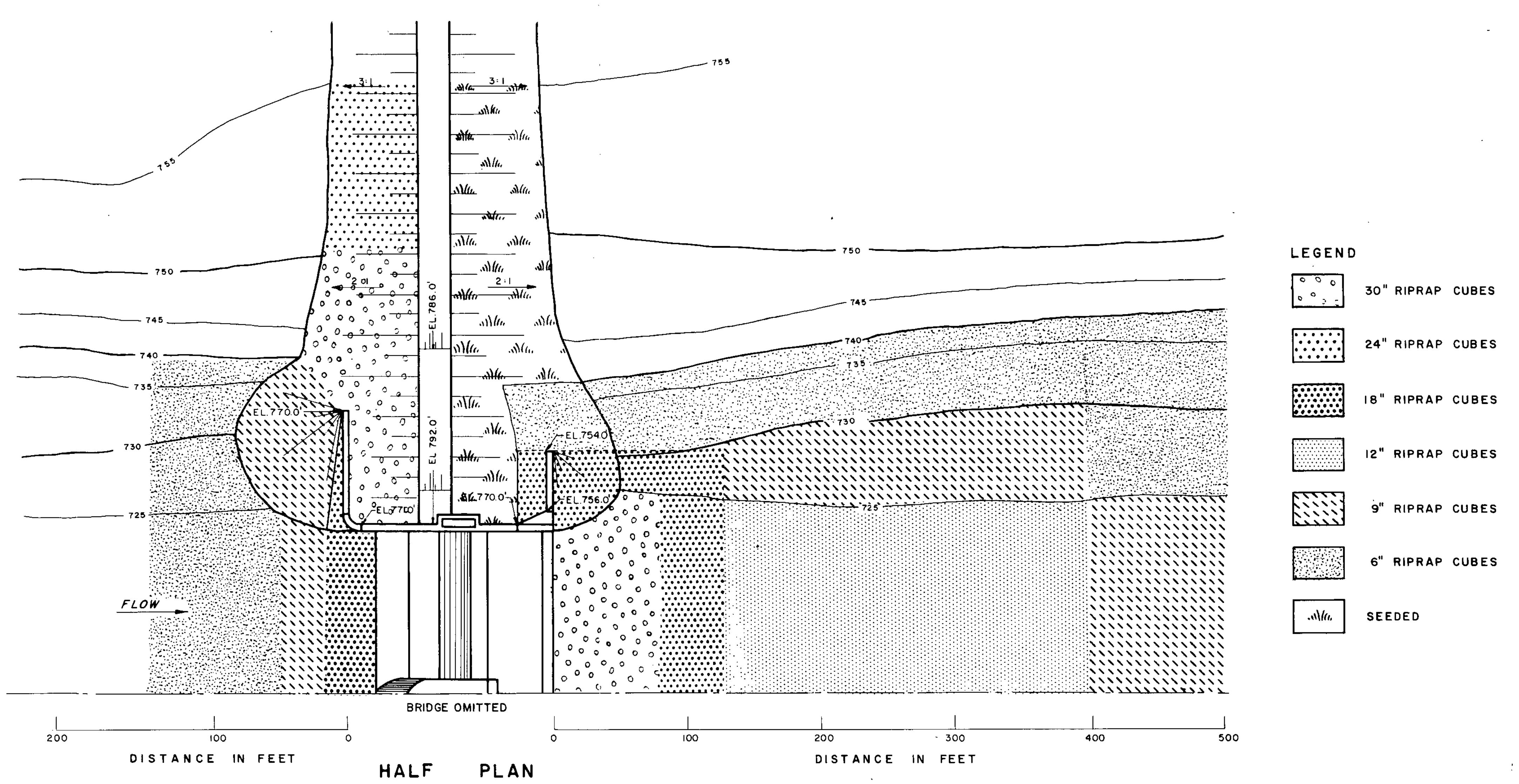
PHOTOGRAPH No. 3 — GENERAL VIEW OF THE MODEL OF THE CONTROL STRUCTURE LOOKING UPSTREAM.

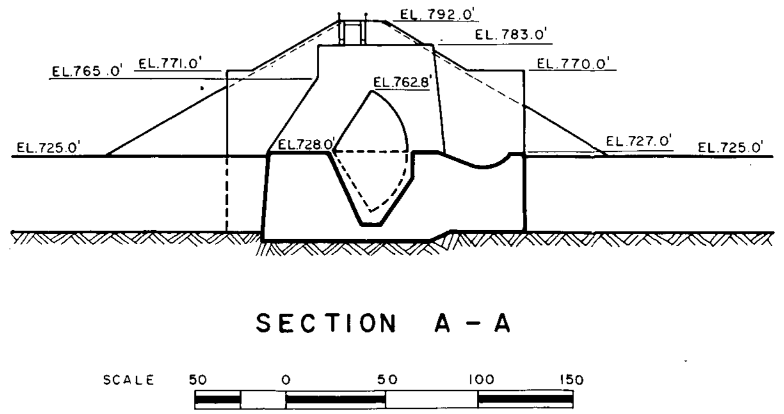


PHOTOGRAPH No. 4 — MAXIMUM PROBABLE FLOOD DISCHARGING THROUGH THE CONTROL STRUCTURE LOOKING DOWNSTREAM.

