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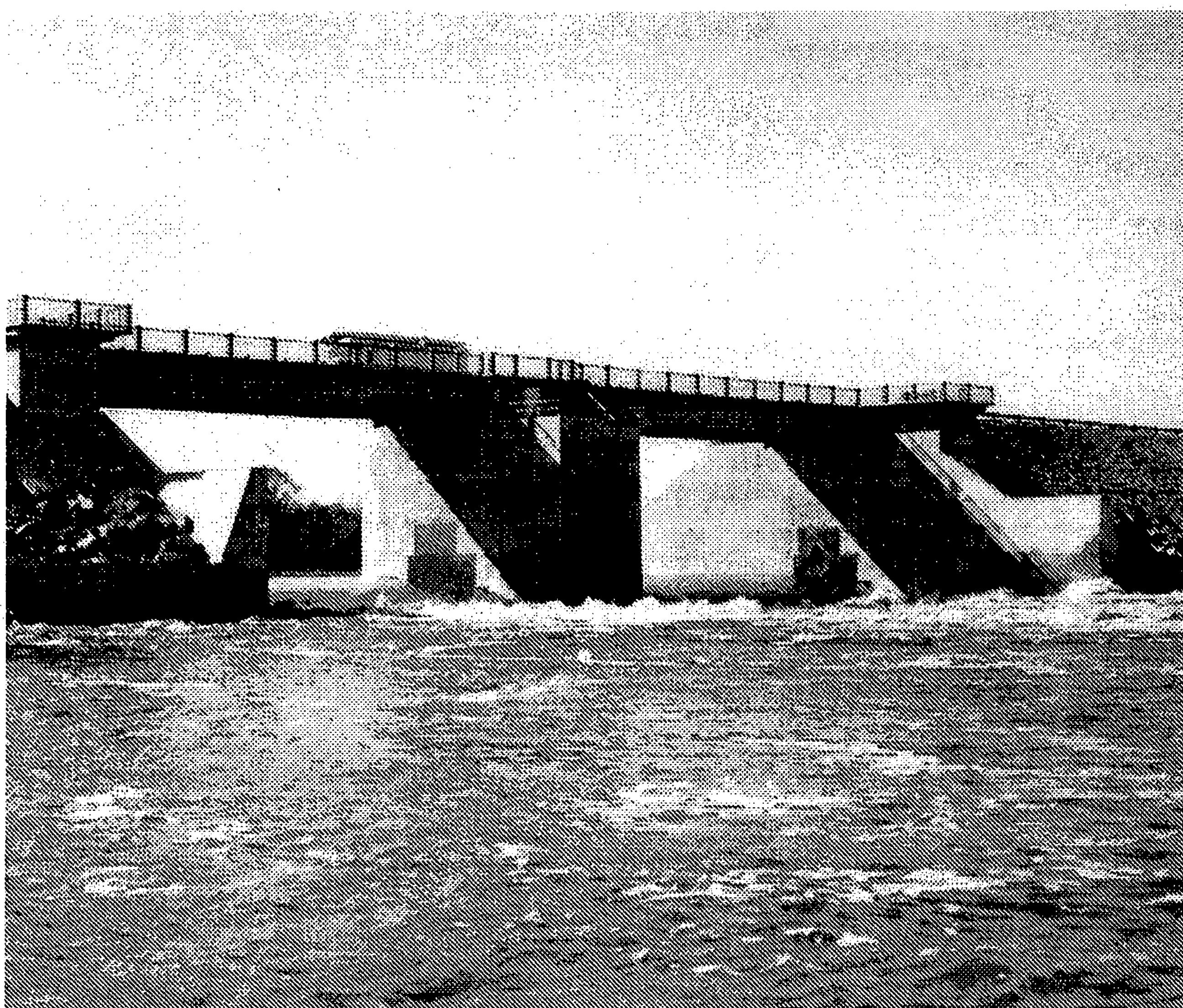
Manitoba

Natural Resources

RED RIVER FLOODWAY

INLET CONTROL STRUCTURE

EROSION STUDY



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November 21, 1995 File No. 95-311-01

Department of Natural Resources
Headquarters Operations Branch
1495 St. James Street
Winnipeg, Manitoba
R3H 0W9

ATTENTION: Mr. Rick Hay, P.Eng.
Project Engineer

RE: Red River Floodway - Inlet Structure
Scour of Downstream Rip Rap

Dear Mr. Hay:

Please find enclosed six copies of our final report on the erosion at the Red River Floodway inlet, *"Red River Floodway - Inlet Control Structure Erosion Study"*, dated November, 1995. The report concludes that the scour hole downstream of the inlet structure is not stable and should be addressed.

The most cost effective method to protect this area is with the installation a concrete mat liner. The estimated cost for the construction of this scour protection scheme is $670,000. Given the high costs associated with total remediation, partial remediation or no remediation and monitoring schemes have also be presented.

We would like to thank you for your cooperation and assistance throughout the study. If you require further assistance, please call us at your convenience.

Yours very truly,

David B. MacMillan, P.Eng.
Principal

PROVINCE OF MANITOBA
D. B. MacMILLAN
11/23/95
REGISTERED ENGINEER

Brian M. Bodnaruk, P.Eng.
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Manitoba
Natural Resources

RED RIVER FLOODWAY
INLET CONTROL STRUCTURE
EROSION STUDY

NOVEMBER, 1995

KGS GROUP
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EXECUTIVE SUMMARY

Over the past 28 years of operation of the floodway inlet structure, a scour hole approximately 15 feet deep has developed downstream of the structure. The extent of the scour hole was defined by surveys conducted in 1975, 1976, 1979 and 1994. Although no significant change in the depth of the scour hole has occurred since 1979, the extent of the scour has increased, and concern has been expressed regarding the integrity of structure and the adjacent earth dam embankment. On this basis, it was decided that the problem should be further assessed and remedial measures taken if necessary. KGS Group was authorized to assess the problem and recommend a course of action for stabilizing the scour hole, if necessary.

Typical operation of the floodway during the passage of a flood event results in a wide range in the hydraulic conditions during which the gates are operated from the completely lowered position at the start of the floodway inlet operation, to the maximum raised position at the time of the peak inflow, and finally to the completely lowered position at the end of operation. The head drop through the inlet structure, and the velocity in the stilling basin and in the downstream channel, can vary through extreme ranges during the passage of a flood event. As a result, the stilling basin and rip rap in the downstream channel can be subjected to adverse flow conditions even during relatively minor flood events.

The design of the rip rap protection for the riverbed downstream of the inlet structure was based on hydraulic model studies. Hydraulic conditions were identified in the model for which the high velocity water "jet" would plunge over the gates and past the end of the concrete stilling basin. However, rather than providing a longer concrete stilling basin to contain the turbulent high velocity flow and dissipate its energy in the design of the structures, it was decided to protect the river bed with large rip rap which was considered to be adequate to prevent potential scour.

A review of survey data collected since 1975, shows that the deepest scour has occurred in an area downstream of each bay, within 50 feet of the downstream end of the control structure. A significant area of scour is also shown to extend 150 feet to 250 feet downstream of the control structure. The scour hole has continued to increase both in depth and in areal extent since the first survey taken in 1975. Therefore, the scour process has not yet reached a stable equilibrium state. However, the amount of additional scour from each flood event is judged to be not that significant.

On the basis of the hydraulic analyses and the review of existing data, including photographs taken during past flood events, it has been concluded that the erosion of the existing rip rap has occurred, in part, from a concrete apron downstream of the stilling basin which was too short. As a result, the water jet ensuing from the inlet gates plunges past the end of the concrete stilling basin and impacts on the rip rap downstream of the structure. The observed loss of rip rap protection demonstrates that the rip rap, as placed, was inadequate to withstand the amount of the energy contained in the high velocity, highly turbulent flow experienced during the 16 years that the structure was operated.

Because of the highly turbulent flow conditions, the stability and ultimate limits of the scour hole is impossible to predict. As such, two alternative schemes for full remediation of the scour have been identified in this study. These include:

1. Replacement of rip rap scoured out with larger sized rip rap which would be stable for possible future flow conditions. The estimated cost is $1,028,000.

2. Lining the scour hole with concrete filled bags and mats. This alternative is estimated to cost $670,000.

Given the relatively high cost for full remediation of the scour problem, a partial solution with annual monitoring was also considered. This scheme would involve only the protection of the toe of the east embankment at the downstream wing wall from unraveling. The estimated cost of the partial solution is approximately $50,000 to $150,000. Under this solution, continued erosion in the downstream channel will likely occur, but based on experience, the erosion will progress at a rate which allow more extensive remedial action to be taken at some future time, if required. Combined with the recommended annual maintenance program, the risk to the structure is considered to be low.

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1.0 INTRODUCTION

The Red River Floodway inlet structure was first available for operation in 1967. During the period 1967 to present, a "scour hole" 10 to 15 feet deep has formed downstream of the structure. The river bed in this area was originally protected by a riprap blanket. The design of the riprap was based upon the results of model studies conducted prior to the construction of the control structure.

The progression of the scour hole has been defined by surveys conducted in 1975, 1976, 1979 and 1994. Although no significant change in the depth of the scour hole has occurred since 1979, concern has been expressed regarding the integrity of structure and the adjacent earth dam embankment. On this basis, it was decided that the problem should be further assessed and remedial measures taken, if necessary.

KGS Group was authorized to assess the problem and recommend a course of action for stabilizing the scour hole if necessary. Following authorization to proceed, the Province of Manitoba and KGS Group met, reviewed existing files/drawings and available background data. In addition to the files, reports and drawings, a number of photographs during flood stage were made available. This data is described in Section 2. A description of the existing facility is given in Section 3.0, while the description of site condition and a preliminary site assessment is given in Section 4.0.

As part of this assessment, KGS Group retained the services of Dominion Divers to investigate the extent of the scour hole and to determine the nature of the erosion and the size of material remaining. As well, hydraulic analyses were conducted to determine the cause of the erosion and correlate the theoretical studies with the field work. The studies and results of the diving program are described in Section 5.0.

Following completion of the hydraulic studies, an assessment of the structure and embankment stability under different scour scenarios was undertaken. These stability calculations are described in Section 6.0. An assessment of the scour conditions and remedial alternatives are given in Section 7.0.

Environmental considerations associated with obtaining approvals are described in Section 8.0. Conclusions and recommendations are presented in Sections 9.0 and 10.0.

2.0 BACKGROUND INFORMATION

This section summarizes the background information utilized by KGS Group to complete this study. Information is grouped in categories of surveys, reports, photos, drawings and model studies.

2.1 SURVEYS

Surveys of the river bed have been performed a number of times since 1974 to identify the scour of the riverbed, including:

- **August 1975** - Echo Sounding survey by Manitoba Water Resources Branch consisting of 5 lines in direction of flow for a distance 600 feet upstream and 1000 feet downstream of the structure.
- **Summer 1976** - spot elevations by Manitoba Water Resources Branch of the riverbed along lines used in 1975 survey.
- **August 1979** - Echo sounding survey by Manitoba Water Resources Branch consisting of 10 profiles 600 feet upstream to 1000 feet downstream of the structure in the direction of flow.
- **Winter 1994** - spot elevation survey by Manitoba Water Resources Branch in the area from the control structure to 300 feet downstream.
- **June 1995** - Underwater diving inspection by Dominion Divers to measure riverbed elevation and to measure the size of riprap at 64 locations in an area from the control structure to approximately 250 feet downstream.

2.2 REPORTS

- A.G. Mensforth, *Selection of the Inlet Structure Site*, June 13, 1957
- J. Mishtak, *Results of Soil Mechanics Investigations - Proposed Greater Winnipeg Floodway Control Structure*, June 13, 1961
- A.O. Dyregrov, *Floodway - Intake Structure*, Feb. 1,1962
- R.L. Walker, *Red River Floodway - Soils Laboratory Tests - Inlet Area Samples*, March 6, 1962
- A.O. Dyregrov, *Red River Floodway - Soils Laboratory Tests - Inlet Area Samples* , March 8, 1962
- G. Halliday, M.Young, *Red River Floodway Inlet Control Structure Erosion Investigation,* Sept.17, 1975
- D.Pickell, *Red River Floodway Inlet Control Structure Erosion Investigation*, July 22,1976
- M.Young, *Riprap on West Bank of the Red River Opposite the Floodway Outlet Control Structure,* Aug. 16, 1979
- M.Young, *Red River Floodway Inlet Control Structure Erosion Investigation,* August 22, 1979
- M.Young, *Red River Floodway Inlet Control Structure Erosion Investigation,* August 27, 1979
- F. Penner, *Red River Floodway Inlet Control Structure Erosion Investigation,* Sept. 5, 1979

2.3 PHOTOS

- Red River Inlet Structure- Erosion of the Downstream Fill - 12 photos (1974)
- Red River Floodway Control Structure - Erosion of concrete downstream of gates - 6 photos (1976)

2.4 DRAWINGS

940-D-2002 Diversion Channel and Detour of St. Mary's Road
940-D-2004 Diversion Channel Excavation and Embankments
940-D-2005 Diversion Channel Exploratory Drilling
940-D-3000 Site Plan
940-C-3001 General Arrangement of Project
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940-D-4000 2/3 Gates and Hoists - General Arrangement
940-D-4000 3/3 Gates and Hoists - General Arrangement

2.5 MODEL STUDIES

H.G. Acres & Co. Ltd., Hydraulic Model Results of the Submersible Gates and the Inlet Structure, March, 1963.

3.0 DESCRIPTION OF THE FLOODWAY INLET STRUCTURES

The Red River Floodway project consists of a diversion channel with the inlet located on the Red River just south of Winnipeg in St. Norbert, and a control structure on the Red River downstream from the diversion channel (Figure 3.1). The entrance to the floodway channel is partially restricted by a low berm constructed to elevation 750 ft (228.6 m).

During flood conditions, the Shellmouth Reservoir, Portage Diversion and the Red River Floodway all contribute to reducing the flow in the Red River in The City of Winnipeg. The diversion and control of the flow by these projects results in a lower than "state of nature" water surface level at the inlet. The submerged gates at the floodway inlet control structure are then partially raised off the bottom to constrict the flow and return the water level upstream of the gates to the estimated "natural" level.

The inlet control structure is a two bay gated concrete structure as shown in Figures 3.2 and 3.3. The bays, which are 112.5 feet in length, are equipped with submersible gates. The gates were designed with the hinge at the upstream edge to reduce the possibility of ice damage to the downstream skin plates during the spring runoff. During periods of normal summer flow conditions, the gates are retained in a lowered position with the top of the gate positioned flush with the base slab at elevation 728.0 feet.

For the design of the stilling basin, a flip bucket was chosen as the most desirable method of dissipating the energy. When the gate is raised to a relatively high level and the depth of flow over the gate is low, the water drops nearly vertically onto the bucket, thus containing the high velocity flow. However, when the depth of flow over the gate is large and the gate is in a low position, the water is deflected upward by the angle of the gate, with the result that the jet plunges at an angle past the end of the basin. From observations of photographs taken during several floods, it appears that this condition occurs frequently.

This hydraulic condition was recognized during the hydraulic model tests. For the design, it was decided that the protection of the channel downstream of the concrete structure could be provided with rip rap. The location and size of the rip rap was also determined by the hydraulic model studies.

Based on the model results, a rip rap protection blanket was constructed for a distance of 800 feet downstream of the structure (see Figure 3.4). The rip rap blanket consisted of a zone of 30-inch (D_{50}) rip rap for the first 80 feet, followed by a 40 foot length of 18-inch rip rap, and approximately 250 feet of 12-inch rip rap. A 9-inch rip rap blanket was provided for the channel from 375 feet to 800 feet downstream of the structure.