LIST OF PLATES







H.G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS

PROVINCE OF MANITOBA
DEPARTMENT OF AGRICULTURE AND CONSERVATION
WATER CONTROL AND CONSERVATION BRANCH

RED RIVER FLOODWAY
INLET CONTROL WORKS

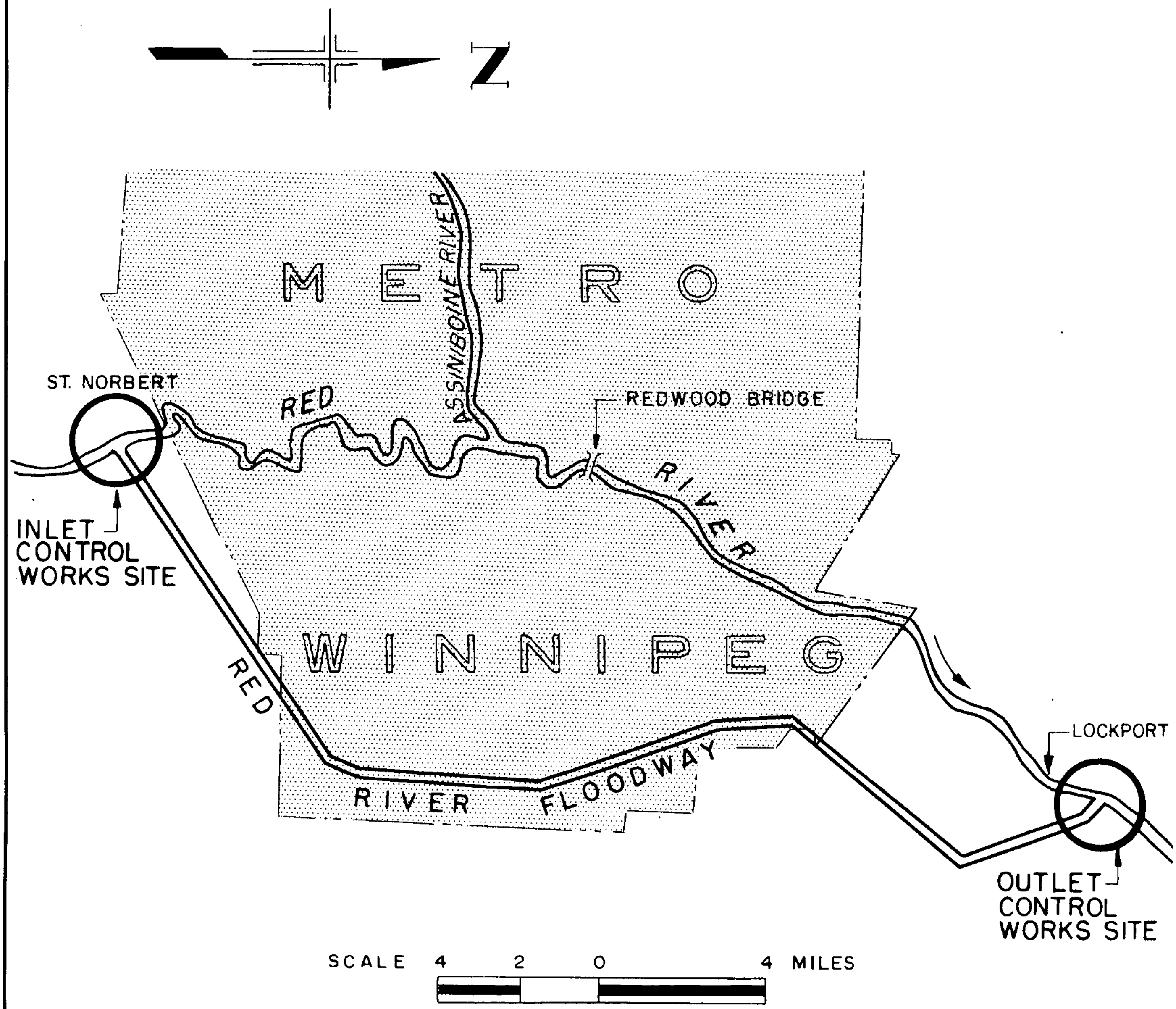
PROPOSED GENERAL ARRANGEMENT
CONTROL STRUCTURE
PLAN AND SECTIONS

H. G. ACRES & COMPANY LIMITED

DATE MARCH 1963

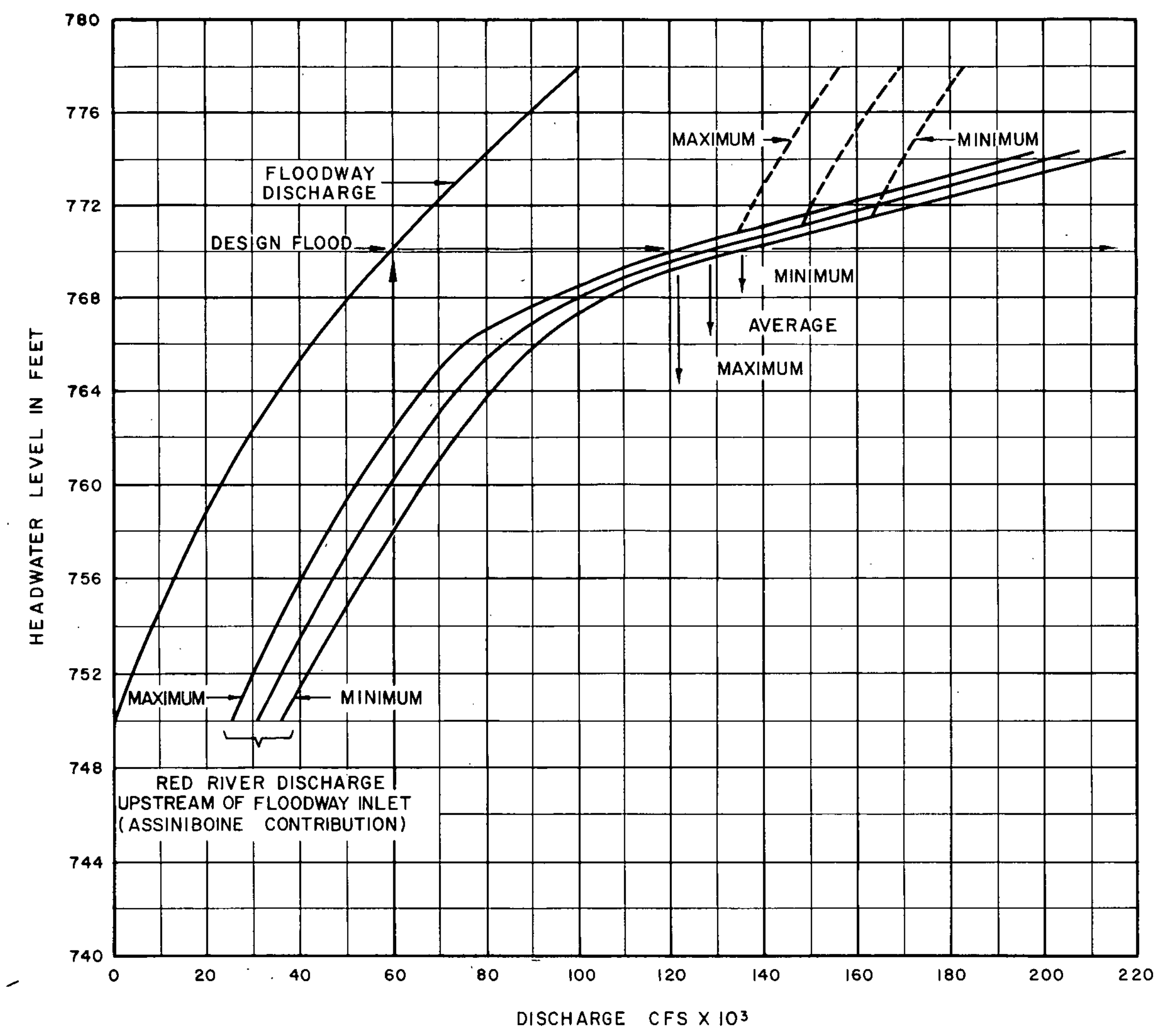
PLATE 1

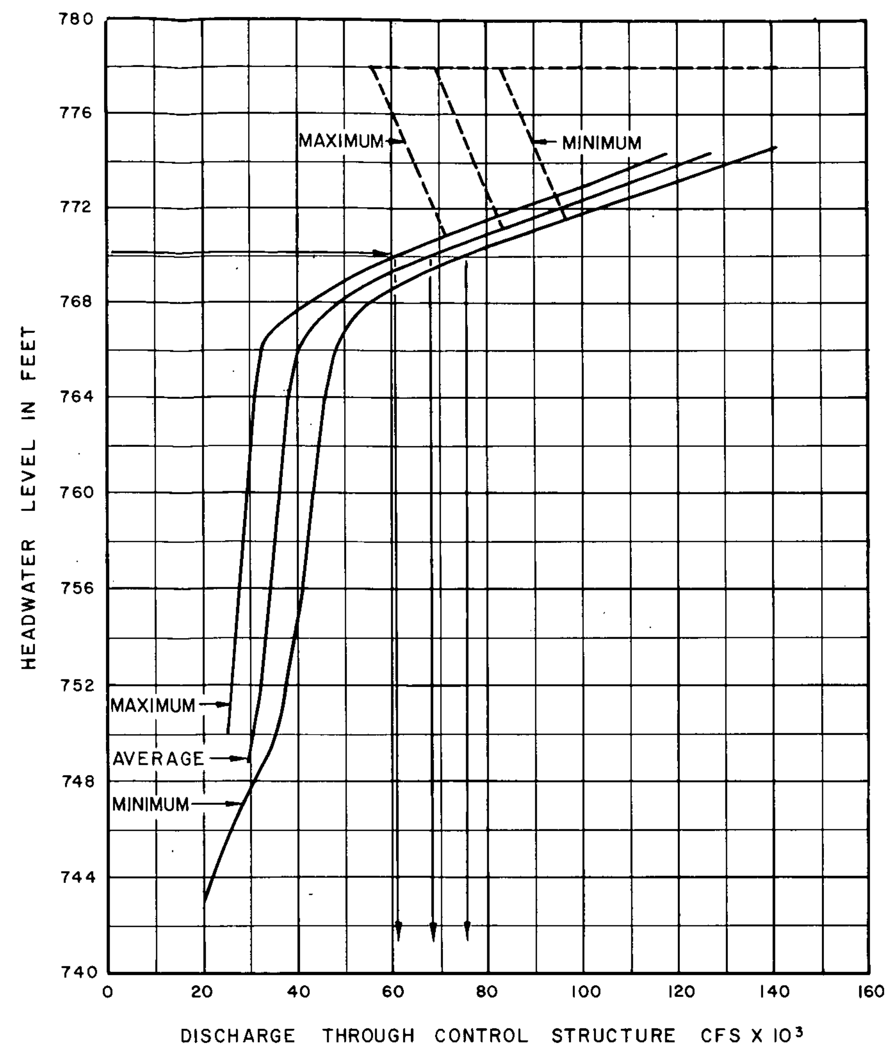
SK-940-SB-147

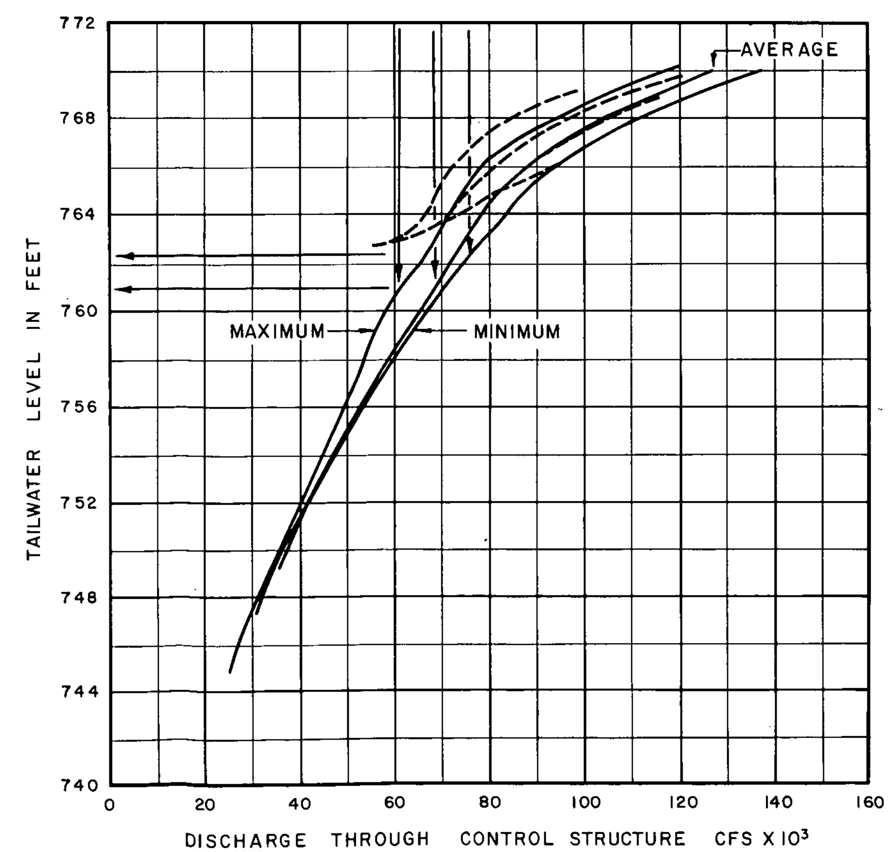


H.G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS	
PROVINCE OF MANITOBA DEPARTMENT OF AGRICULTURE AND CONSERVATION WATER CONTROL AND CONSERVATION BRANCH	
RED RIVER FLOODWAY INLET CONTROL WORKS	
KEY PLAN RED RIVER FLOODWAY	
H.G. ACRES & COMPANY LIMITED	DATE MARCH 1963
	PLATE 2