4.0 PRELIMINARY SITE ASSESSMENT

The site assessment at the floodway inlet considered the following:

- **Visual Inspection** - Photographs taken during operation of the floodway inlet structure from 1969 to 1995 were reviewed for evidence of flow turbulence and indications of erosion. (Appendix A - Photographs).

- **Divers Report** - An assessment of the condition of the existing rip rap protection was conducted from a review of divers inspection reports undertaken in 1995. The underwater reconnaissance inspection program, conducted as part of this study, included the measurement of the river bed elevation at specific locations, as well as an assessment of the condition of the existing rip rap at each inspection location. At a number of locations the inspection included a detailed measurement and count of all stones within a measured area. (The divers reconnaissance report is included in Appendix A).

- **Discussions with Divers** - Discussions with the divers during and after the underwater inspection provided further information and proved valuable in assessing any anomalies in the observations.

- **Riverbed Contours** - Detailed contours of the riverbed were defined from echo sounding measurements following the spring floods in 1979 and from spot soundings taken in 1994. Contours were also prepared for the same area from the 1995 underwater inspection program. The contour plans prepared from the 1979 and 1995 data, are shown on Figures 3.4 and 3.5, respectively.

- **Hydraulic Model Studies** - Hydraulic model studies conducted for the design of the Inlet Control Structure were utilized to provide information for the assessment of the performance of the stilling basin. Data in the model study report included photographs of a sectional view of the model, which illustrated the flow patterns of the water jet as it plunged from the control gate to the stilling basin and the downstream channel.

5.0 HYDRAULIC ASSESSMENT

5.1 REVIEW OF PREVIOUS STUDIES

The erosion of the rip rap downstream of the control structure was first surveyed by the Manitoba Water Resources Branch using echo sounders in August and September 1975. The results of these surveys showed that erosion of the rip rap had occurred in the first 50 feet downstream of the control structure. A second hole was noted between 100 feet and 200 feet downstream of the control structure. The scour in the first 50 feet was shown to have progressed through the Class 11 (30-inch D_{50}) and into the Class 5 (6-inch filter stone) material.

No attempt was made to review the cause of the erosion. The recommended remedial action was to replace the rip rap scoured with larger sized rip rap. It was estimated that approximately 2200 yd^3 of rip rap would be required to fill the eroded bed to the original design grade. Since it was uncertain if the scour had reached an equilibrium state, it was recommended that no remedial work be undertaken at that time, and another survey was proposed for the following year.

The channel was re-surveyed by spot soundings by the Water Resources Branch in 1976, following the spring flood. This survey showed no noticeable further erosion compared to that shown in the previous year. As a result, no remedial work was proposed. Further surveys, however, were recommended.

Following the spring flood in 1979, the Water Resources Branch conducted an echo sounding survey of the riverbed downstream of the inlet control structure. The results of the survey were summarized in the memorandum from M.Young to F. Penner, dated August 22, 1979. A similar pattern of erosion to that measured in 1975 and 1976 was noted. It was concluded that the rip rap placed in the river bed downstream of the structure was inadequate to prevent erosion. As in the previous studies, no attempt was made to analyse the cause of the erosion. The recommended remedial measure was to fill the scour holes with large rip rap. Approximately 4900 yd^3 of rip rap was estimated to fill the scour hole. The recommended works were not undertaken.

In 1994, the river bed was re-surveyed by taking spot soundings of the river bed. This survey showed that the basic pattern of the scour had remained similar to the previous surveys. The depth of scour, however, was shown to have progressed by another 2 feet since 1979. As a result of the 1994 findings, KGS Group was commissioned to review the scour problem, including the analysis of effects of the scour on the stability of the concrete and earth structures, and to propose remedial measures.

5.2 ORIGINAL DESIGN OF RIP RAP PROTECTION

The design of the rip rap protection for the riverbed downstream of the inlet structure was based on hydraulic model studies. Hydraulic conditions were identified in the model for which the jet would plunge past the end of the concrete stilling basin. However, rather than providing a longer stilling basin to contain the turbulent, high velocity flow, it was decided to protect the river bed with rip rap to resist the potential scour.

The rip rap protection, as originally designed, is shown on Figure 3.4. The rip rap for the first 80 feet was designed with a D_{50} size equal to 30 inches with a layer thickness of 5 feet. This area was followed by a 40 foot length of 18-inch , D_{50}, rip rap having a layer thickness of 2.5 feet. A layer of 12-inch rip rap with a layer thickness of 2 feet was specified for the river bed from 120 feet downstream to 375 feet downstream.

5.3 DIVERS INSPECTION PROGRAM - 1995

5.3.1 Extent of Survey

The underwater inspection program performed by Dominion Divers Ltd. in June 1995 consisted of the determination of the river bottom elevation and the measurement of the size of the rip rap at 64 pre-defined points. The divers report is included as Appendix A in this report.

The survey points were located on a 10 metre grid with 8 longitudinal lines (Lines 1 to 8) in the direction of the flow and 8 transverse lines (Lines A to H). Lines 1 and 8 were located at the limits of the east and west abutments, respectively . Line A was located 10 metres downstream of the end of the concrete structure.

5.3.2 Riverbed Elevations

The river bottom elevations measured at each observation point were used to prepare a contour plan of the riverbed shown on Figure 5.1. In preparing the contour plan, the field data provided in the divers report was first checked for anomalies. For example, the elevation of the river bed at point C-6 was measured as 713.22 ft. which is approximately 4 feet lower than the observations at adjacent points. The divers, however, reported that the scour at this location was very localized. On further review, it was concluded that local scour occurred adjacent to a 7 foot rock as a result of local turbulence induced by the large boulder. The local scour had exposed the underlying bed material and resulted in the further erosion of the river bed and the lowering of the boulder. It was concluded that the localized scour was not indicative of the general riverbed surface and the observation was excluded from the preparation of the contour plan.

Discussions with the divers also revealed that the type of transducer cone used in their echo sounder was too wide, which resulted in recording the tops of the boulders rather than the river bed. The data obtained from the echo sounder was therefore judged to be unreliable and was therefore not used in the preparation of the contour drawing.

The basic pattern of scour in 1995 remained similar to the scour in the 1994 survey. The maximum depth of the scour, as measured in the 1995 survey, remained essentially as recorded in 1994 but the lateral extent of the scour in 1995 had increased from that in 1994. The maximum depth immediately downstream from the east abutment, however, was shown to be approximately 2 feet greater in 1995 compared to the maximum depth in 1994. Also, the extent of the scour measured in 1995 was greater throughout the surveyed area. A significant increase in the scour was shown to have occurred in the area approximately 150 feet downstream of the structure. The total volume of scour below the design grade level of elevation 725 feet is estimated to be 7800 cubic yards. A similar estimate, based on the 1979 survey, was 4900 cubic yards.

5.3.3 Measured Rip Rap Size

The underwater inspection of the rip rap showed the rip rap to be larger than that specified in the original design. For example, the largest boulder at point A-6 is shown to be 96 inches. Boulders measuring 72"x60"x36", 48"x60"x24" and 48"x48"x24" were observed at point B-6. Rip Rap having dimensions greater than 48 inches were common throughout the first 20 metres downstream of the control structure.

5.4 DETAILED ASSESSMENT OF HYDRAULIC CONDITIONS

5.4.1 Floodway Operation Rules

The criteria used for operation of the floodway works are as follows:

- During normal summer runoff conditions, the water level in the river at the floodway entrance is below the crest level of the inlet lip. For these conditions, the entire discharge on the river is conveyed through the control structure and downstream on the Red River through Winnipeg.

- Under spring flood conditions or during extreme summer flood conditions, the water level at the floodway entrance may exceed the level of the inlet lip. A portion of the flow in the river will then be diverted into the floodway.

- The main operating criterion for the floodway is to prevent flooding within The City of Winnipeg while maintaining water levels upstream from the floodway inlet at "state of nature" levels which would have occurred without the benefit of the flood control facilities. During periods of high flows, the Portage Diversion and the Shellmouth Reservoir are operated to reduce the water level in the Assiniboine River between Portage la Prairie and Winnipeg and in Winnipeg. Reduced flows from the Red River, both due to the Assiniboine river flood control works and the diversion of water into the floodway result in a depressed water level at the inlet. The gates of the inlet control structure are then raised in order to increase the water level upstream of the inlet structure to the required "state of nature" level. This operation increases the discharge into the floodway channel and results in a corresponding reduction in the flow through Winnipeg.

5.4.2 Summary of Operation

Table 5.1 summarizes the history of the operation of the floodway since it construction.

Table 5.1
Floodway Inlet Structure - Summary of Operation

Year	Peak HWL (ft)	Peak TWL (ft)	Peak Floodway Discharge (cfs)	Peak Control Structure Discharge (cfs)	Duration of Operation (Days)
1967	No operation[1]				
1968	No operation[1]				
1969	759.60	748.72	21,900	36500	25
1970	759.40	750.32	21,300	39700	31
1971	754.08	749.56	7,940	40,400	11
1972	751.52	749.92	1,170	43,800	5

Year	Peak HWL (ft)	Peak TWL (ft)	Peak Floodway Discharge (cfs)	Peak Control Structure Discharge (cfs)	Duration of Operation (Days)
1973	No operation[1]				
1974	764.94	746.99	37,400	32,100	43
1975	754.56	747.85	9,350	32,210	27
1976	754.96	748.44	10,290	30,255	13
1977	No operation[1]				
1978	758.68	749.13	19,390	32,980	25
1979	765.48	752.92	39,830	44,810	40
1980	No operation[1]				
1981	No operation[1]				
1982	751.35	749.67	1,280	36,520	6
1983	751.97	750.66	2,850	36,890	4
1984	No operation[1]				
1985	No operation[1]				
1986	754.79	751.15	9,870	30,150	21
1987	758.33	751.15	18,550	35,520	12
1988	No operation[1]				
1989	752.82	749.70	4,960	39,310	11
1990	No operation[1]				
1991	No operation[1]				
1992	752.79	748.95	3,100	30,980	5
1993	No operation[1]				
1994	No operation[1]				
1995	757.41	751.46	15,590	38,390	33

1. Inlet Control structure gates not operated for this year.

5.4.3 Historic Hydraulic Conditions

As described in Section 5.4.1, the operation rules for the inlet control structure require that the floodway inlet control structure be operated to maintain "state of nature" water levels upstream of the inlet control structure. This operating rule requires that the control gates be continually adjusted during the course of a flood event in order to compensate for the diversion of Assiniboine River flows into the Portage Diversion, the reduction of Assiniboine River flow due to storage in Shellmouth Reservoir, and the diversion of Red River flows into the floodway channel. Depending on the flow in the Assiniboine River, the water level upstream of the inlet structure can vary considerably in response to the gate adjustments. The controlled discharge downstream of the inlet structure, however, is governed primarily by the flood stage in Winnipeg. The discharge through the control structure and the tailwater level therefore remain relatively constant. These conditions of increasing headwater level and a relatively constant tailwater level, result in severe flow conditions in the stilling basin and the downstream channel, even during relatively minor flood events.

The performance of the stilling basin is influenced by the head drop across the control structure, defined as the difference between the upstream and downstream water levels. If the head is too high for the given tailwater condition, the hydraulic jump will be forced downstream, and under severe conditions, it can be swept out past the end of the stilling basin. See photographs 5, 7, and 10.

The observed conditions at the inlet structure for seven of the 16 years of operation of the control structure were analysed in detail to assess the performance of the stilling basin and to determine flow velocities at the rip rap in the channel downstream of the inlet control structure. The flood events prior to 1974 were not reviewed due to insufficient data for those years.

- **Upstream and downstream water levels** - The upstream and downstream water levels were used to determine the total head drop across the control structure. The head drop is an indication of the potential energy which prevailed over the duration of the operation.

  Figure 5.2(a) to 5.2(g) illustrate the water levels measured during the flood years 1974, 1975, 1976, 1978, 1979, 1986 and 1995, respectively. As shown on these Figures, the upstream water level varies considerably during the period of operation, the downstream water level remains relatively constant at approximately elevation 750 ft. In 1974, for example, the upstream water level rose from its initial elevation of 751 feet on April 18, to a peak elevation of 765 feet on April 24, and then receded to an elevation of 751 feet on May 19. The downstream water level remained within the range from elevation 749 feet to elevation 750 feet for nearly the entire duration.

- **Discharge through Control Structure** - Figures 5.3(a) to 5.3(g) illustrate the recorded discharge through the control structure.

- **Gate tip elevation** - The elevation of the gate tip provides an indication of the submergence of the flow at the gate. Flow conditions at the gate remain essentially free flow, or unsubmerged, for tailwater depths less than 70% of the depth of the upstream level above the gate. The recorded gate tip elevations are shown in Figures 5.4(a) to 5.4(g).

- **Flow velocity downstream of the control structure** - The velocity of the water at the exit of the stilling basin was estimated from the observed discharges and the computed depths of flow. The depth of the water jet at the downstream end of the stilling basin was computed using the observed upstream water level and the observed flow patterns recorded in the model study report. The model study results were also used to estimate the dispersal of the jet downstream of the stilling basin.

  Figure 5.5 shows an envelope curve of estimated maximum velocities downstream of the stilling basin measured in the hydraulic model. The velocity is approximately 22 feet per second for the first 50 feet downstream of the structure. The velocity measured in the model reduces to approximately 15 feet per second at a distance of 100 feet downstream of the control structure and is about 13 feet per second 300 feet downstream of the structure.

  The velocity in the channel downstream of the control structure was computed for the seven years of operation considered in this study. Figures 5.6(a) to 5.6(g) illustrate the respective computed velocities at locations 0 to 70 feet, 70 to 120 feet and 120 to 300 feet downstream of the control structure. The computed flow velocities are shown to be in the range from 18 to 20 feet per second for the first 70 feet downstream of the structure. The velocity is reduced to 10 to 12 feet per second 120 feet downstream and to 8 to 10 feet per second in the distance from 120 to 300 feet downstream of the structure.

- **Stable Rip Rap Size**

The stable rip rap sizes for the velocities computed above, were determined from the Isbash formula,

$$D_{50} = \frac{V^2}{C^2(2g)((\gamma_s - \gamma_w)/\gamma_w)}$$

where:

D_{50} = Rip rap diameter in feet (50%)
C = Isbash coefficient
g = Acceleration of gravity, ft/sec^2
γ_s = Specific weight of stone, lb/ft^3
γ_w = Specific weight of water, lb/ft^3

Figure 5.7 illustrates the relationship between velocity and rip rap size (D_{50}) for Isbash coefficients of 0.86, 0.93, 1.0 and 1.2. An Isbash coefficient of 0.86 is recommended by the US Army Corps of Engineers for high turbulence stilling basins. A coefficient of 0.93 was assumed for this analysis, since the flow conditions are not considered to be excessively turbulent. The computed rip rap sizes required to prevent erosion corresponding to the computed velocities is shown in Figures 5.8(a) to 5.8(g). The stable rip rap size for the flow velocities experienced in the 7 years examined and for the maximum velocities defined in the hydraulic model study are shown in Table 5.2, below:

Table 5.2
Stable Rip Rap Size

Distance Downstream of Structure (ft)	Stable Rip Rap Size (ft) - D_{50}	
	Computed Historic	Model Study
0 to 70	3.8	5.4
70 to 120	1.6	4.0
120 to 300	1.2	2.0

- **Froude Number for Flow Entering Stilling Basin**

The Froude Number corresponding to the flow entering the stilling basin can be used as an index of the performance of the stilling basin. Extensive model testing by the US Bureau of Reclamation identified four Froude criteria for the performance of stilling basins as described below:

Froude No. less than 1.7 - When the Froude number is unity the water is flowing at critical depth and a hydraulic jump cannot form. For Froude numbers between 1 and 1.7, the surface of the water exhibits a slight ruffle. As the Froude number approaches 1.7, a series of small rollers develop on the surface.

Froude No. between 1.7 and 2.5 - The action of the water remains similar to conditions less than 1.7. However, the turbulence increases up to a Froude number of 2.5. The velocity through the cross section remains fairly uniform. The energy loss in the poorly formed hydraulic jump is less than 20 percent.

Froude No. between 2.5 and 4.5 - When the incoming water has a Froude number in this range, the hydraulic jump is characterized by a pulsating action where the entering jet oscillates from the bottom of the channel to the water surface and has no regular period. Turbulence can occur on the bottom in one instant and entirely on the surface the next. Each oscillation produces a large wave of irregular period. These turbulent conditions can result in damage to rip rap and to unprotected earth embankments. The recommended length of the stilling basin to contain the turbulent flow is approximately six times the downstream water depth when the incoming jet has a Froude number in this range. The energy loss in the jump is typically less than 30 percent.

Froude No between 4.5 and 9 - The hydraulic jump for incoming flow in this range is well balanced and the stilling action is the optimum. The energy absorption ranges from 44 to 70 percent.

Figures 5.9(a) to 5.9(g) illustrate the computed Froude numbers for the performance of the hydraulic jump. The Froude numbers are shown to lie in the range from 3 to 4 for the seven years of operation analysed in this study. For these Froude No. conditions, the incoming high velocity jet will impact on the bottom for a considerable distance downstream of the control structure. Since only about 30 percent of the energy of the incoming flow is dissipated in the hydraulic jump, the turbulent flow will extend far downstream of the control structure.

- **Energy Loss in the Stilling Basin**

The energy losses in the hydraulic jump for flow conditions experienced at the inlet structure are illustrated in Figures 5.10(a) to 5.10(g). The estimated energy losses for the majority of the flow events is between 30 and 40 percent.

5.5 EROSION PROCESS

The flow conditions causing the erosion of the rip rap in the channel downstream of the floodway inlet control structure were defined from an assessment of the hydraulics as described above, from information presented in the hydraulic model studies report, from a review of photographs taken during the previous floods, and from an examination of the underwater surveys of the rip rap and the contour survey of the river bed conducted this year.

The flow conditions downstream of the control structure conducted from this review are summarized below:

- The water jet leaving the control structure gate for certain flow conditions will plunge over the end of the concrete stilling basin and impact the downstream channel bed. This condition is illustrated in the photograph of the sectional view of the hydraulic model. The water jet at this point is clearly defined without any dispersal and is shown to have a depth approximately equal to the critical depth above the gate. (See Figure 5.11).
- The abrupt expansion of the flow from the narrower control structure into the wider river channel is accentuated by the 90^0 flare to the downstream wing walls. This contributes to the formation of strong return currents along each river bank. These currents deflect the water jet from the structure toward the centre of the channel. Strong vertical vorticies form on the boundary between the high velocity channel flow and the relatively quite water along the river bank. These vertical vorticies extend down to the river bed and insure that highly turbulent flow conditions occur on the bottom (See Photograph 2).
- The low Froude number of the incoming flow from the gate is typically in the range from 3.0 to 4.0. These Froude number conditions are characterized by high turbulence with the water jet fluctuating alternatively from the water surface to the river bed. The turbulent flow is estimated to extend for a distance of 150 to 200 feet downstream of the inlet control structure.
- The underwater reconnaissance program identified significantly larger sized rip rap than specified in the original design. While this material would be sufficiently large to resist

movement due to the high flow velocity, its size would be too large for the design depth of the rip rap layer. It is speculated that large voids in the rip rap layer resulted in either an exposure of the finer underlying bed material or these voids were filled with smaller sized rip rap which would be easily displaced by the high velocity flow. Since the estimated stable rip rap size for the first 50 feet downstream of the structure is between 3.5 and 4.0 feet for the conditions experienced, the smaller material would have been eroded thus exposing the underlying fine bedding material to potential erosion.

In summary, the scour of the river bed has been shown to have progressed in every survey conducted since 1974. From our investigation, it has been concluded that the scour has occurred as a result of several factors. The primary causes of the erosion were that the design of the stilling basin was too short to contain the high velocity turbulent flow and the design of the rip rap protection was inadequate to protect the river bed for the hydraulic conditions to which it was subjected. Erosion of the rip rap occurred as a combination process involving the loss of the smaller sized rip rap due to the high velocity flow and the erosion of the bed material in the areas where the bed material was exposed to the flow. The process was exacerbated by the fact that some of the rip rap placed was too large for the design thickness of the rip rap protection. This prevented overlapping of the stones and allowed voids to occur in the rip rap cover. The exposed rip rap bedding was easily eroded in these locations and resulted in the subsequent lowering of the larger boulders.

Nevertheless, the erosion has been shown to be a gradual process. The most significant area of erosion is in the area approximately 150 feet downstream from the structure. The scour in this area, even it were to continue, will not endanger the concrete structures or the earth embankments. The area nearest the structure is of more concern. The scour in this area has shown a continual progress with the passage of each flood. At this time, it appears that the scour has not stabilized and the scour will continue. Based upon examination of past surveys, the increase in depth of scour from any one flood has not been substantial. Continued downward erosion of the bed near the wing walls will contribute to minor sloughing of the earth fill which could endanger the earth embankment if not maintained continuously.

6.0 STABILITY OF STRUCTURES

In consideration of the potential impact of the scour downstream of the structure, an assessment of the concrete and earth dam stability was conducted for existing and projected long-term scour conditions. The scour hole downstream of the structure is approximately 15 ft deep. In addition to these conditions, an extreme assumption of a 30 ft scour hole was assessed for the concrete structures. For the earth dam, alternate depths of the scour at the toe of the embankment of 15, 20 and 25 ft were considered. The stability of the concrete structures is addressed in Section 6.1 while the embankment section is addressed in Section 6.2.

6.1 STRUCTURE STABILITY

6.1.1 Arrangement of Structures

The concrete section of the inlet structure consists of the central pier with the control room on the upstream side of the pier, two gate sections, each with a single submerged gate, and wing walls at both end of the structure shown on Figure 6.2. The structure is founded on the existing bedrock at the approximate depth of 40 ft (12m) below the original river bed. The evaluation of structure stability was conducted by analysing the structure as a single unit and by considering sections of the structure as independent blocks.

6.1.2 Stability of existing Structures

Stability of the inlet structure was examined in accordance with the design criteria outlined below. The criteria is consistent with the Canadian Dam Safety Guidelines (1995) and considers flotation, sliding, location of the resultant force and the maximum allowable base pressure. On this basis, the structure has been assessed for the most unfavourable combination of loads and scour assumptions under normal and extreme operating conditions. The following load parameters were considered:

Water Levels

Table 6.1 shows the water elevation upstream and downstream from the inlet structure for maximum flood, 160-year (design flood), 50-year, and 2-year return periods for assumed minimum and maximum flow contributions from the Assiniboine River to the flood flow at James Avenue.

Table 6.1
Water levels Upstream and Downstream of the Red River Inlet Structure

Return Period	Upstream Water Level [ft]		Downstream Water Level [ft]	
	Minimum Flow Contribution from the Assiniboine River	Maximum Flow Contribution from the Assiniboine River	Minimum Flow Contribution from the Assiniboine River	Maximum Flow Contribution from the Assiniboine River
Maximum Flood	778.1	-	-	762.7
160	770.2	769.5	761.5	759.7
50	767.9	768.0	756.9	754.5
2	751.0	745.0	749.6	744.0

Normal operating conditions considered the hydrostatic pressure to be calculated for the high upstream and low downstream water level on the basis of the worst historic combination of upstream and downstream water levels. In this respect, the governing hydraulic conditions occurred on April 24, 1974 (Red River Floodway Operation Summary) when the upstream and downstream water levels were 764.94 ft (233.15 m) and 746.99 ft (227.68 m), respectively. This combination of the upstream and downstream water levels represents the most extreme condition on record. However, for the calculation of the uplift force, water levels corresponding to minimum Assiniboine River contribution were selected. These assumptions for the downstream river levels produced the most conservative results.

Extreme conditions considered the maximum flood condition identified in the model studies.

Unit Mass

Structural concrete . . . . 150 lb/ft^3
Water . . . . 62.4 lb/ft^3
Granular Backfill (saturated) . . . . 105 lb/ft^3

Ice Loads
The structure is not subject to ice forces at the extreme water level since the ice floes from the winter ice cover will move well before the high river stages occur.

Earthquake Loads
The project is not located in an earthquake risk area. Earthquake loading is therefore not applicable for the analysis of the stability of the floodway inlet structures.

Earth Loads
For gravity structures founded on rock or similar material, where movement of the structure due to rotation, deflection or foundation yielding is not expected to reach or exceed 1/1,000 of its height, the at rest pressure P_O, shall be used as defined by the relationship:

$P_O = K_O GH$

where:

K_O = at-rest pressure coefficient = 1 - sin Ø

Ø = angle of internal friction of soil

G = unit weight of soil (saturated and submerged are treated separately), and

H = height of soil in metres

This pressure is assumed to vary linearly from zero at the top of the soil to P_O at the base. The resultant force due to such pressure shall be assumed to act at 0.33 H above the base.

At rest soil pressures were considered for the upstream end and for the downstream part of the structure as well. Furthermore, two scour hole depths of 15 ft and 30 ft, were assumed at the downstream toe of the inlet structure, representing the present and some assumed future condition, respectively.

6.1.3 Limiting Criteria

The sections analysed for stability were the central pier, gate section, and wing wall sections with the results shown on Figure 6.3. The structure was analysed as a monolith and as independent structural blocks. Results are presented in Table 6.2 and Table 6.3 for the case of 15 ft and 30 ft deep scour hole at the foot of the structure, respectively.

Table 6.2
Summary of Limiting Criteria for 15 Foot Deep Scour Hole at the Toe of the Structure

Criteria	Limiting Criteria	Computed Safety Factors, Location of Resultant Force and Base Pressure					
	Normal[1] — Extreme[2]	Pier Section	Gate Section	D/S Gate Section	Wing Wall Section	D/S Wing Wall Section	Entire Structure
Flotation[3]	1.5 1.3	2.0 1.8	1.7 1.6	1.8 1.6	1.9 1.7	2.5 2.0	1.8 1.6
Sliding[4]	1.5 1.3	5.0 4.6	3.4 3.2	3.8 4.3	2.8 2.2	3.3 2.6	2.9 2.4
Overturning[5]	1.5 1.3	1.9 1.7	1.5 1.4	1.5 1.4	1.7 1.5	1.6 1.3	1.7 1.6
Location of[6] the Resultant Force [ft]	Within Kern	1.0 22.7	12.2 22.7	12.7 13.9	5.5 30.0	5.5 9.0	12.9 30.1
	Within Mid-half	0.1 34.0	14.0 34.0	12.0 20.9	13.0 45.1	7.9 13.6	17.7 45.1
Max. Base Pressure [Ksf]	40.0 60.0	7.9 6.9	5.3 5.4	6.5 6.4	5.8 6.4	10.3 10.0	5.6 5.8

Notes:

1 *"Normal" refers to all loading cases in which normal uplift (UN) occurs. Normal uplift was assumed to include most severe conditions on record (ie: 1974)*

2 *"Extreme" refers to conditions in which extreme uplift (UE). Extreme uplift was assumed to consider the condition under maximum design flood condition.*

3 *Minimum Floatation Factors of Safety is equal to V/U.*

4 *Minimum Sliding Factor of Safety is equal to* $\frac{(V-U)\tan Ø}{H}$

where:

- *V = Sum of vertical downward forces*
- *U = Sum of vertical upward forces*
- *H = Sum of horizontal forces*
- *Ø = Angle of internal friction for foundation material*
- *A = Effective area*

5 *Overturning criteria is specified as a ratio of sum of moments that tend to stabilize to the sum of moments that tend to overturn the structure.*

6 *Location of the resultant force measured from the centre of the structure should be less than the limiting criteria which is given for each structure block separately. The results are shown as follows:* $\frac{\text{Distance from centre}}{\text{Distance to Kern}}$ *for normal conditions, and* $\frac{\text{Distance from centre}}{\text{Distance to mid-half from centre}}$ *for extreme conditions*

Table 6.3
Summary of Limiting Criteria for 30 Foot Deep Scour Hole at the Toe of the Structure

Criteria	Limiting Criteria	Computed Safety Factors, Location of Resultant Force and Base Pressure					
	Normal[1] -- Extreme[2]	Pier Section	Gate Section	D/S Gate Section	Wing Wall Section	D/S Wing Wall Section	Entire Structure
Flotation[3]	1.5 1.3	2.0 1.8	1.7 1.6	1.7 1.6	1.9 1.7	2.5 1.9	1.8 1.6
Sliding[4]	1.5 1.3	4.8 4.3	3.1 2.0	3.1 3.7	2.7 2.1	3.0 2.2	2.8 2.1
Overturning[5]	1.5 1.3	2.0 1.5	1.5 1.3	1.5 1.4	1.7 1.5	1.6 1.3	1.7 1.6
Location of[6] the Resultant Force [ft]	Within Kern	1.3 22.7	12.7 22.7	13.0 13.9	6.0 30.1	6.0 9.0	13.3 30.1
	Within Mid-half	0.4 34.0	24.4 34.0	12.6 20.9	13.5 45.1	9.3 13.6	21.3 45.1
Max. Base Pressure [Ksf]	62.0 83.0	8.1 6.9	5.4 6.7	6.6 6.6	5.9 6.5	10.6 10.1	5.7 6.2

Notes:

1 *"Normal" refers to all loading cases in which normal uplift (UN) occurs. Normal uplift was assumed to include most severe conditions on record (ie: 1974).*

2 *"Extreme" refers to conditions in which extreme uplift (UE). Extreme uplift was assumed to consider the condition under maximum design flood condition.*

3 *Minimum Floatation Factors of Safety is equal to V/U.*

4 *Minimum Sliding Factor of Safety is equal to* $\frac{(V-U)\tan\varnothing}{H}$

where:

V = Sum of vertical downward forces
U = Sum of vertical upward forces
H = Sum of horizontal forces
Ø = Angle of internal friction for foundation material
A = Effective area

5 *Overturning criteria is specified as a ratio of sum of moments that tend to stabilize to the sum of moments that tend to overturn the structure.*

6 *Location of the resultant force measured from the centre of the structure should be less than the limiting criteria which is given for each structure block separately. The results are shown as follows:* $\frac{\text{Distance from centre}}{\text{Distance to Kern}}$ *for normal conditions and* $\frac{\text{Distance from centre}}{\text{Distance to mid-half from centre}}$ *for extreme conditions*

6.1.4 Summary

From a comparison of the safety factors, it is apparent that the stability of the structures is not sensitive to the depth of scour, and that the structures meet all of the limiting criteria for stability even under extreme scour conditions. If a formal dam safety review is conducted some time in the future, these values should be re-evaluated for that specific purpose.

6.2 EARTH DAM EMBANKMENT ASSESSMENT

6.2.1 Introduction

The stability conditions of the embankment section adjacent to the downstream side of the inlet control structure abutments were assessed. The lower and overall slopes were evaluated to determine the existing stability conditions, the effect of potential additional scour on slope stability, and the remedial measures to improve the stability conditions. The location and geometry of the cross section analysed at the northeast abutment are shown on Figures 6.1 and 6.4.