SK-940-L3-148







LEGEND

- - - - - EMERGENCY OPERATION
———— NORMAL OPERATION

H.G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS

PROVINCE OF MANITOBA
DEPARTMENT OF AGRICULTURE AND CONSERVATION
WATER CONTROL AND CONSERVATION BRANCH

RED RIVER FLOODWAY
INLET CONTROL WORKS

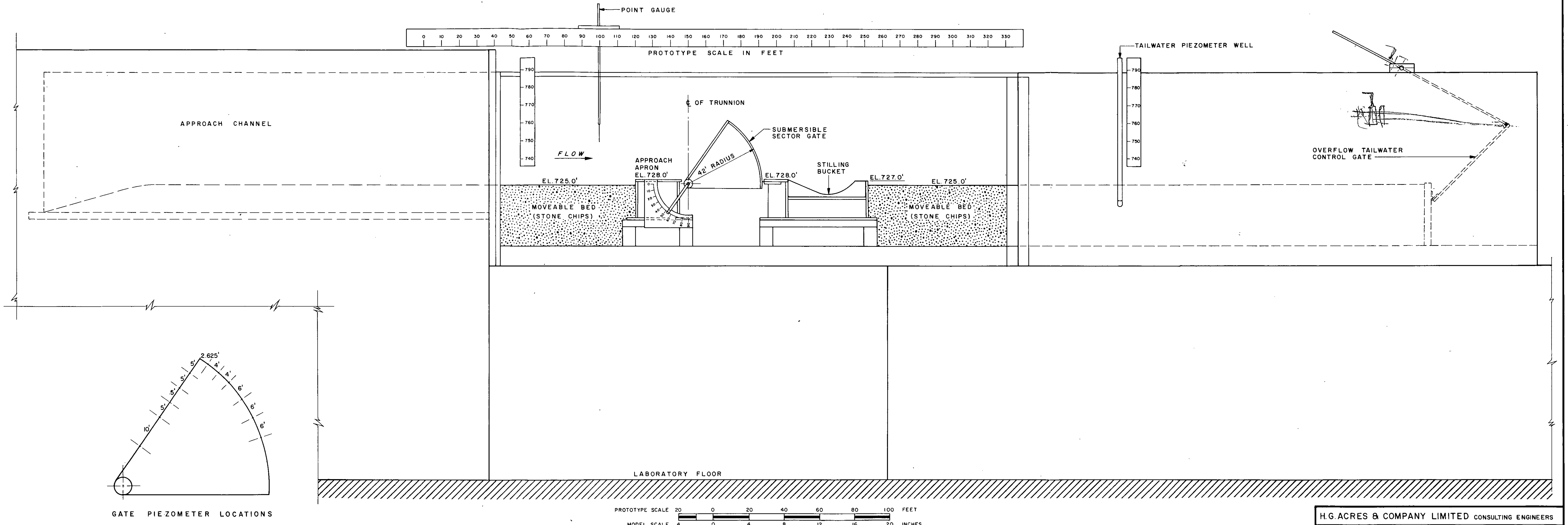
DISCHARGE RATING CURVES
INLET CONTROL WORKS