The existing stability was assessed based on a two dimensional stability analysis. The analysis consisted of a back analysis of the existing geometry to determine representative strength parameters. These parameters were then used to assess the effects of potentially additional erosion of the base from elevation 715 ft to elevation 710 ft.

Remedial works to improve the stability were evaluated, including slope flattening of the lower slope area and erosion protection. The stratigraphy used in this analysis was taken from the Department of Agriculture and Conservation floodway inlet control drawings, dated November 25, 1963.

6.2.2 Existing Geometry

The existing geometry below the river level was estimated from the surveyed erosion depth within the channel and from the approximate location of the failure scarp relative to the summer river level. The elevation of the lower slope area at the abutments was estimated from a projection of the survey results in the main channel area. The slope was estimated to be approximately 1H:1V from elevation 736 ft to the maximum scour depth surveyed in the channel at elevation 715 ft, as shown in Figure 6.4. For a safety factor, FS, of unity for slip surface 1 and the winter river level, which are representative of the existing failure conditions, the required strength parameters for the rock fill material (Classes 3, 5, 6, and 9) were determined to have an effective friction angle, $\phi' = 44^\circ$ and effective cohesion, $c' = 0$. These parameters are considered representative of a rock fill material, consistent with the material shown on the design drawings. The slip surface is relatively shallow, which is typical for cohesionless materials and consistent with the observed failures. The estimated FS for the overall potential slip surface 2 is 1.48, as shown in Table 6.4. Continued unravelling and shallow lower slope movements can be expected if the slope is not remediated.

6.2.3 Continued Erosion

The stability conditions of the slope, assuming that the erosion of the base was allowed to continue to both elevation 710 and elevation 700 ft, were assessed to determine the impact on the estimated FS. It was assumed that the lower slope would be maintained at 1H:1V, with the toe of the slope vertically below the assumed existing toe. Assuming the base at elevation 710 ft, the estimated FS for the lower bank slip surface 1 is 1.0, which suggests continual shallow unravelling. The overall potential slip surface 2 has an estimated FS of 1.27, which is a 0.21 decrease from the existing conditions. For a base of slope at elevation 700 ft, the majority of the slope would be at 1H:1V, with potential slip surfaces 1 and 2 being similar in geometry. The estimated FS was 1.0 for slip surface 1 and 1.06 for slip surface 2.

6.2.4 Remedial Measures - Backfill Lower Slope At 2H:1V

To improve the stability conditions and prevent continued unravelling of the lower slope, backfilling was assessed at a 2H:1V slope from the channel bottom to the failure scarp near summer river level. The estimated FS of both the lower and overall slope potential slip surfaces was greater than 1.7 for the winter river level conditions. Adequate erosion protection of the surface, to prevent scour of the bottom, must be provided to prevent a repeat of the existing condition with unravelling instability.

As long as no remedial measures are undertaken, continued unravelling failures consistent with those previously and currently observed can be expected. If the erosion of material from the toe of the slope continues, the shallow failures will progress until the slope reaches a minimum of 1H:1V. In the event that base erosion continues to elevation 700 ft, the stability of the entire embankment from near the top of the wing wall (elevation 765 ft) would be in jeopardy of shallow slope failures. No effect is anticipated on the impervious core, which is approximately 100 feet from the wing walls.

If the material is replaced to flatten the slope to 2H:1V, but adequate erosion protection can not be maintained, movements similar to those already observed can be expected. It is, however, unlikely that during any one flood event, failures would be more extensive than those already observed, providing that the slope was flattened to 2H:1V prior to the flood. If adequate erosion protection can not be provided, the abutments should be inspected during and after each flood event and replacement of the rockfill should be undertaken if erosion and slope failures are observed.

6.2.5 Summary of Factors of Safety

A summary of the estimated factors of safety for all the analyses is shown on Table 6.4.

Table 6.4
Summary of Estimated Safety Factors

Case		Slip Surface	Estimated FS	Change
1.	Existing Geometry - Base Erosion at El 715 ft, Lower Slope at 1H:1V	1 - Lower 2 - Upper	1.0 1.48	- -
2.	Additional Erosion of Base to El 710 ft., Lower Slope at 1H:1V	1 - Lower 2 - Upper	1.0 1.27	- -0.21
3.	Additional Erosion of Base to El 700 ft, Lower Slope at 1H:1V	1 - Lower 2 - Upper	1.0 1.06	- -0.42
3.	Backfill Lower Slope to 2H:1V to Existing Bottom, and Provide Erosion Protection	1 - Lower 2 - Upper	1.73 1.78	+0.73 +0.30

6.3 DOWNSTREAM EMBANKMENT STABILITY

The stability of the embankment slopes downstream of the inlet structure along the sides of the river was also assessed relative to the erosion. The stability conditions were evaluated based on visual inspection and engineering judgement. The embankment slopes were constructed at 6H:1V, which is significantly flatter than the 2H:1V adjacent to the wing walls. However, the stratigraphy consists of the natural clay soils with rip rap protection. The clays have much lower strengths than the rockfill, and the 6H:1V side slopes are appropriate to maintain stability. To date, no failures have been observed, and the erosion at the base of these slopes has been limited. Continued visual monitoring is recommended to ensure failures are not being initiated.

7.0 REMEDIAL ALTERNATIVES

7.1 INTRODUCTION

Alternatives which were considered to address the erosion problem in the riverbed included:

- extending the length of the concrete stilling basin to contain the high velocity turbulent flow on a concrete apron
- the use of rip rap to fill the scour hole
- the use of rock filled gabion structures, and
- the use of concrete mats constructed underwater to line the scour hole.

The use of gabions in high velocity flow downstream of the floodway inlet structure was rejected since rock fill in the gabion tends to be dragged to the downstream end of the baskets by the high velocity flow. This could expose the underlying finer bed material.

The extension of the stilling basin was not considered practical due to the high cost and difficulties associated with dewatering the area.

The site conditions at the inlet control structure limits the choice of methods available for the protecting the river bed with rip rap or with concrete mats. The normal summer river level at the inlet structure is controlled by the St. Andrews Dam at Lockport at approximately elevation 735 feet. The water level drops marginally during late October and early November, after the gates at the St.

Andrews Dam are opened. However, except for extremely low flow conditions, the water level remains relatively high at approximately elevation 732 feet. Consequently, the river bed is submerged by up to 20 feet of water even under the low draw down conditions. The placing of rip rap on the river bed, or other protection schemes such as concrete mats, would therefore have to be undertaken "in the wet".

7.2 REPLACEMENT OF RIP RAP ALTERNATIVE

7.2.1 Rip Rap Size

Restoration of the rip rap cover would require rip rap of sufficient size to withstand not only the range in velocities experienced in the past 26 years of operation, but also the maximum possible velocity resulting from all combinations of possible flood flows. The envelope curve of maximum velocities which was determined in the hydraulic model studies is shown in Figure 5.5. The required rip rap size (D_{50}) required to withstand these velocities is given in Table 7.1. The maximum (D_{100}) and minimum size (D_{15}) of rip rap required to provide the required gradation is also shown in Table 7.1. The locations of the rip rap zones referenced in Table 7.1 are illustrated in Figure 7.1.

Table 7.1
Rip Rap Size

Zone	Distance D/S of Structure (ft)	D_{50} (ft)	D_{100} (ft)	D_{15} (ft)	Layer Thickness (ft)
A	0 to 70	5.4	6.75	2.2	10
B	70 to 120	4.0	5.0	1.5	8
C	120 to 250	2.0	4.0	0.8	4

The depth of the scour below the design grade is less than the required thickness of riprap layer in many locations. Placing the rip rap to the depths specified in Table 7.1, would result in the top of the rip rap extending well above the design grade. As a result, fill only to the design grade was considered for these areas. The placement of the rip rap to a maximum of elevation 725 feet was considered to be preferable to placing the rip rap to the specified depth for the following reasons:

- fill above the design grade elevation would expose the rip rap to impact from the high velocity flow and produce locally severe flow turbulence, which may increase the problem of erosion for these areas, and
- those areas which have experienced lesser depths of erosion would also have experienced less severe velocity conditions. The magnitude of the local velocities in these areas would be difficult to determine due to the non uniform turbulent flow conditions. Nevertheless, It is likely that more of the original rip rap still exists for those areas where scour had not progressed to the maximum depth. This would allow a less stringent specification for the rip rap in these locations.

7.2.2 Construction

Replacement of the rip rap was discussed with local contractors to identify practical methods of construction. The distance from the riverbank to the scour hole is in excess of 100 feet which makes placing of the rip rap from the river bank impossible. Methods of placing the rip rap could include the construction of finger groins from the bank and the placing of the rip rap by backhoe or dragline from the groin, or the dumping of the rip rap from a barge. The use of finger groins does not appear to be practical for this case, due to the excessive depth of water at the site. The placement of the rip rap would have to be inspected continuously by divers in order to ensure that the rip rap was not segregated and that voids in the rip rap blanket did not occur.

The requirement for the large size of rip rap was also discussed with local quarry operators. Difficulties were identified in obtaining rock that large in the quarry operation. Other difficulties identified included problems of segregation in the stockpiling and transporting of the rip rap, as well as the lack of available equipment required to transport the large rip rap. These difficulties were reflected in the high unit costs for supply and installation of the rip rap presented below.

7.2.3 Cost Estimate

The summary of the cost for the rip rap alternative is listed in Table 7.2, below:

Table 7.2
Cost Estimate - Rip Rap Alternative

Description	Quantity	Unit Price	Amount
Mobilization/Demobilization	1	$15,000	$15,000
Rip Rap - Supply & Install	7800 yd^3	$100	$780,000
Divers - Inspection	20 days	$1,000	$20,000
Total Direct Costs			$815,000
Engineering			$50,000
Contingencies (20% of Direct)			$163,000
Total Project Cost			$1,028,000

7.3 CONCRETE MAT ALTERNATIVE

7.3.1 Description

The concrete mat alternative consists of the use of fabric forms into which a grout mixture is injected (See Appendix B - Fabriform Technical Data). This scheme involves the following:

- the construction of a smooth granular base for the construction of the concrete mat by placing a layer of granular fill over the existing irregular rip rap surface to fill in the void in the rip rap. The scour hole would not be filled. A layer of geotextile fabric would be used to line the granular base before the installation of the fabric form.
- the placement of the fabric form on the existing scour hole and the filling of the forms with

a concrete grout would be done under water with the aid of divers. Concrete grout mixture would be supplied by pumping from the adjacent river bank.

- A barge would be required to supply the material and provide support facilities for the divers. The mat would be constructed on the existing scour hole.
- A concrete mat having a thickness of approximately 3 feet would be required for the first 70 feet downstream of the structure to provide weight and stability in the high velocity area. The mat would consist of a single layer of concrete bags having dimensions when filled equal to 3 feet deep by 5 feet wide. (See Appendix B - Fabriform Concrete Bags Technical Data). A plan showing the locations where the concrete bags would be required is given on Figure 7.2. At the final design stage, consideration could be given to reducing the mat thickness, thereby significantly reducing the total protection cost. In discussion with suppliers and with the US Corps of Engineers, however, concerns were identified regarding the stability of the thin concrete mat under such high flow velocities. Further research or model studies would be required to verify the suitability of such an approach.
- The uplift forces and the turbulence in the flow are considerably less in the area from 70 feet to 250 feet downstream of the structure. An 8-inch articulating block (AB) mat is proposed for this area. The preparation of the river bed would be similar to that required for the concrete bags described above. Geotextile fabric would be placed on the river bed prior to the placement and filling of the AB-mat.

7.3.2 Cost Estimate

The cost estimate for supply and placement of the mat was based upon discussions with contractors specializing in this type of work. Based upon these discussions, expertise associated with placing and pumping the mats will likely have to come from out of the Province.

The summary of the cost estimate for the concrete mat alternative is listed below in Table 7.2.

Table 7.2
Cost Estimate - Concrete Mat Alternative

Description	Quantity	Unit Price	Amount
Mobilization/Demobilization	lump sum	$15,000.00	$15,000
Divers Inspection	30 days	$1,000.00	$30,000
Fabric form	54,400 ft^2	$1.15	$63,000
Concrete Grout	1,975 yd^3	$140.00	$277,000
Concrete Pump	150 hr	$100.00	$15,000
Granular Base (supply & install)	5,100 yd^3	$20.00	$102,000
Geotextile Fabric (supply & install)	5,100 yd^2	$3.00	$15,300
Total Direct Costs			$517,000
Engineering			$50,000
Contingencies (20%) of Indirect Costs			$103,000
Total Project Cost			$670,000

7.4 PARTIAL REMEDIATION

Given the cost associated with remediation of the entire scoured area, alternatives associated with partial remediation or no remediation were considered. Although the scour hole is not considered to be stable (ie: it has not necessarily reached a limiting depth) the rate of change of growth of the scour hole from flood event to flood event has not been significant. Furthermore, stability calculations associated with the structure and overall earth embankment show that the structures are stable under extreme assumptions of scour. As such, a partial or no remediation alternative and annual monitoring approach has merit. Since the scour process has not stabilized at this time, the partial or no remediation alternative could only be implemented with a commitment to continuous monitoring during and after each significant event.

Historically, the toe of the embankment at the downstream wing wall has been rehabilitated following significant spring events. As described in Section 6.2, local sloughing can be anticipated to be an ongoing process, unless the toe is stabilized. This could involve local riprapping, placing concrete at the toe or sheet piling locally to protect this corner.

In this regard, the most significant concern will be the unravelling of the toe of the embankment as discussed in Section 6.2. Under the extreme loading conditions assumed, it has been shown that this unravelling will not threaten the overall integrity of the earth embankment. Furthermore, the structure stability of the concrete structures is not a concern under these conditions, as described in Section 6.1.

Estimated costs for these alternatives are anticipated to be approximately $50,000 to $150,000, depending upon the degree of protection offered.

8.0 APPROVALS

The Regulatory approvals from three agencies were identified at the proposal stage:

- Manitoba Environment (Environment License)
- City of Winnipeg, River and Streams Authority No. 1 - Rivers and Streams Permit
- Canadian Coast Guard - Navigable Waters Protection Act

A copy of the application for license approval, correspondence with Manitoba Environment and a copy of the advertisement is enclosed in Appendix C. The license was issued on September 6, 1995 and is valid indefinitely providing that the terms of the license are maintained.

Applications for the other agencies, Rivers and Streams Authority and the Coast Guard will be filed when the extent of the proposed works is defined at the final design stage.

9.0 CONCLUSIONS

The operation of the inlet control structure has resulted in the scour of the river bed downstream of the control structure. A review of survey data taken in 1975, 1976, 1979, 1994 and 1995 shows that the deepest scour has occurred in an area downstream of each bay, within 50 feet of the downstream end of the control structure. A significant area of scour has also occurred 150 feet to 250 feet downstream of the control structure. The review of the survey data shows that the scour hole has not reached equilibrium, but has progressively increased in size since 1975. The rate of scour from each flood event has not been significant.

The causes of the erosion of the rip rap identified in this study included:

- the integrated operation of the Red River floodway, the Portage Diversion and the Shellmouth Reservoir during floods produce adverse headwater/tailwater conditions at the inlet which result in severe turbulence and high velocities even under relatively low flood flows.