H. G. ACRES & COMPANY LIMITED

DATE MARCH 1963

PLATE 3

SK-940-C-149



H.G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS

PROVINCE OF MANITOBA
DEPARTMENT OF AGRICULTURE AND CONSERVATION
WATER CONTROL AND CONSERVATION BRANCH

RED RIVER FLOODWAY
INLET CONTROL WORKS

GENERAL ARRANGEMENT
PILOT MODEL OF THE GATE

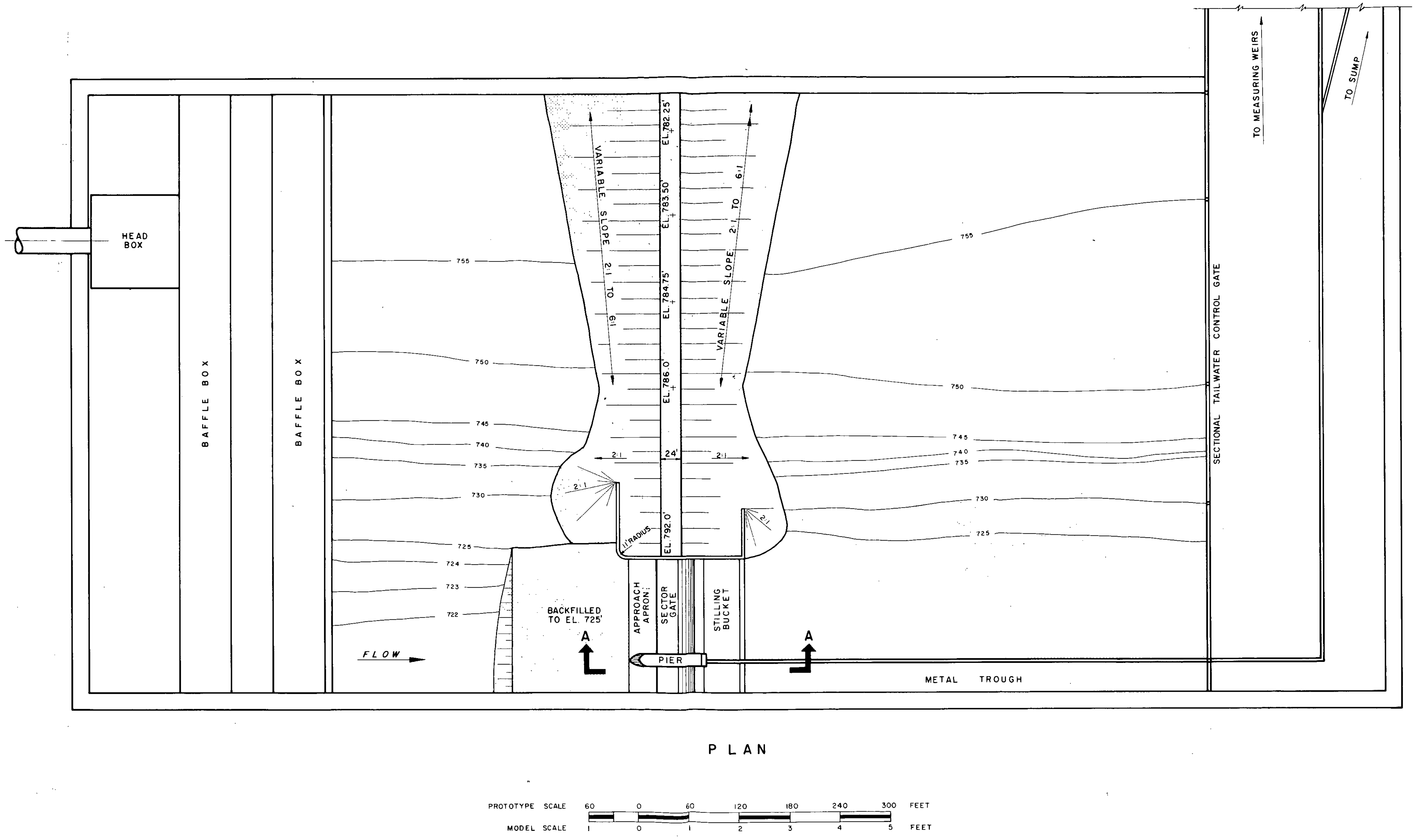
H. G. ACRES & COMPANY LIMITED

DATE MARCH 1963

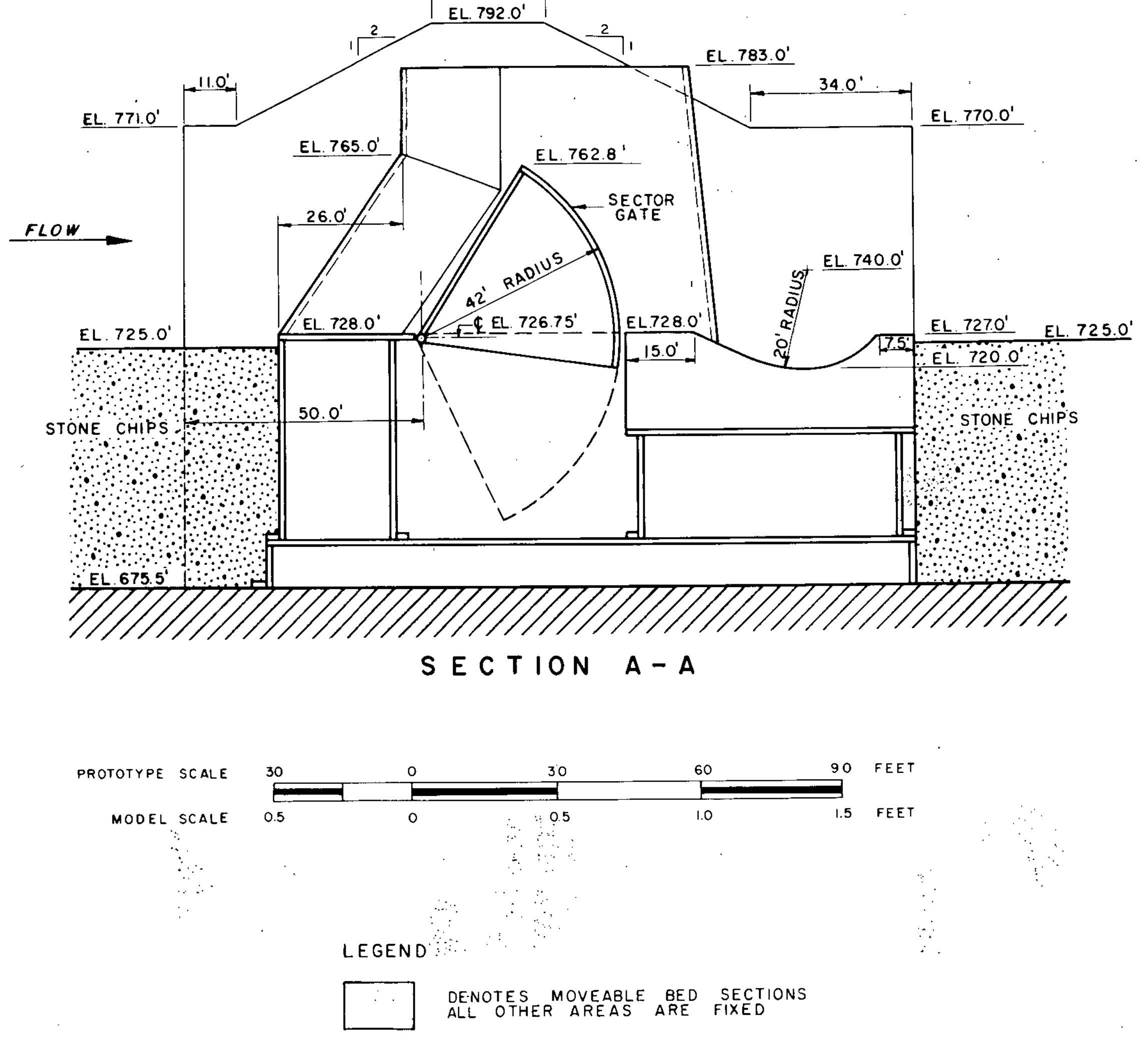
PLATE 4

SK - 940 - SB - 150

SK-9_-SB-151



PLAN



SECTION A-A

H.G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS

PROVINCE OF MANITOBA
DEPARTMENT OF AGRICULTURE AND CONSERVATION
WATER CONTROL AND CONSERVATION BRANCH

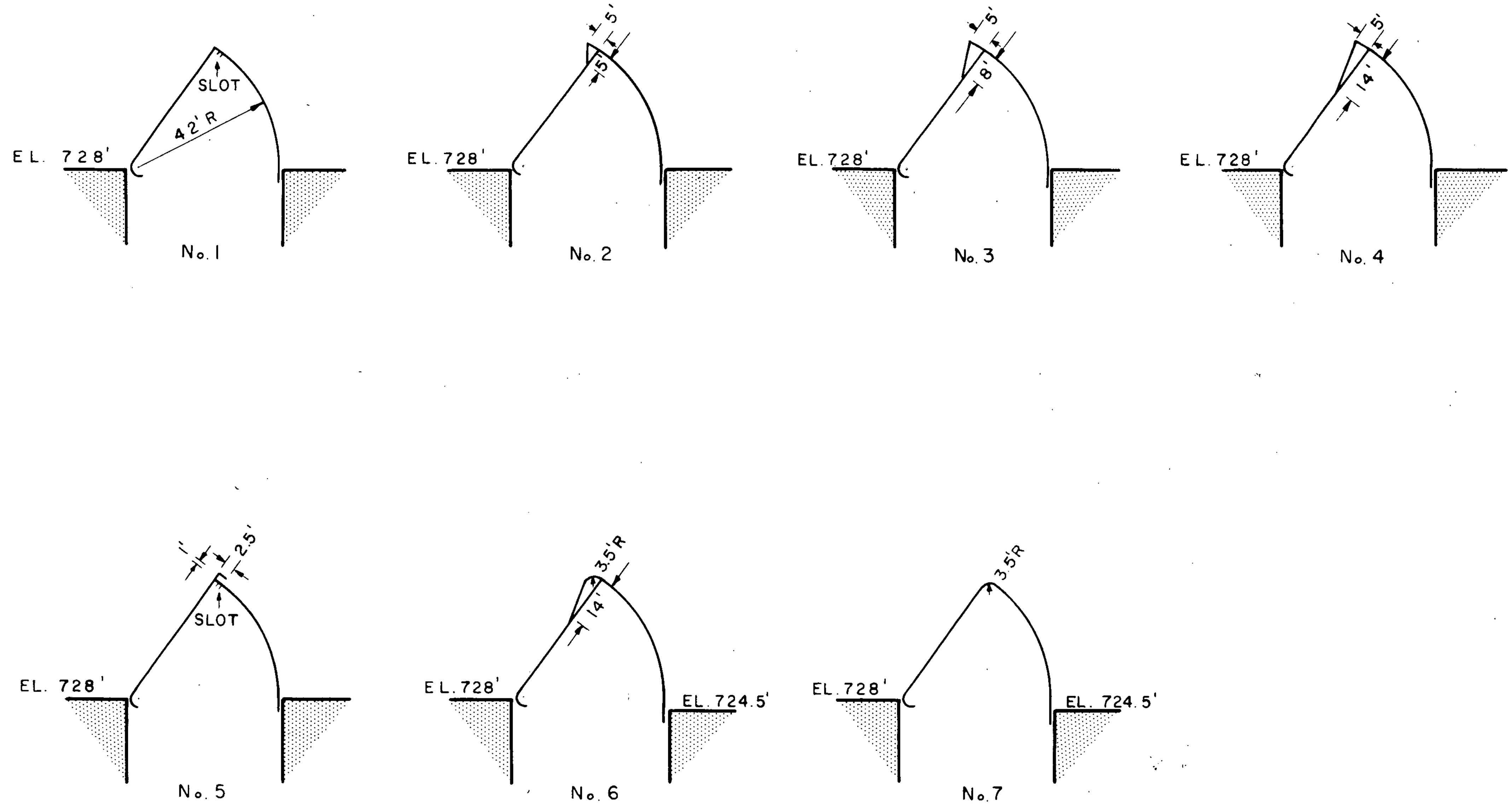
RED RIVER FLOODWAY
INLET CONTROL WORKS

GENERAL ARRANGEMENT
MODEL OF CONTROL STRUCTURE

H. G. ACRES & COMPANY LIMITED

DATE MARCH 1963

PLATE 5



H.G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS

PROVINCE OF MANITOBA
DEPARTMENT OF AGRICULTURE AND CONSERVATION
WATER CONTROL AND CONSERVATION BRANCH

RED RIVER FLOODWAY
INLET CONTROL WORKS

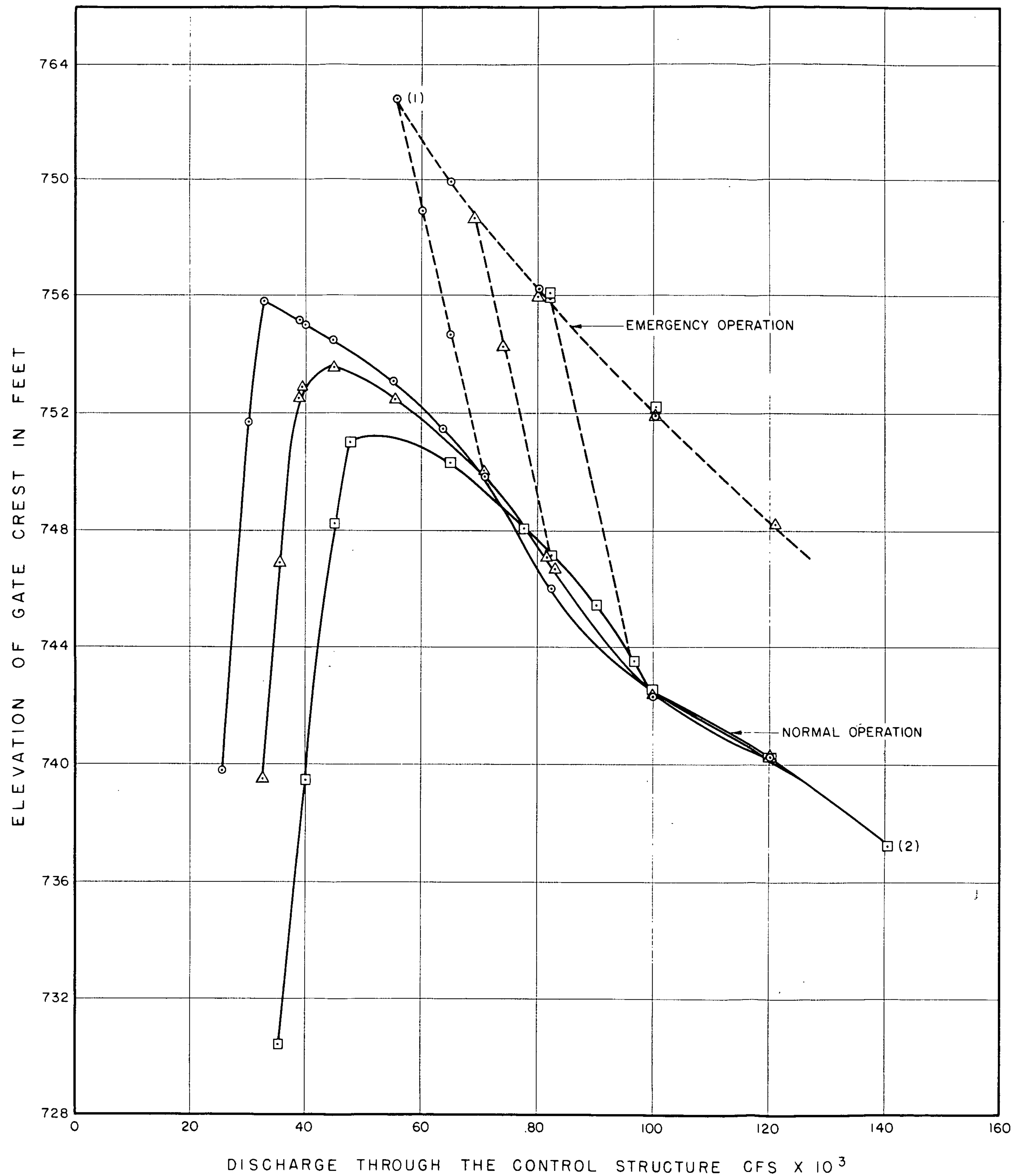
GATE SHAPES
TESTED IN PILOT MODEL

H.G. ACRES & COMPANY LIMITED

DATE MARCH 1963

PLATE 6

SK-940-A-152



LEGEND

⊙ — MAXIMUM ASSINIBOINE CONTRIBUTION

△ — AVERAGE ASSINIBOINE CONTRIBUTION

⊡ — MINIMUM ASSINIBOINE CONTRIBUTION

NOTES

(1) COMBINATION OF DISCHARGE AND WATER LEVELS WHICH DETERMINES MAXIMUM GATE HEIGHT

(2) MAXIMUM PROBABLE FLOOD DETERMINES STRUCTURE WIDTH

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WATER CONTROL AND CONSERVATION BRANCH

RED RIVER FLOODWAY
INLET CONTROL WORKS

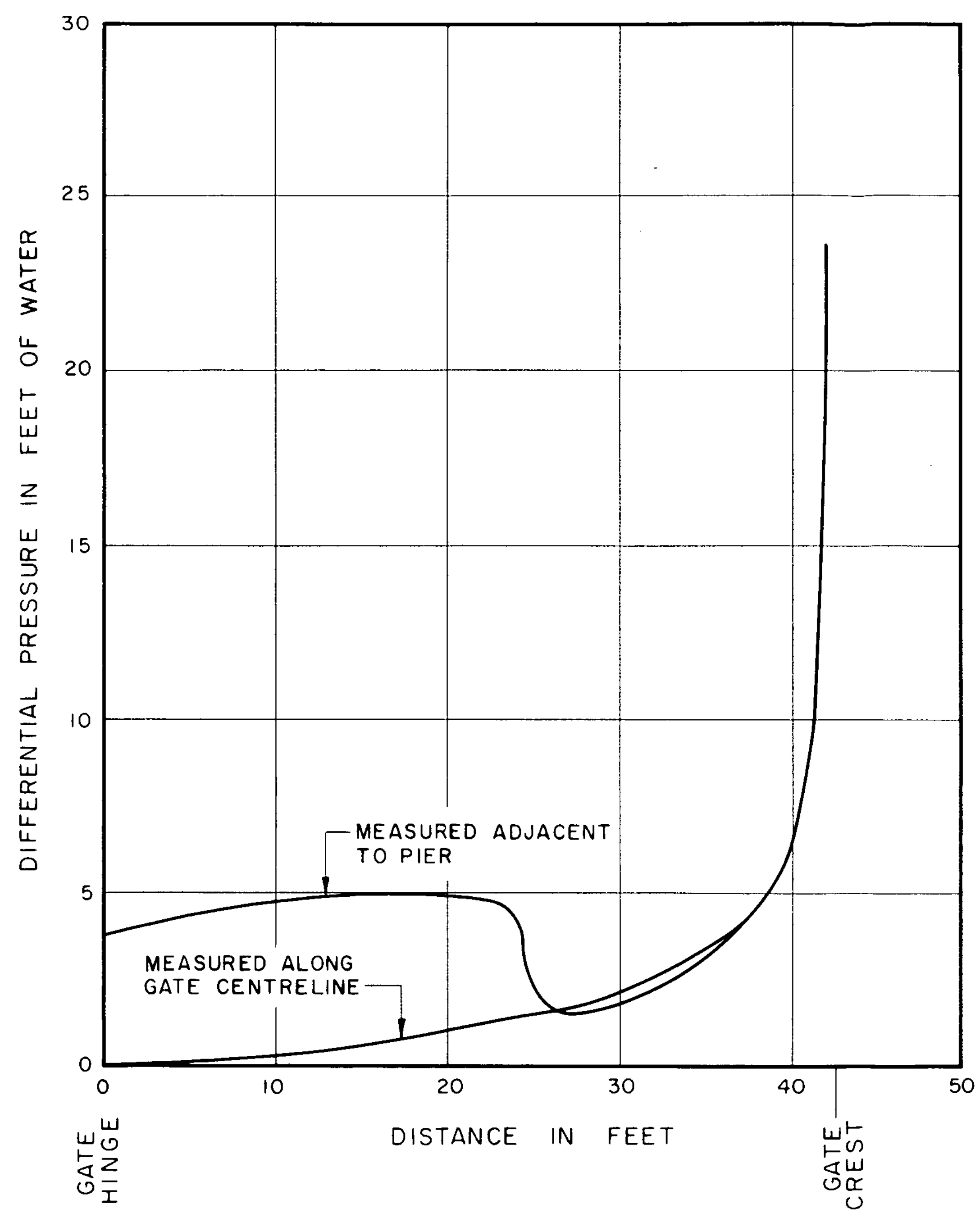
OPERATING CURVES
SUBMERSIBLE SECTOR GATES
FROM MODEL TESTS

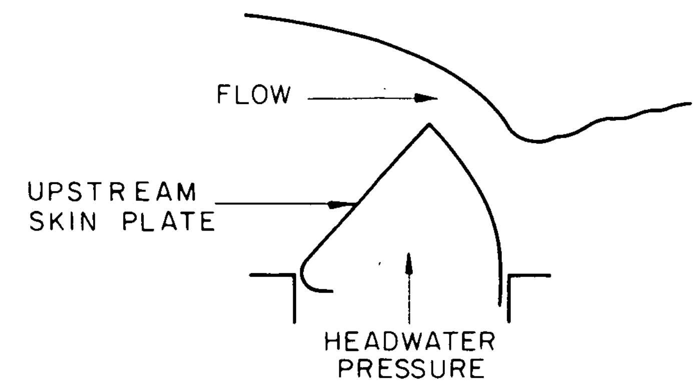
H.G. ACRES & COMPANY LIMITED

DATE MARCH 1963

PLATE 7

SK-940-A 153





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RED RIVER FLOODWAY
INLET CONTROL WORKS