- the length of the stilling basin allows the water jet to plunge past the end of the stilling basin and impact the rip rap downstream of the structure. For certain conditions of upstream water level and downstream water level, a weak hydraulic jump is possible downstream of the control gates. Under these conditions, the length of the stilling basin is too short, which allows the hydraulic jump to occur past the end of the concrete base slab. The Froude number of the water entering the jump was also shown to be in the undesirable range from 2.5 to 4.0. These conditions result in only a small amount of the energy of the flowing water to be dissipated in the hydraulic jump. As a result, the high velocity flow conditions extend for a considerable distance downstream which results in the erosion of the rip rap in the area 150 feet to 250 feet downstream of the structure.

- the rip rap, as placed, was not adequate for the high velocity, high turbulent flow conditions experienced during the 16 years in which the structure was operated. Specifically, the rip rap, as designed, was too small to be stable under these hydraulic conditions. The design thickness of the rip rap blanket, while sufficient for the size of the rip rap specified in the design, was too small for some of the large boulders which were actually placed. This meant that in some areas, only one layer of rip rap was possible. This allowed voids to be present in the rip rap cover, thus exposing the underlying bed material to the high velocity flow. Because of the highly turbulent flow conditions, the stability and ultimate limits of the scour hole is impossible to predict.

- The stability of the concrete structures and earthfill embankments section was assessed with respect to the impact of scour downstream of the structure. The concrete structure meets the specified limiting criteria under extreme scour assumptions. The earth dam embankment will be subject to unravelling of the lower slope due to scour at the toe. This is not anticipated to threaten the integrity of the dam structure under extreme conditions but should be monitored following the the spring runoff in each year that the floodway is used.

Two alternatives for full remediation of the scour have been identified in this study. These include:

- Placement of larger sized rip rap which would be stable for all possible future flow conditions. The estimated cost is in the order of $1,028,000.

- Lining the scour holes with concrete filled bags and mats. This alternative is estimated to cost $670,000. These costs could possibly be reduced if concrete mats were used instead of the bags. Possible high uplift forces in the region of the hydraulic jump required the use of the larger sized concrete bags rather than the thinner concrete mat alternative. The feasibility of the mat alternative could be investigated further in hydraulic model studies if funds do not permit the construction of the concrete bag concept.

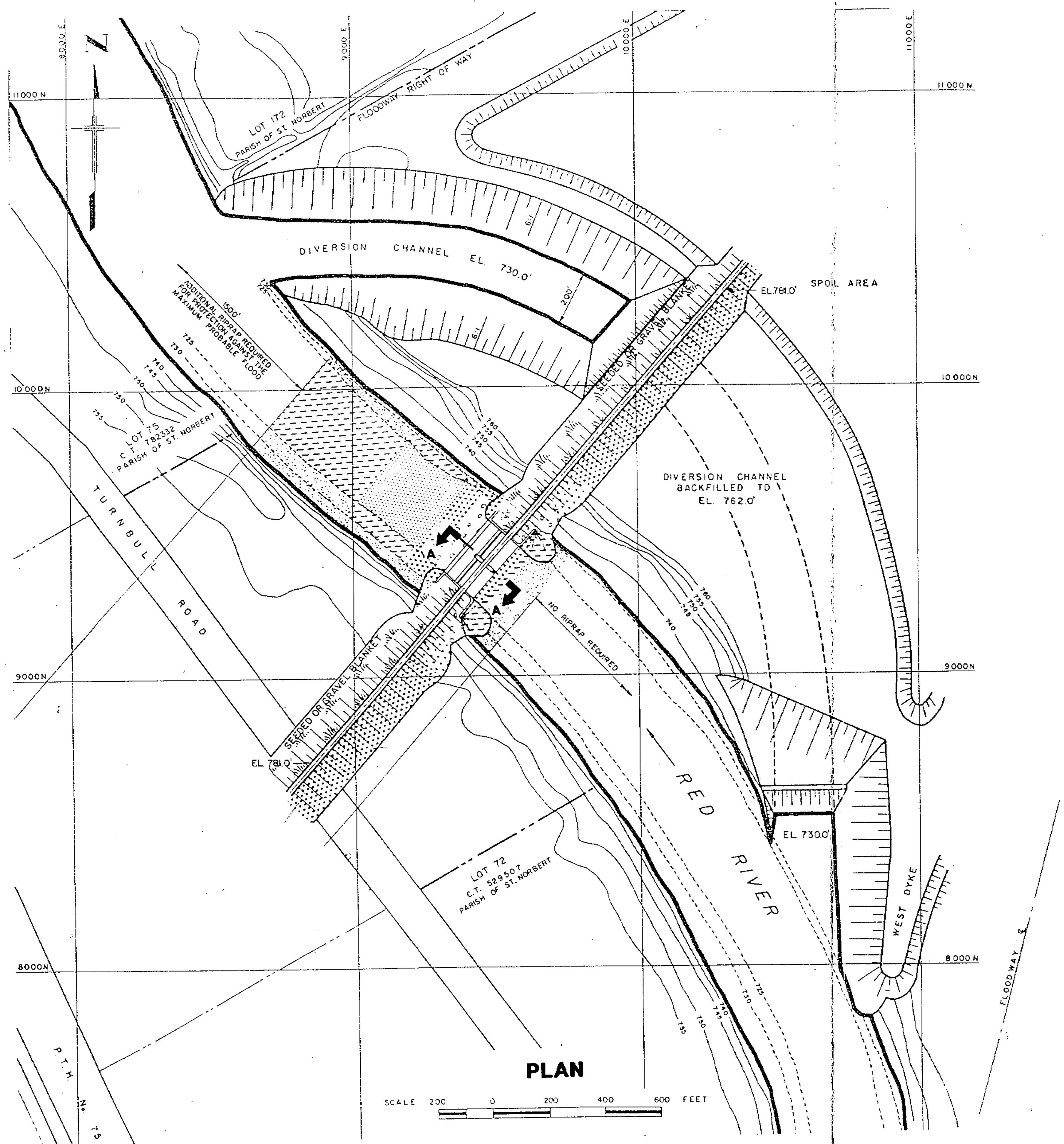
Given the relatively high cost for full remediation of the scour problem, lower cost alternatives associated with annual monitoring have been considered. This scheme would involve the protection of the toe of the embankment at the downstream wing wall to prevent unravelling. Possible methods identified in this study include the annual replacement of the rip rap at the toe of the embankment, the protection of the embankment with concrete filled bags to line the slope, or the use of sheet piles. The estimated cost of the partial solution is approximately of $50,000 to $150,000.

10.0 RECOMMENDATIONS

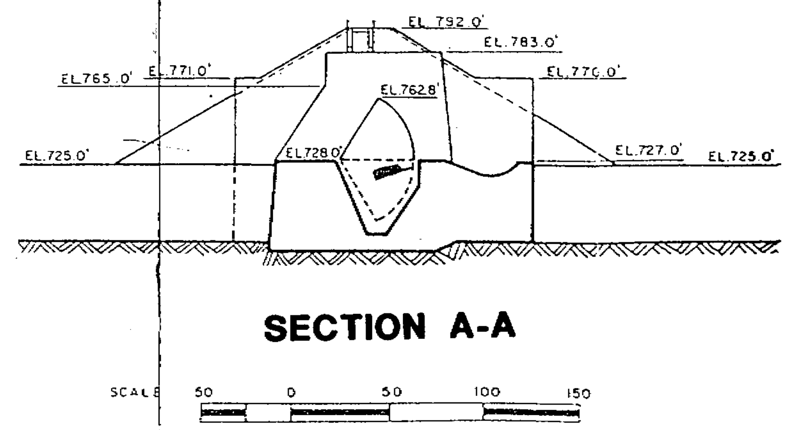
The recommended solution for the total remediation of the scour problem is to line the scour hole with concrete filled bags and mats having an estimated cost of $670,000. Consideration could be given to using concrete mats over the entire area to reduce this cost.

Given that the rate of erosion is not excessively large, the alternative associated with annual monitoring could be pursued. This would require visual monitoring during the flood, followed by a detailed survey of the river bed and the preparation of an annual monitoring report after the passage of the flood.

FIGURES



PLAN



SECTION A-A

KGS GROUP

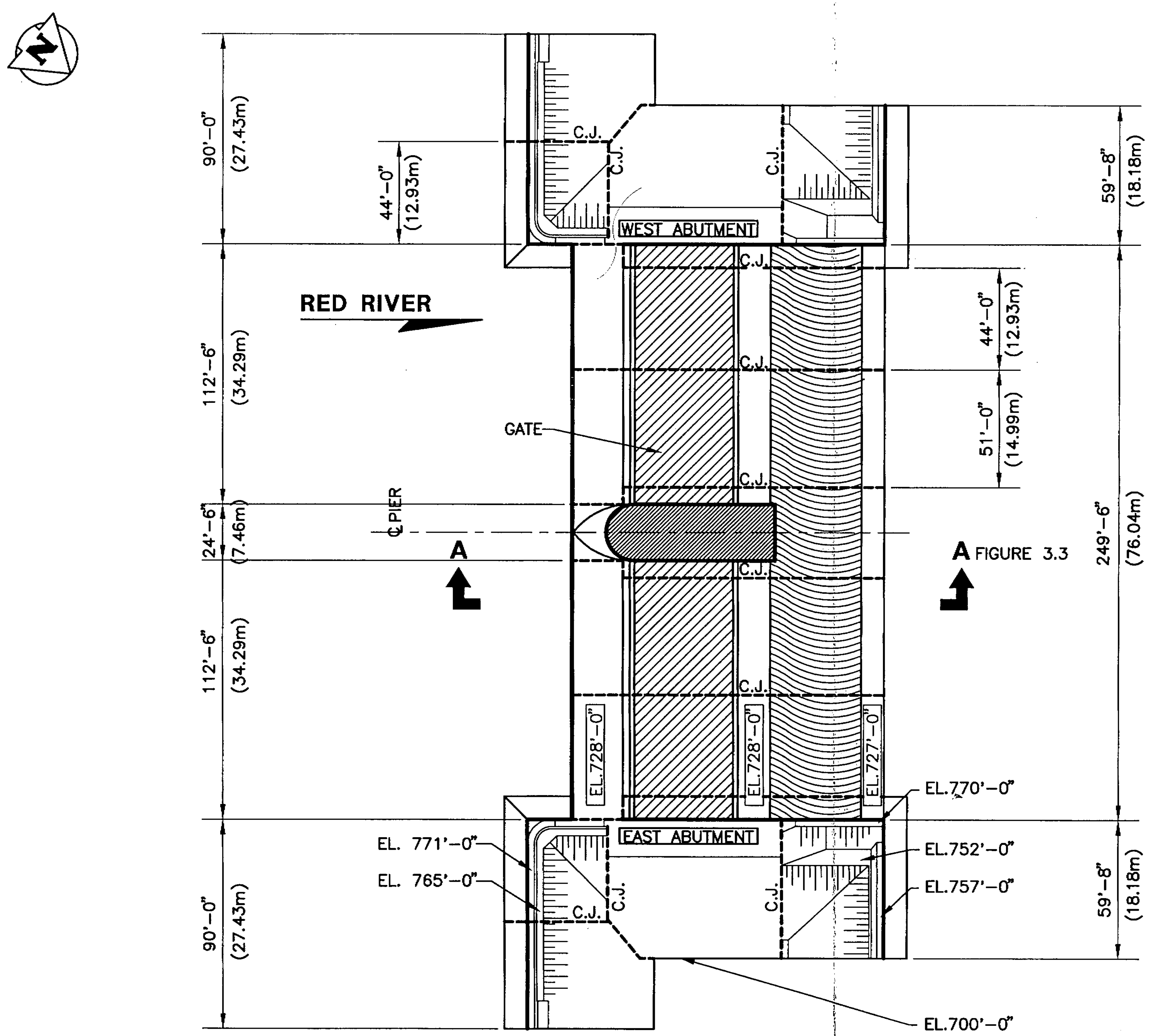
MANITOBA DEPARTMENT OF NATURAL RESOURCES