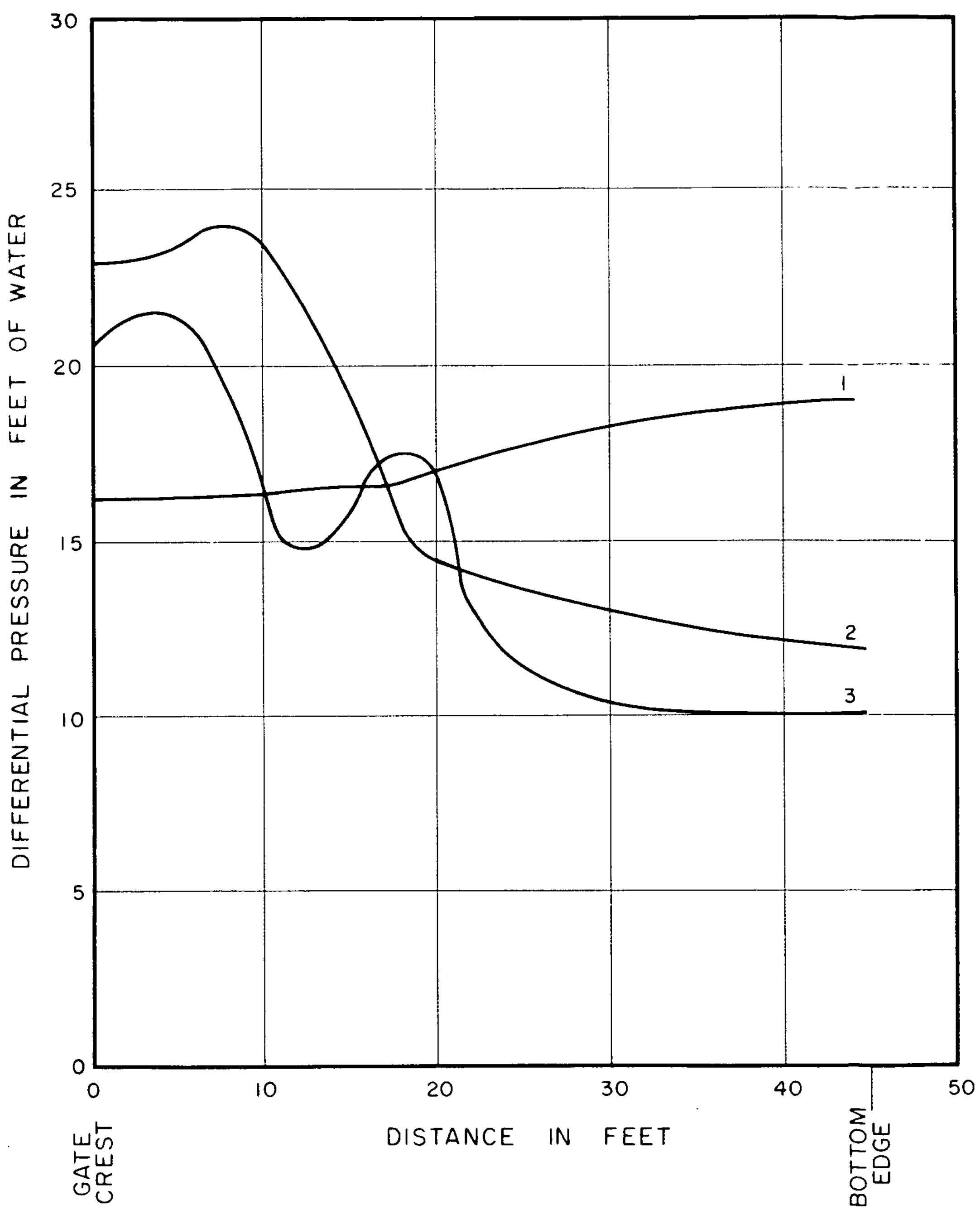
UPSTREAM SKIN PLATE
DIFFERENTIAL PRESSURE DISTRIBUTION
FROM MODEL TESTS

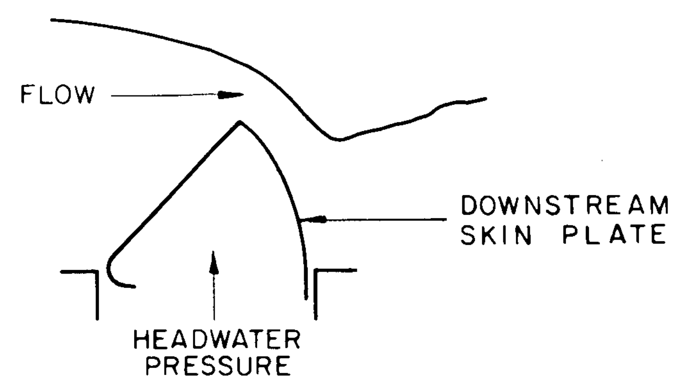
H.G. ACRES & COMPANY LIMITED

DATE MARCH 1963

PLATE 8

SK - 940 - A - 154





	DISCHARGE	ASSINIBOINE CONTRIBUTION	OPERATION
1	40,000	MAXIMUM	NORMAL
2	55,800	MAXIMUM	EMERGENCY
3	32,500	MAXIMUM	NORMAL

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INLET CONTROL WORKS

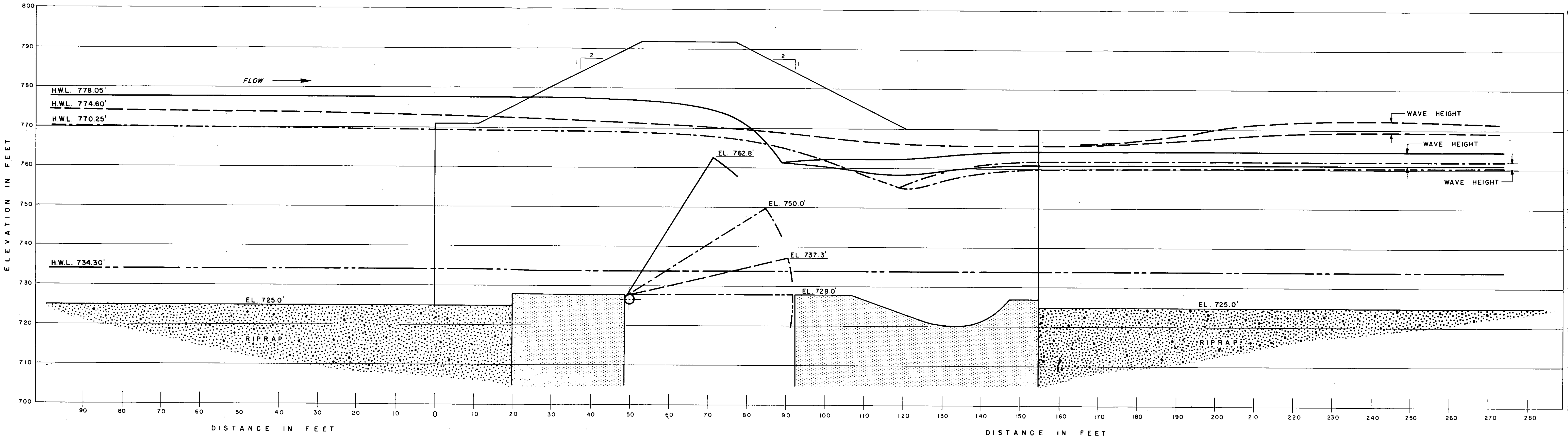
DOWNSTREAM SKIN PLATE
DIFFERENTIAL PRESSURE DISTRIBUTION
FROM MODEL TESTS

H.G. ACRES & COMPANY LIMITED

DATE MARCH 1963

PLATE 9

SK - 940 - A - 155



LEGEND

SYMBOL	DISCHARGE - CFS	ASSINIBOINE CONTRIBUTION	OPERATING CONDITION
———	55,800	MAXIMUM	EMERGENCY
—·—·—	70,700	AVERAGE	NORMAL
— — —	141,000	MINIMUM	NORMAL
—··—··—	5,000	SUMMER FLOW	SUMMER FLOW

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RED RIVER FLOODWAY
INLET CONTROL WORKS

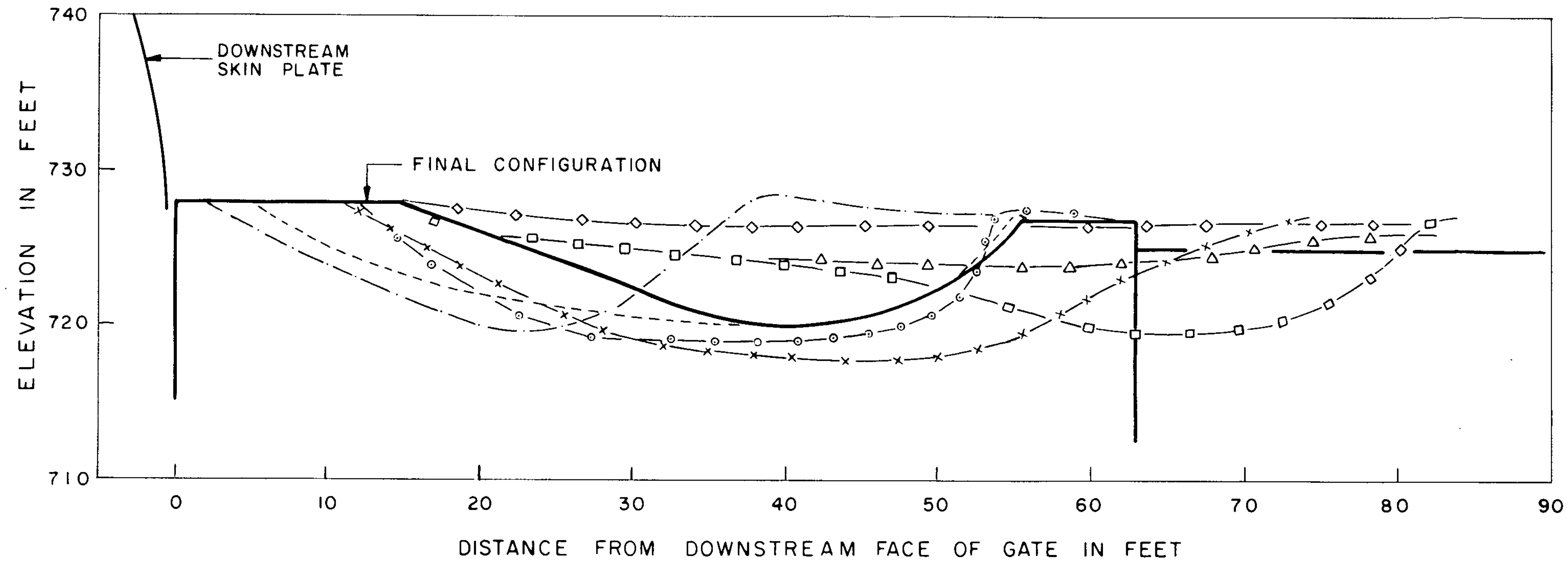
WATER SURFACE PROFILES
FROM MODEL TESTS

H. G. ACRES & COMPANY LIMITED

DATE MARCH 1963

PLATE 10

SK-940-SB-156



LEGEND

	DISCHARGE CFS	ASSINIBOINE CONTRIBUTION	OPERATING CONDITION
—·—·—	32,750	MAXIMUM	NORMAL
- - - - -	40,000	AVERAGE	NORMAL
—×—×—	50,000	MINIMUM	NORMAL
—○—○—	55,800	MAXIMUM	EMERGENCY
—□—□—	60,000	MINIMUM	NORMAL
—△—△—	70,000	AVERAGE	NORMAL
—◇—◇—	90,000	MAXIMUM	NORMAL

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PROVINCE OF MANITOBA
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WATER CONTROL AND CONSERVATION BRANCH

RED RIVER FLOODWAY
INLET CONTROL WORKS

SCOUR PROFILES STILLING BUCKET AREA FROM PILOT MODEL

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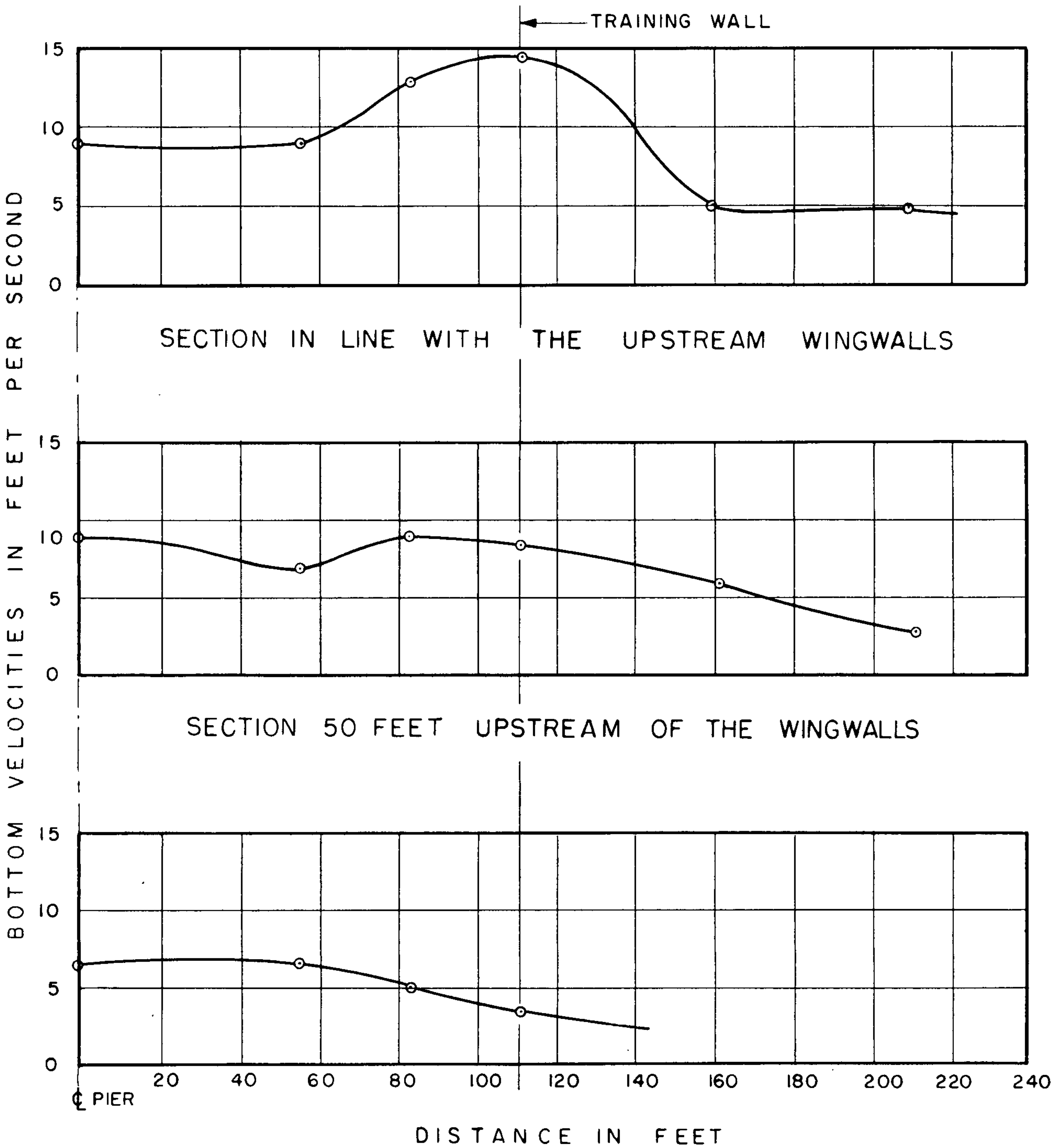
PLATE 11

SK-940-A-157



SECTION 100 FEET UPSTREAM OF THE WINGWALLS

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RED RIVER FLOODWAY
INLET CONTROL WORKS