RED RIVER FLOODWAY - INLET CENTRAL STRUCTURE EROSION STUDY

INLET CONTROL STRUCTURE GENERAL ARRANGEMENT

AUGUST 1995

FIGURE 3.1



LEGEND

C.J. – CONSTRUCTION JOINT

SCALE 50 40 30 20 10 0.0 25 50 100 FEET

KGS GROUP

MANITOBA DEPARTMENT OF NATURAL RESOURCES

RED RIVER FLOODWAY - INLET CENTRAL STRUCTURE EROSION STUDY

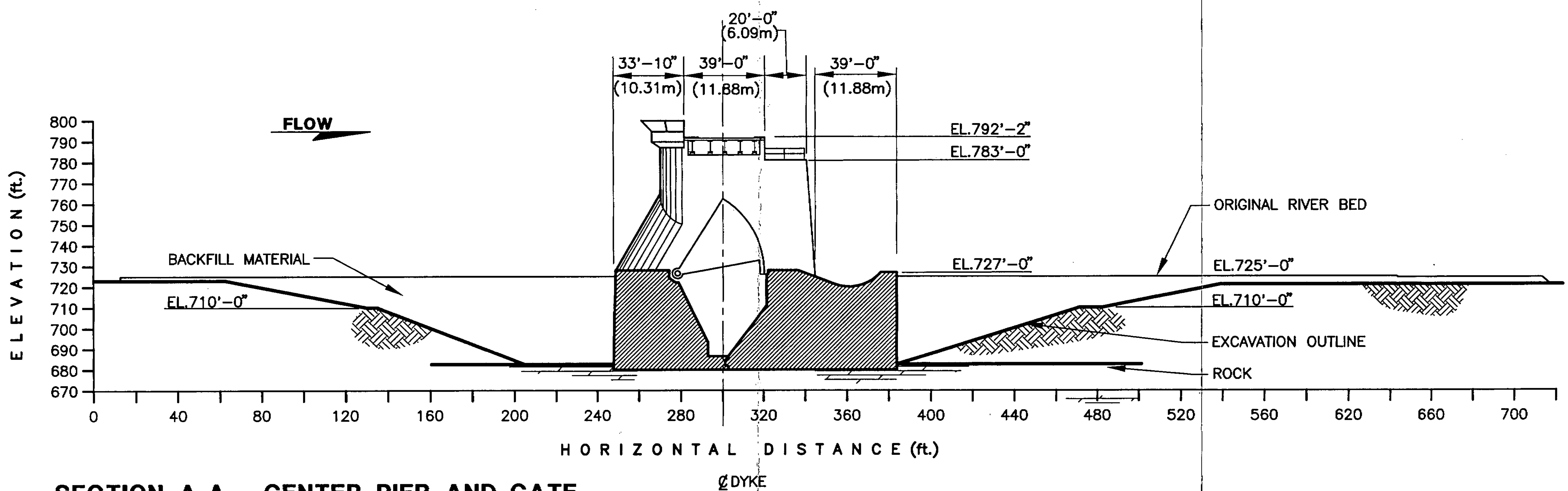
INLET CONTROL STRUCTURE PLAN

AUGUST 1995

FIGURE 3.2

17" X 11"/ PLOT SCALE:1=50

KGS No. 53110198.DWG



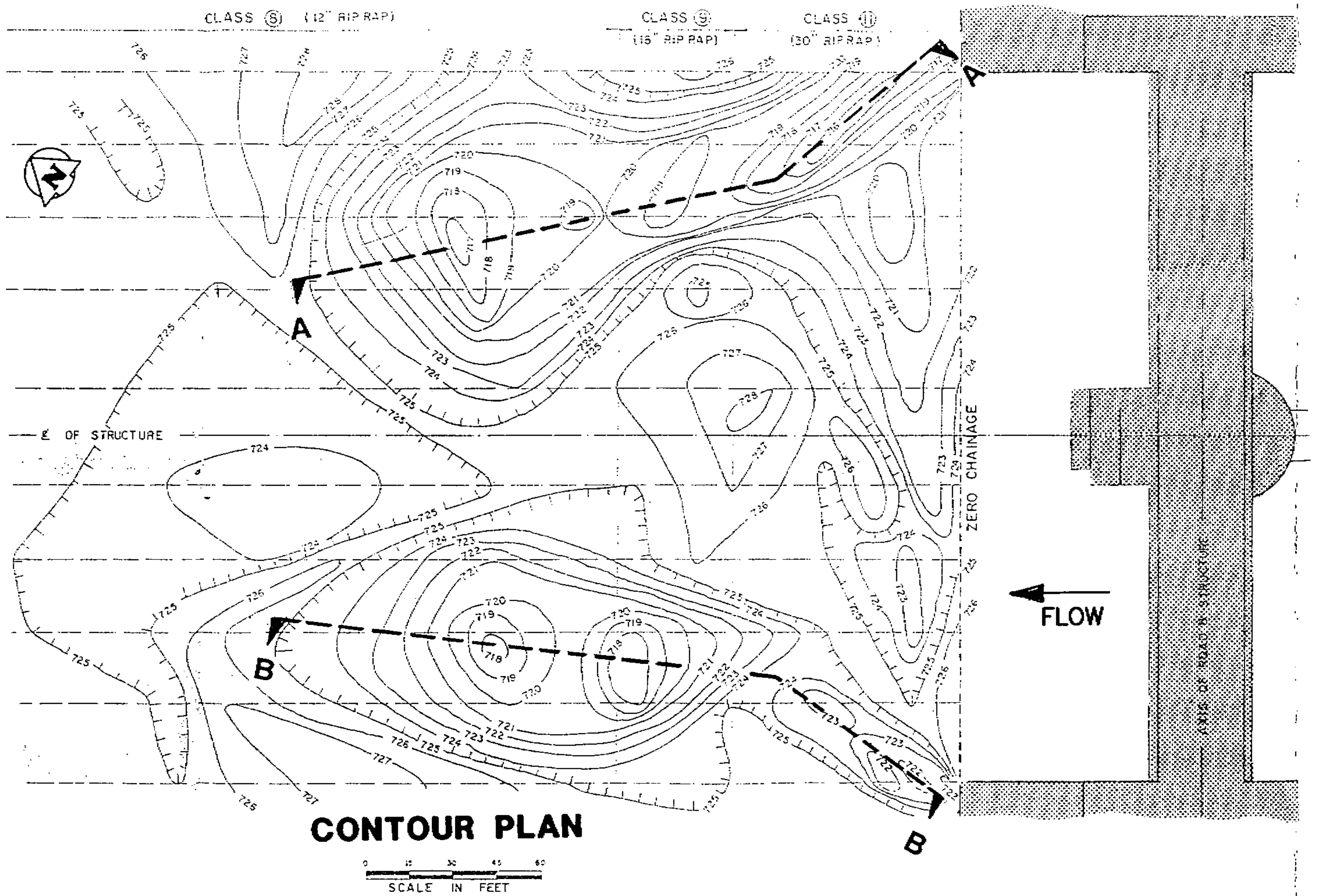
SECTION A-A - CENTER PIER AND GATE

LEGEND
C.J. – CONSTRUCTION JOINT

KGS GROUP	MANITOBA DEPARTMENT OF NATURAL RESOURCES
RED RIVER FLOODWAY - INLET CENTRAL STRUCTURE EROSION STUDY	
INLET CONTROL STRUCTURE SECTION	
AUGUST 1995	FIGURE 3.3

11"x17"/SCALE: 1"=50'-0"

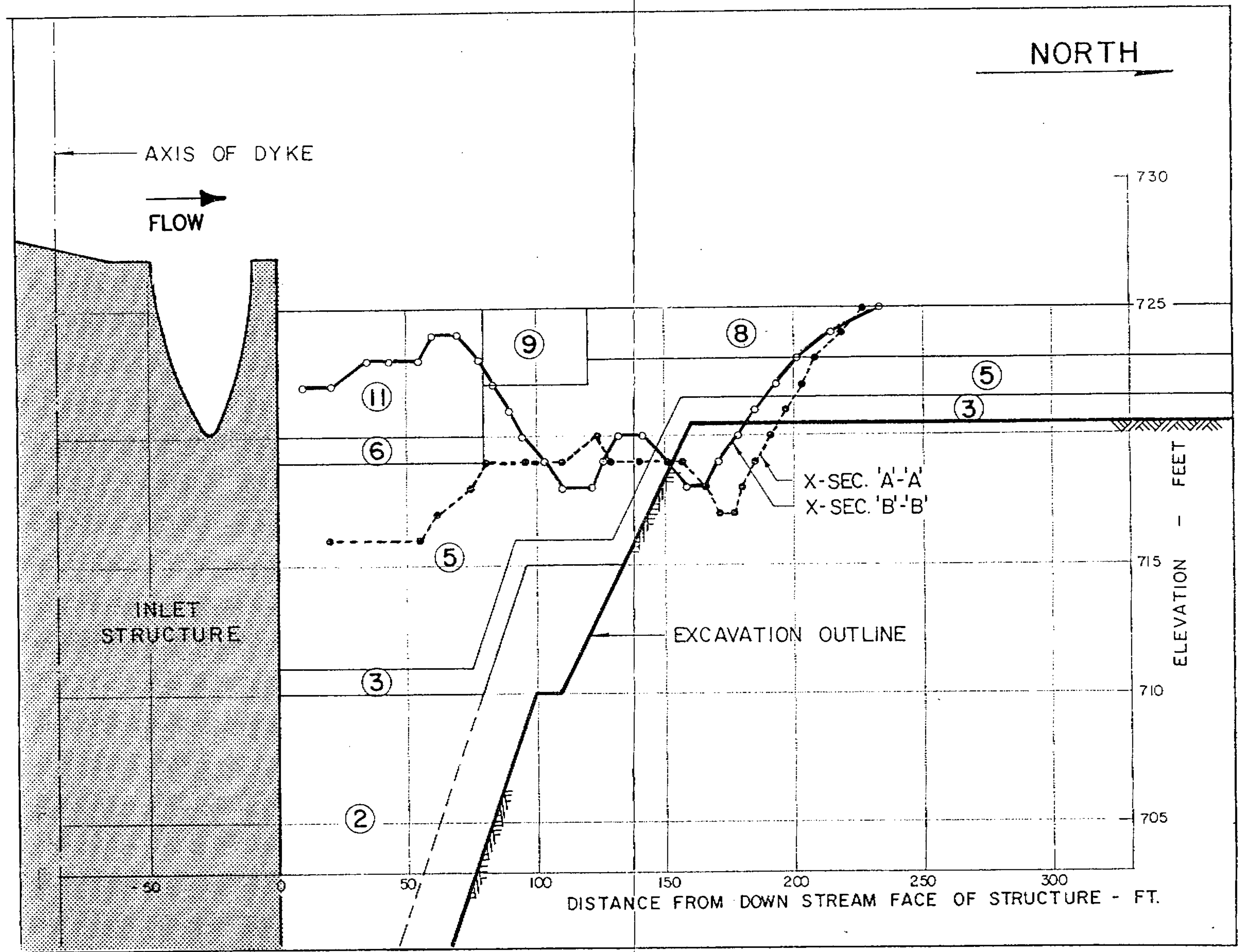
KGS DWG. NO. 53110199.DWG



CONTOUR PLAN

NOTES

1. Erosion depth as shown is based on soundings survey in June 25, 1979
2. Erosion protection to consist of rip rap or erosion mats placed downstream of structure to elevation 725 ft.



SECTION AT CENTERLINE OF STRUCTURE

LEGEND:

CLASS OF MATERIAL	SIZE
11	30" RIPRAP
9	18" RIP RAP
8	12" RIP.RAP
6	6" RIP RAP
5	6" FILTER STONE
3	11/2" FILTER STONE
2	TILL FILL

KGS GROUP

MANITOBA DEPARTMENT OF NATURAL RESOURCES

RED RIVER FLOODWAY - INLET CENTRAL STRUCTURE EROSION STUDY

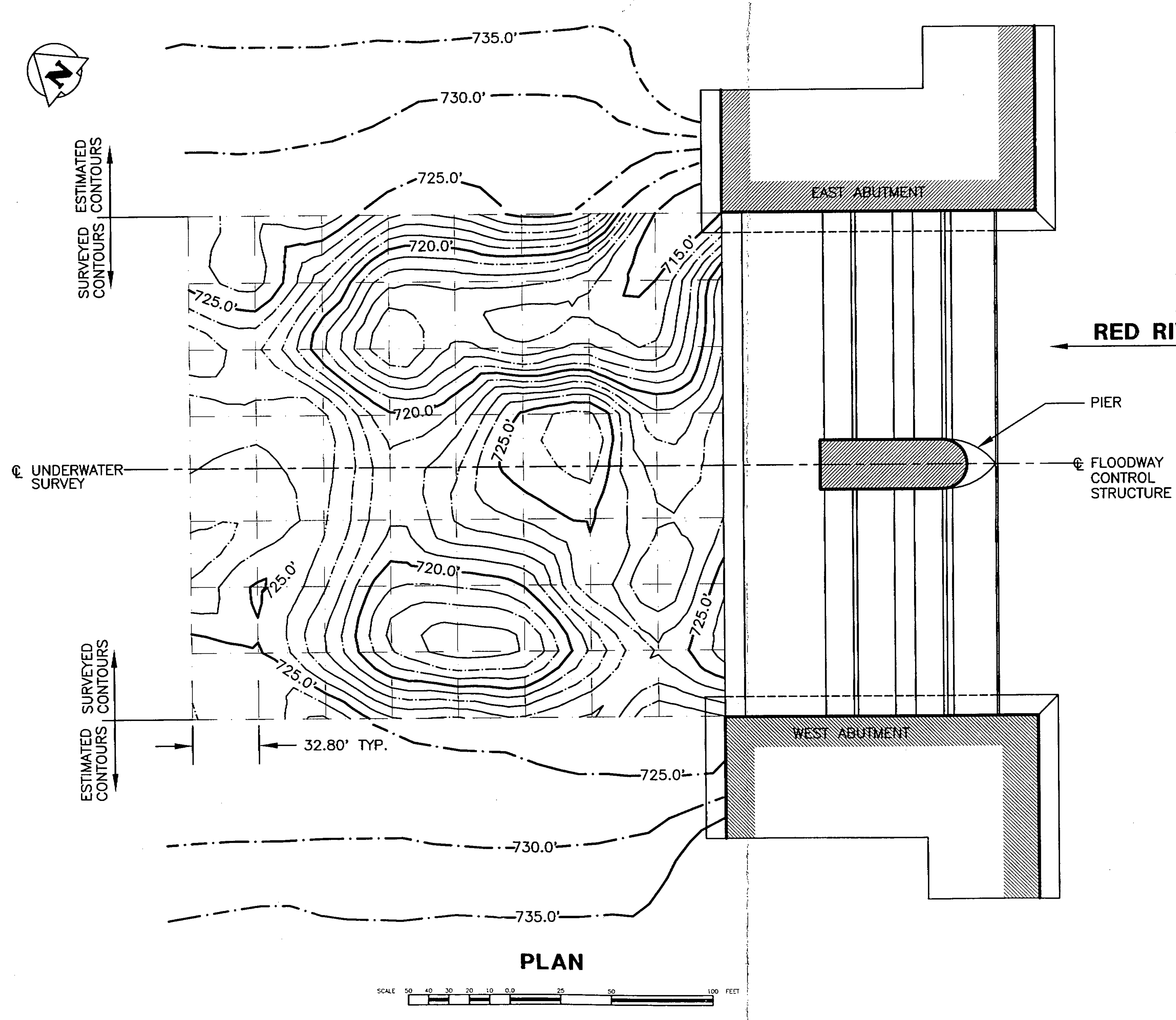
EROSION DOWNSTREAM OF INLET STRUCTURE - 1979

AUGUST 1995

FIGURE 3.4

11" x 17"/ BASE SCALE: 1" = 1"

KGS FILE NO. 95-311-01



PLAN

NOTE : CONTOURS ARE BASED ON 1995 DIVERS RECONNAISANCE SURVEY

KGS GROUP

MANITOBA DEPARTMENT OF NATURAL RESOURCES

RED RIVER FLOODWAY - INLET CENTRAL STRUCTURE EROSION STUDY

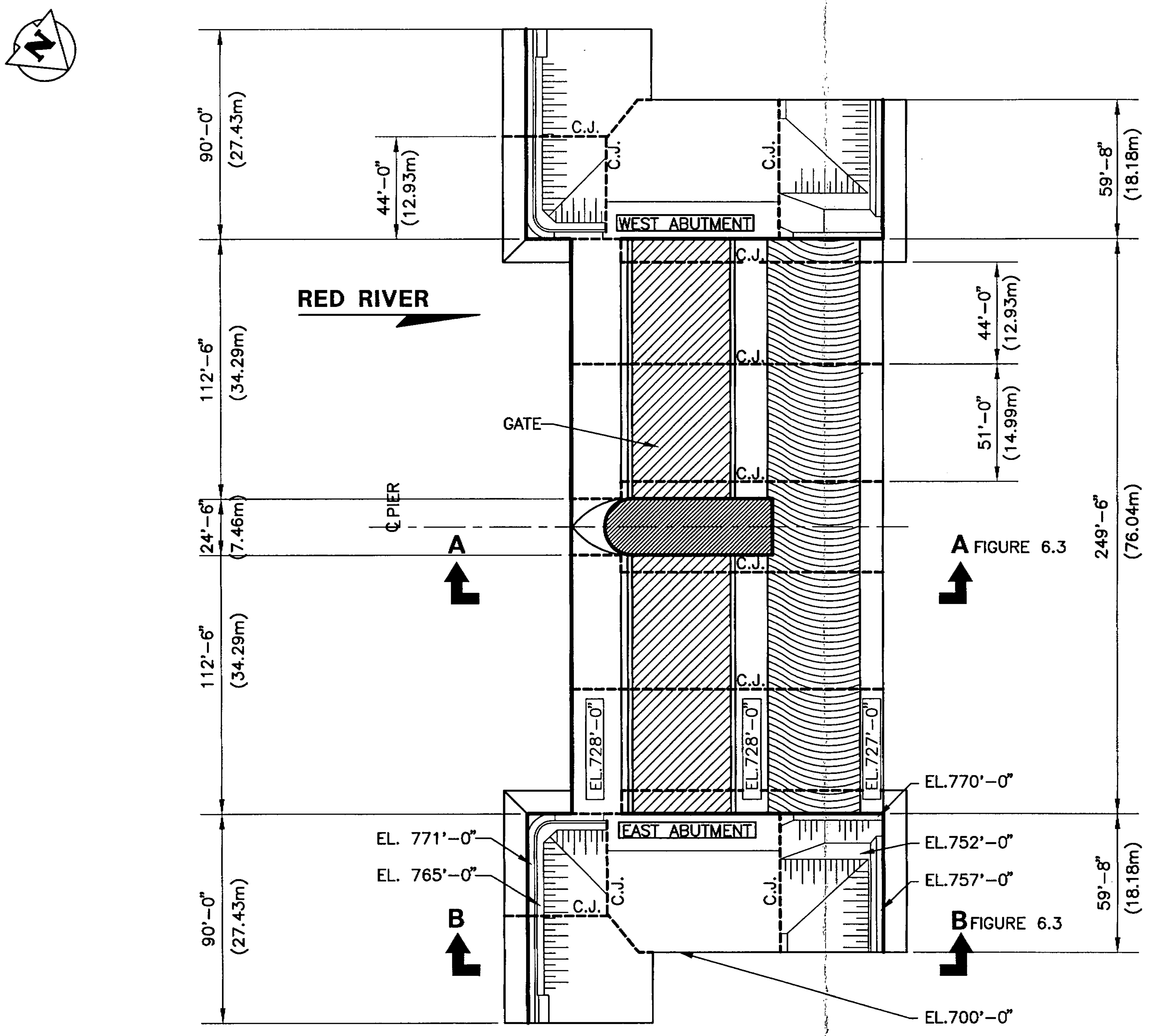
EROSION DOWNSTREAM & INLET STRUCTURE - 1995