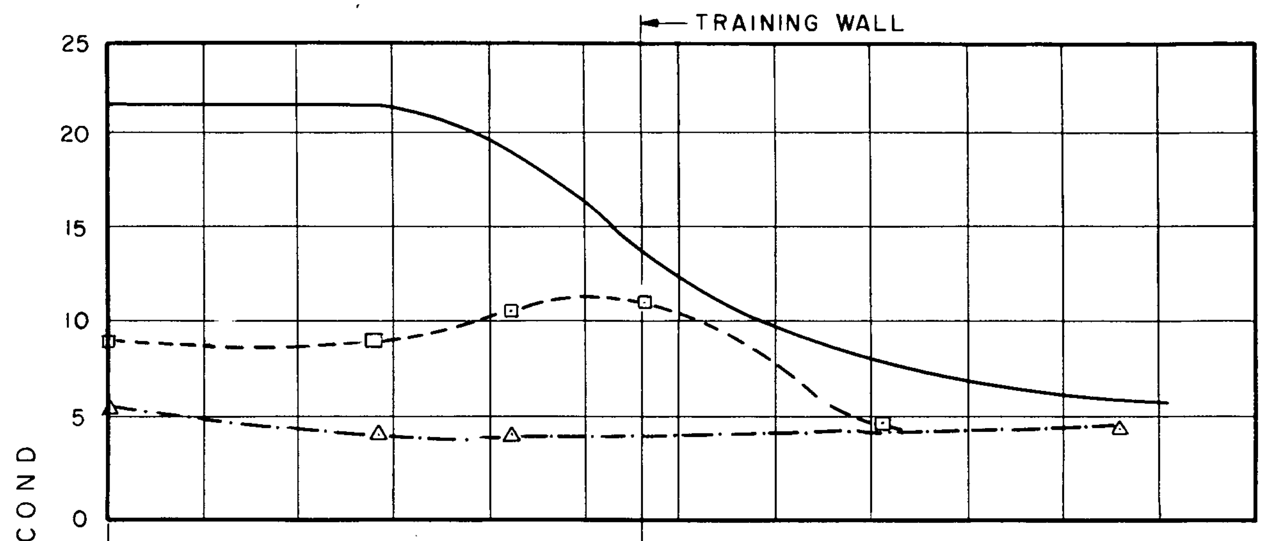
ENVELOPES OF MAXIMUM
UPSTREAM BOTTOM VELOCITIES
FROM MODEL TESTS

H.G. ACRES & COMPANY LIMITED

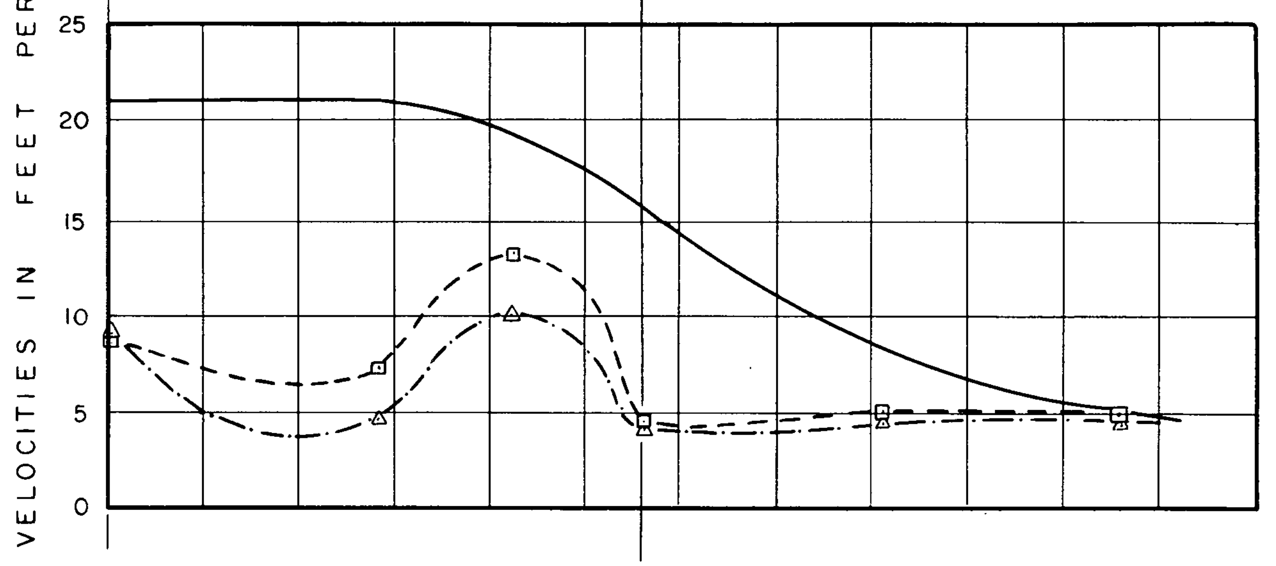
DATE MARCH 1963

PLATE 12

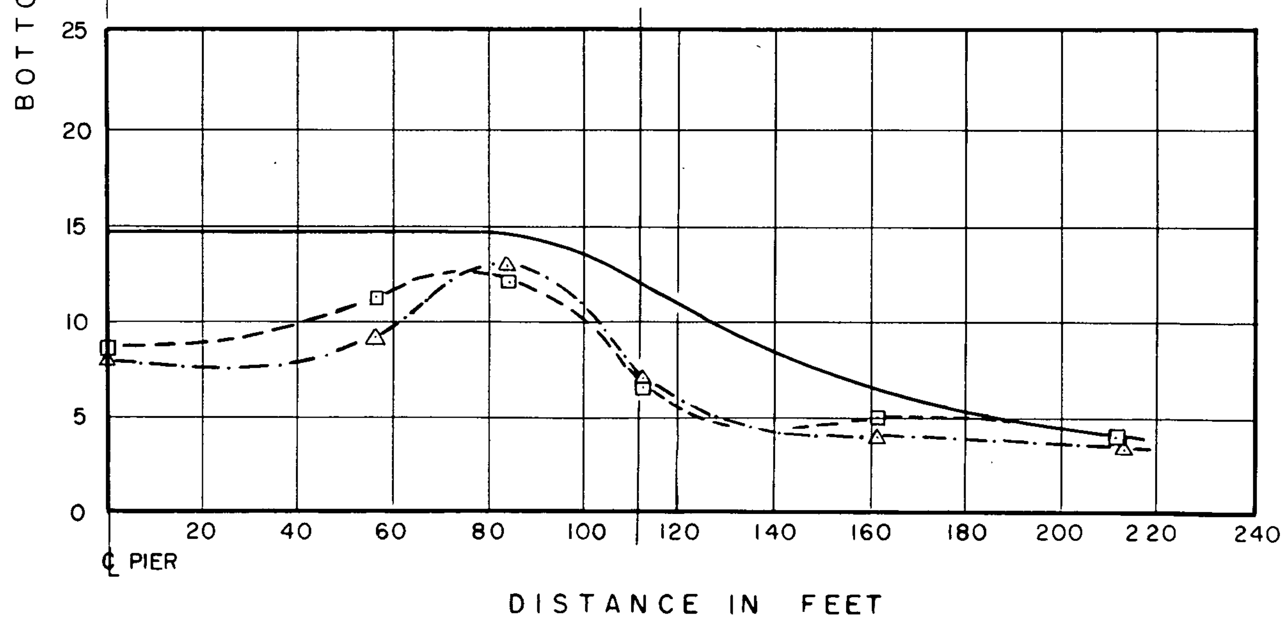
SK-940-L3-158



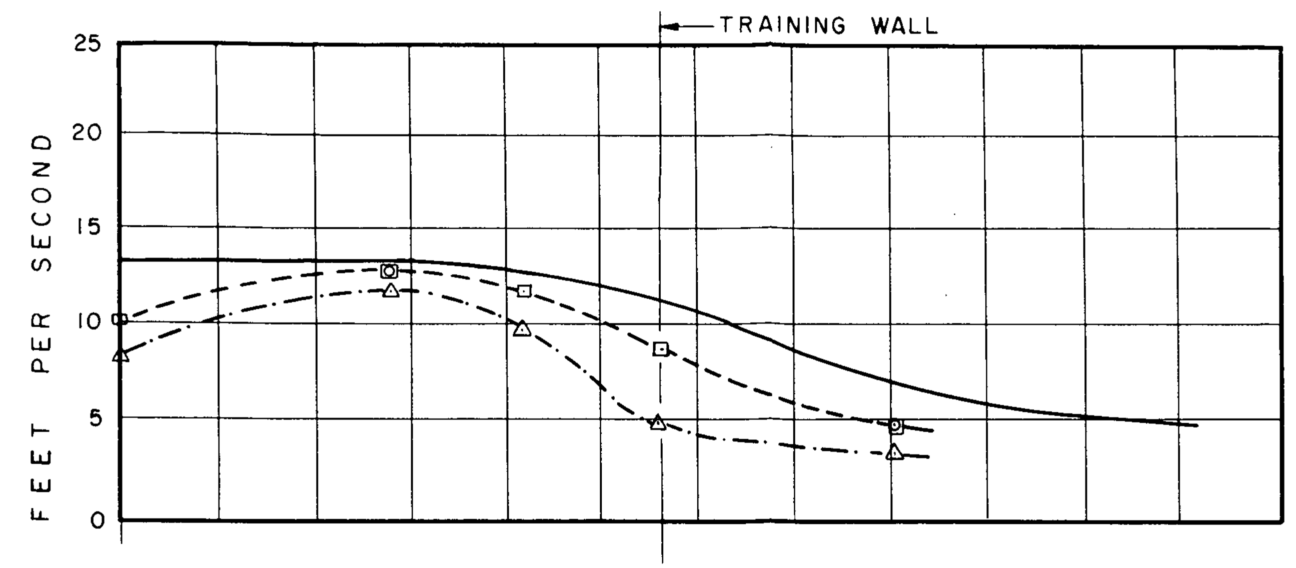
SECTION IN LINE WITH THE DOWNSTREAM WINGWALLS



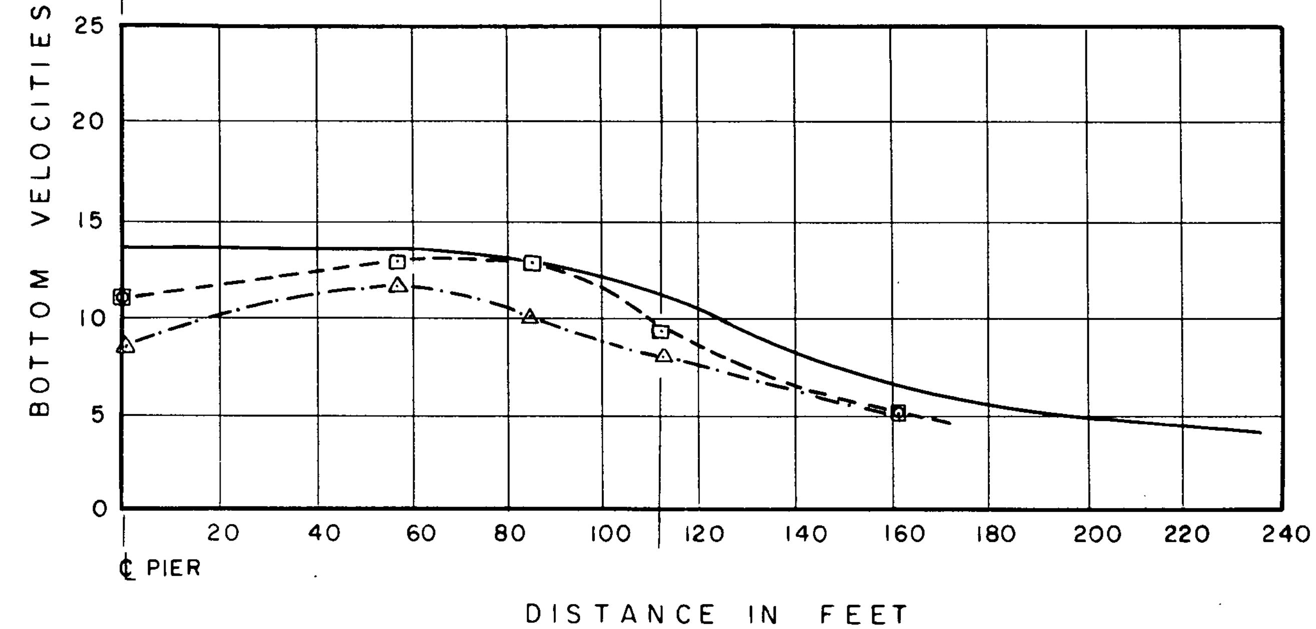
SECTION 50 FEET DOWNSTREAM FROM THE WINGWALLS



SECTION 100 FEET DOWNSTREAM FROM THE WINGWALLS



SECTION 200 FEET DOWNSTREAM FROM THE WINGWALLS



SECTION 300 FEET DOWNSTREAM FROM THE WINGWALLS

LEGEND

- ——— ENVELOPE OF MAXIMUM READINGS
- ---□--- MAXIMUM PROBABLE FLOOD MEASUREMENTS
- -·-△-·- DESIGN FLOOD MEASUREMENTS

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INLET CONTROL WORKS

ENVELOPES OF MAXIMUM
DOWNSTREAM BOTTOM VELOCITIES
FROM MODEL TESTS

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PLATE 13

SK-940-A-159