AUGUST 1995

FIGURE 5.1

17" X 11"/ SCALE: 1"=50' OR 1:600

KGS No. ERODED.DWG EDS FILE



LEGEND

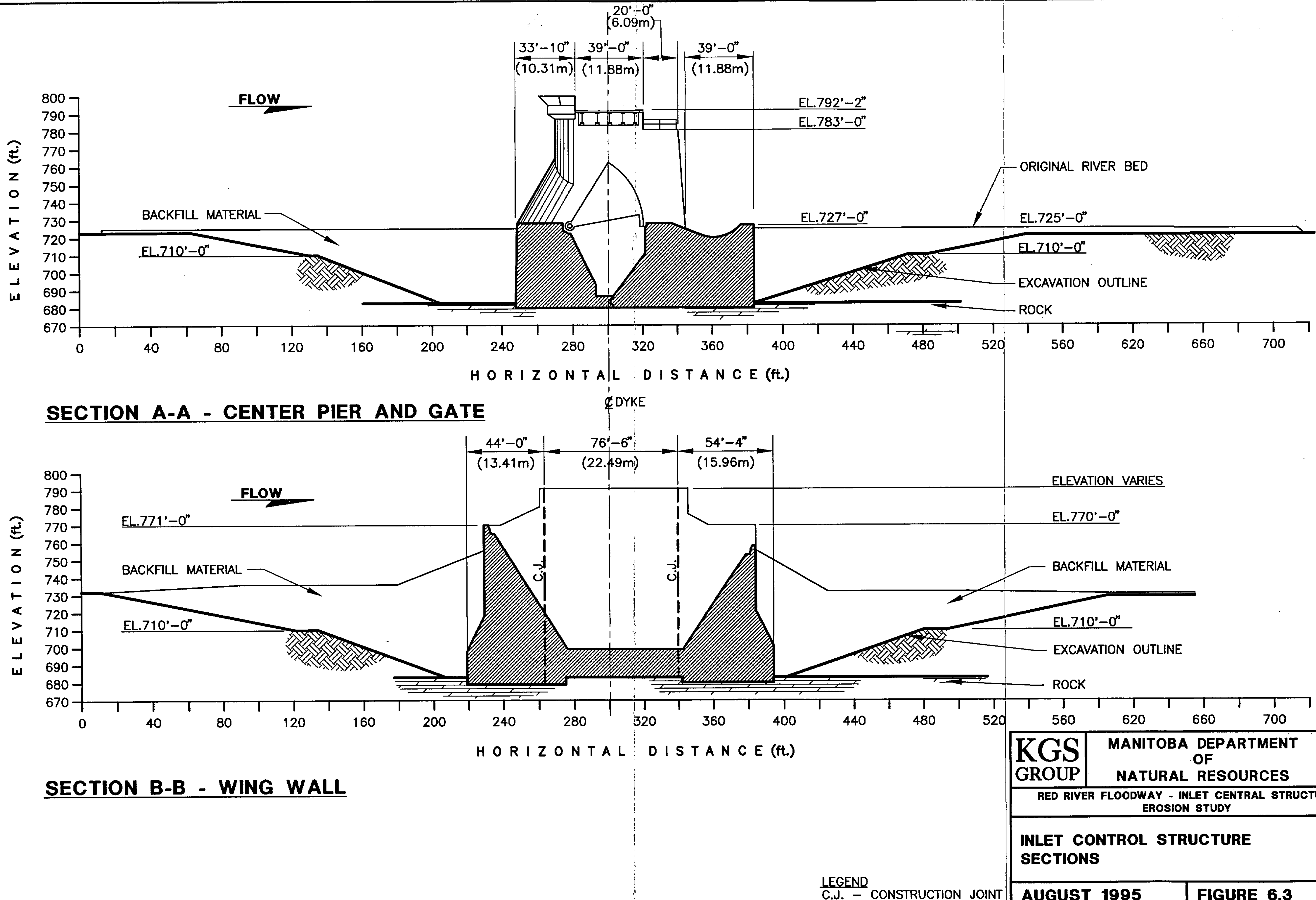
C.J. – CONSTRUCTION JOINT

SCALE 50 40 30 20 10 0.0 25 50 100 FEET

KGS GROUP	MANITOBA DEPARTMENT OF NATURAL RESOURCES
RED RIVER FLOODWAY - INLET CENTRAL STRUCTURE EROSION STUDY	
INLET CONTROL STRUCTURE PLAN	
AUGUST 1995	FIGURE 6.2

17" X 11"/ PLOT SCALE: 1=50

KGS No. 53110198.DWG



KGS GROUP

MANITOBA DEPARTMENT OF NATURAL RESOURCES

RED RIVER FLOODWAY - INLET CENTRAL STRUCTURE EROSION STUDY

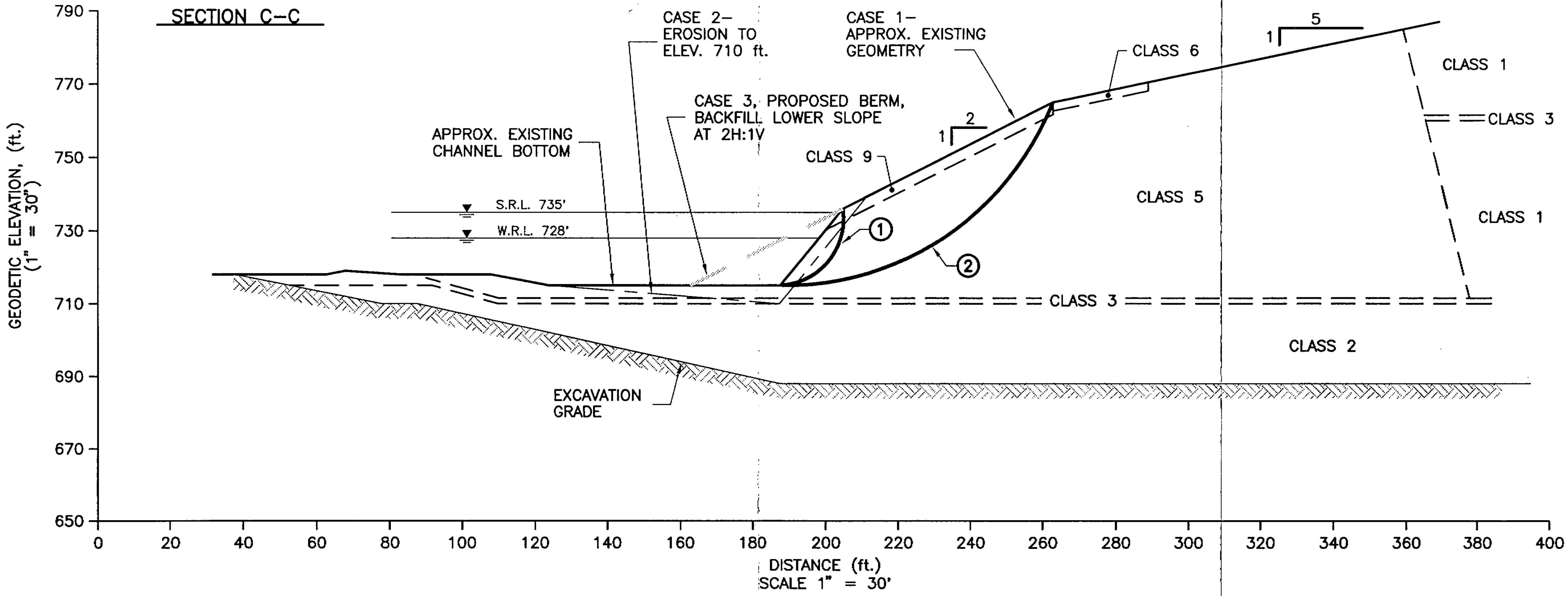
INLET CONTROL STRUCTURE SECTIONS

AUGUST 1995

FIGURE 6.3

11"x17"/SCALE: 1"=50'-0"

KGS DWG. NO. 53110199.DWG



LEGEND:

TYPE	MATERIAL	STRENGTH ø' ø'	PARAMETERS c' (kPa) c' (kPa)
CLASS 1	Unknown	NA	NA
CLASS 2	Till Fill	35°	0
CLASS 3	Filter Stone, D_{50} = 1.5"	44°	0
CLASS 5	Filter Stone, D_{50} = 6"	44°	0
CLASS 6	Riprap, D_{50} = 6"	44°	0
CLASS 9	Riprap, D_{50} = 18"	44°	0

S.R.L. – SUMMER RIVER LEVEL
W.R.L. – WINTER RIVER LEVEL
NA – NOT APPLICABLE

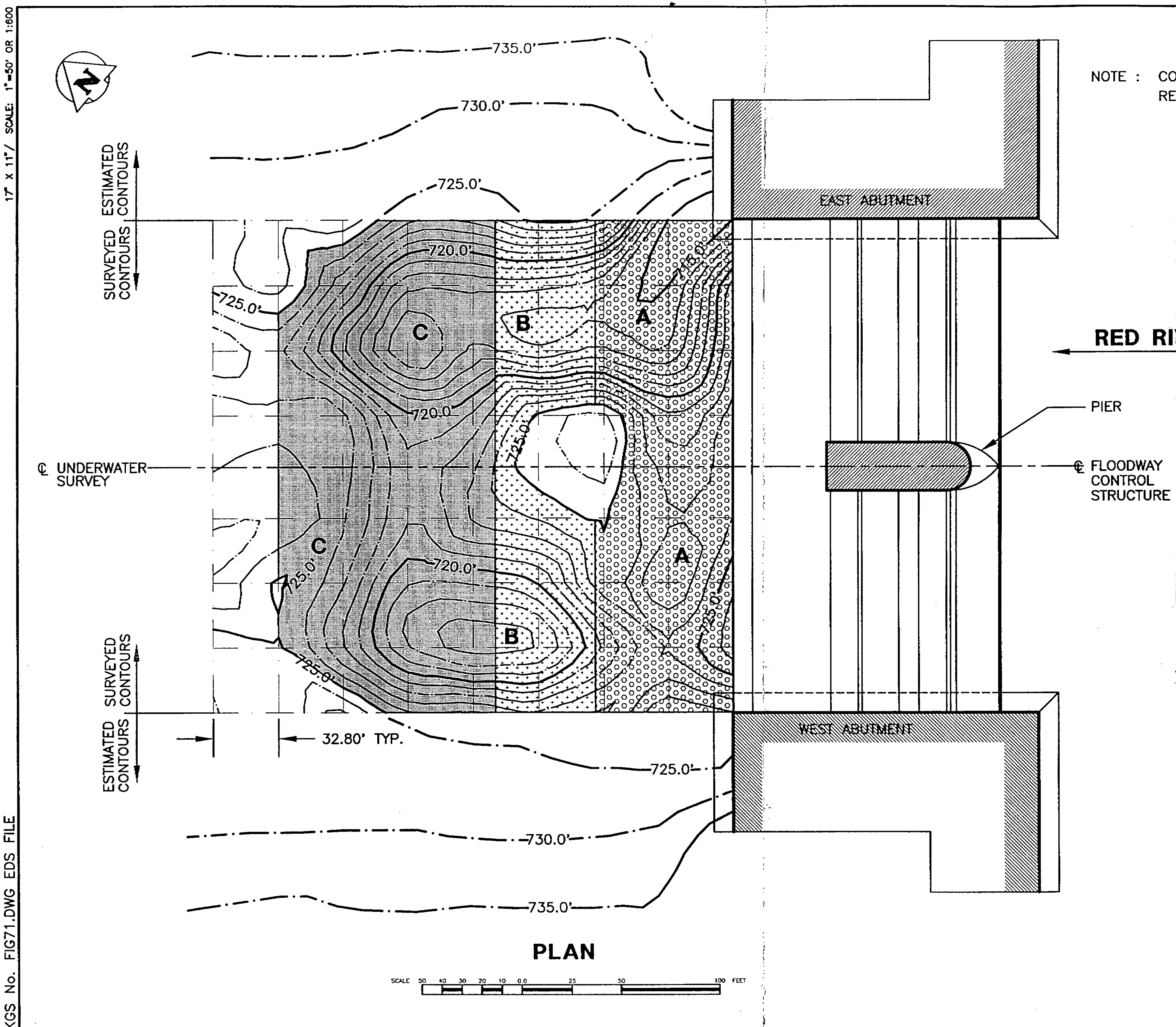
NOTE:

1. CROSS SECTION BASED ON PROVINCE OF MANITOBA DEPARTMENT OF AGRICULTURE AND CONSERVATION WATER CONTROL AND CONSERVATION BRANCH DRAWINGS FOR THE RED RIVER FLOODWAY INLET CONTROL WORKS, DRAWINGS 940–C–3009 SHEETS 1 AND 2 AND 940–C–3010, NOV. 25, 1963.
2. STRENGTH PARAMETERS BASED ON A BACK ANALYSIS OF THE EXISTING GEOMETRY FOR AN ESTIMATED FS = 1.0 AND W.R.L. SEE TABLE 1 FOR A SUMMARY OF THE ESTIMATED SAFETY FACTORS.
3. THE EXISTING GEOMETRY BELOW RIVER LEVEL WAS ESTIMATED BASED ON THE RESULTS OF THE CHANNEL SURVEY AND THE LOCATION OF THE HEAD SCARP APPROXIMATELY 1 TO 2ft ABOVE S.R.L. THE ACTUAL GEOMETRY WAS NOT SURVEYED.
4. SEE FIGURE 6.1 FOR LOCATION OF CROSS SECTION C–C.

KGS GROUP	MANITOBA DEPARTMENT OF NATURAL RESOURCES
RED RIVER FLOODWAY - INLET CENTRAL STRUCTURE EROSION STUDY	
STRATIGRAPHIC AND STABILITY CROSS SECTION C-C	
AUGUST 1995	FIGURE 6.4

17" X 11"/SCALE: 1" = 30'

KGS No. 95–311–0197



KGS GROUP

MANITOBA DEPARTMENT OF NATURAL RESOURCES

RED RIVER FLOODWAY EROSION STUDY

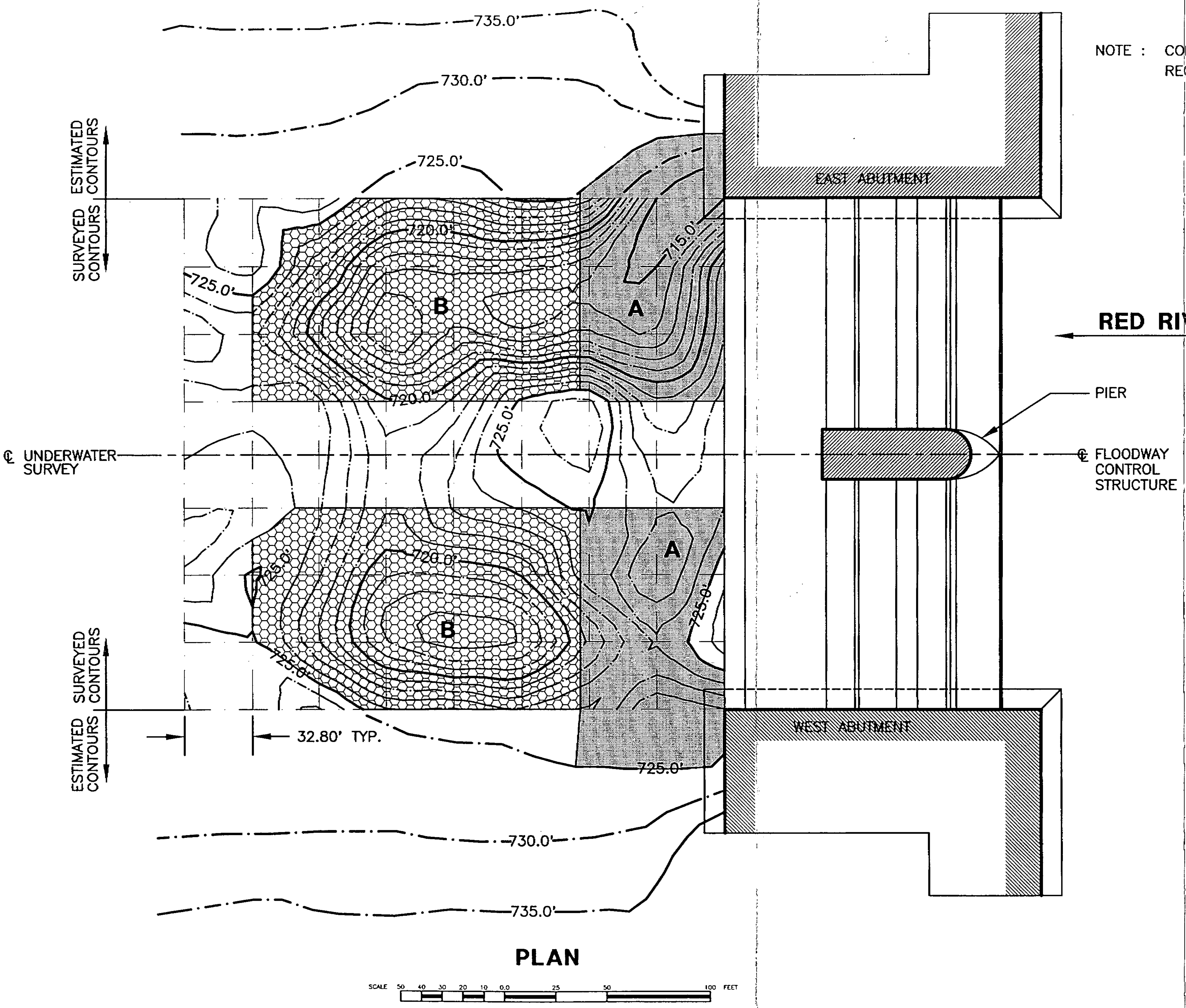
RIP RAP BLANKET ALTERNATIVE

OCT. 1995

FIGURE 7.1

17" X 11"/ SCALE: 1"=50' OR 1:600

KGS No. FIG71.DWG EDS FILE



NOTE : CONTOURS ARE BASED ON 1995 DIVERS RECONNAISANCE SURVEY

LEGEND

ZONE		CONCRETE MAT
	A	3 FT CONCRETE BAG
	B	8 INCH AB–MAT

KGS GROUP

MANITOBA DEPARTMENT OF NATURAL RESOURCES

RED RIVER FLOODWAY EROSION STUDY

CONCRETE MAT ALTERNATIVE

OCT. 1995 | **FIGURE 7.2**

KGS No. fig72.DWG EDS FILE

17" x 11"/ SCALE: 1"=50' OR 1:600

Red River Floodway Inlet Control Structure Erosion Study
Upstream and Downstream Water Levels - 1974

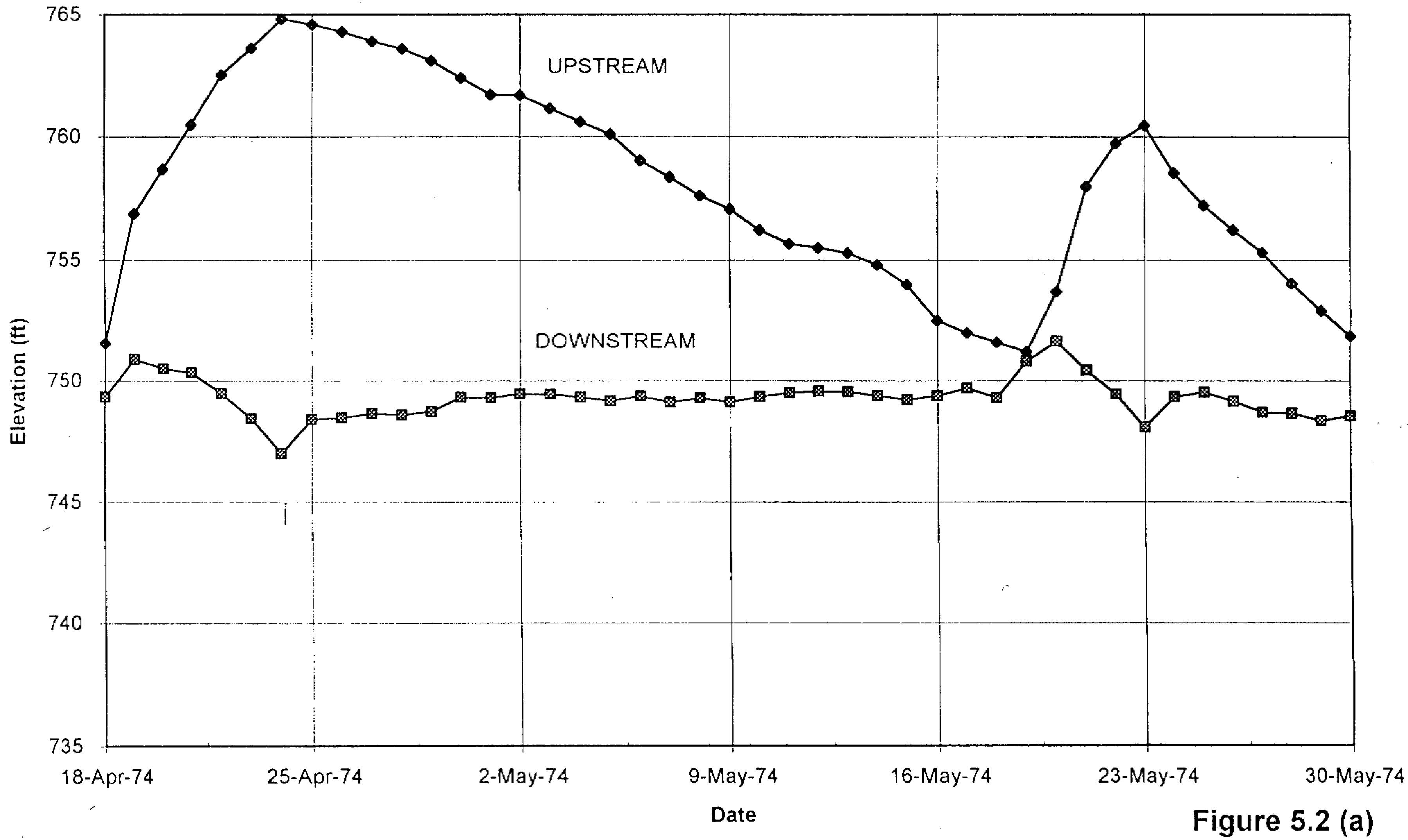


Figure 5.2 (a)

Red River Floodway Inlet Control Structure Erosion Study
Upstream and Downstream Water Levels - 1975

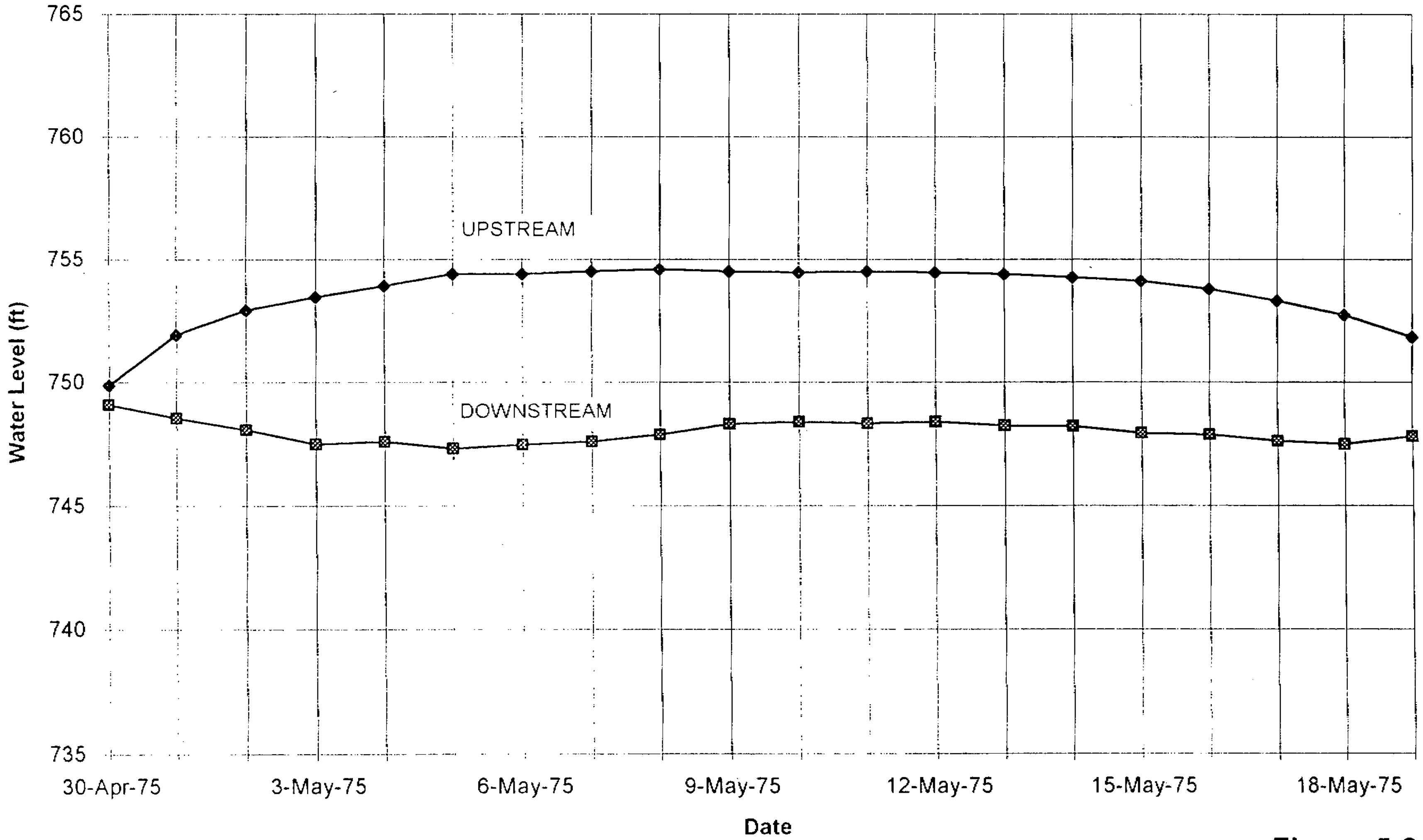


Figure 5.2 (b)

Red River Floodway Inlet Control Structure Erosion Study
Upstream and Downstream Water Levels - 1976

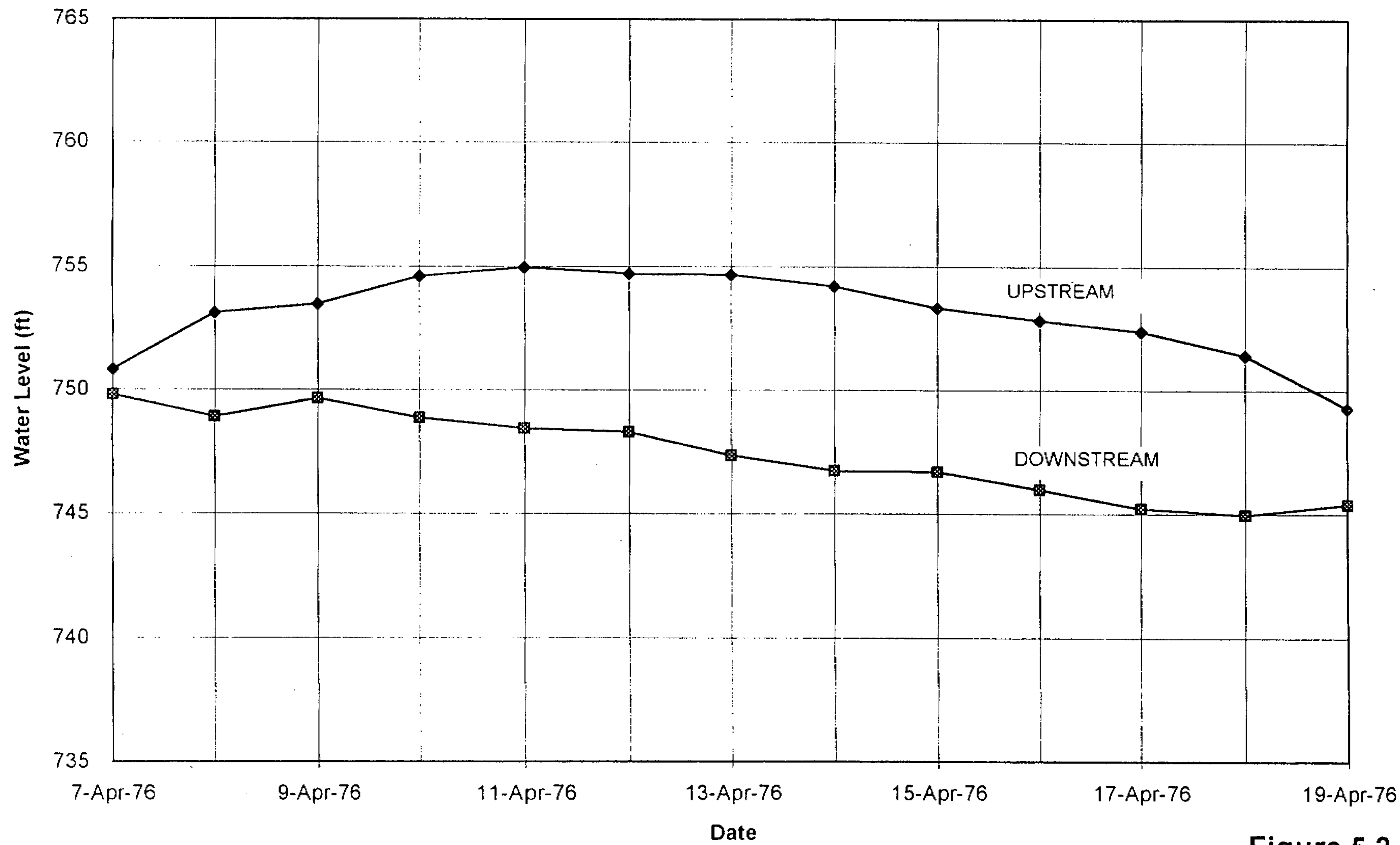


Figure 5.2 (c)

Red River Floodway Inlet Control Structure Erosion Study
Upstream and Downstream Water Levels - 1978

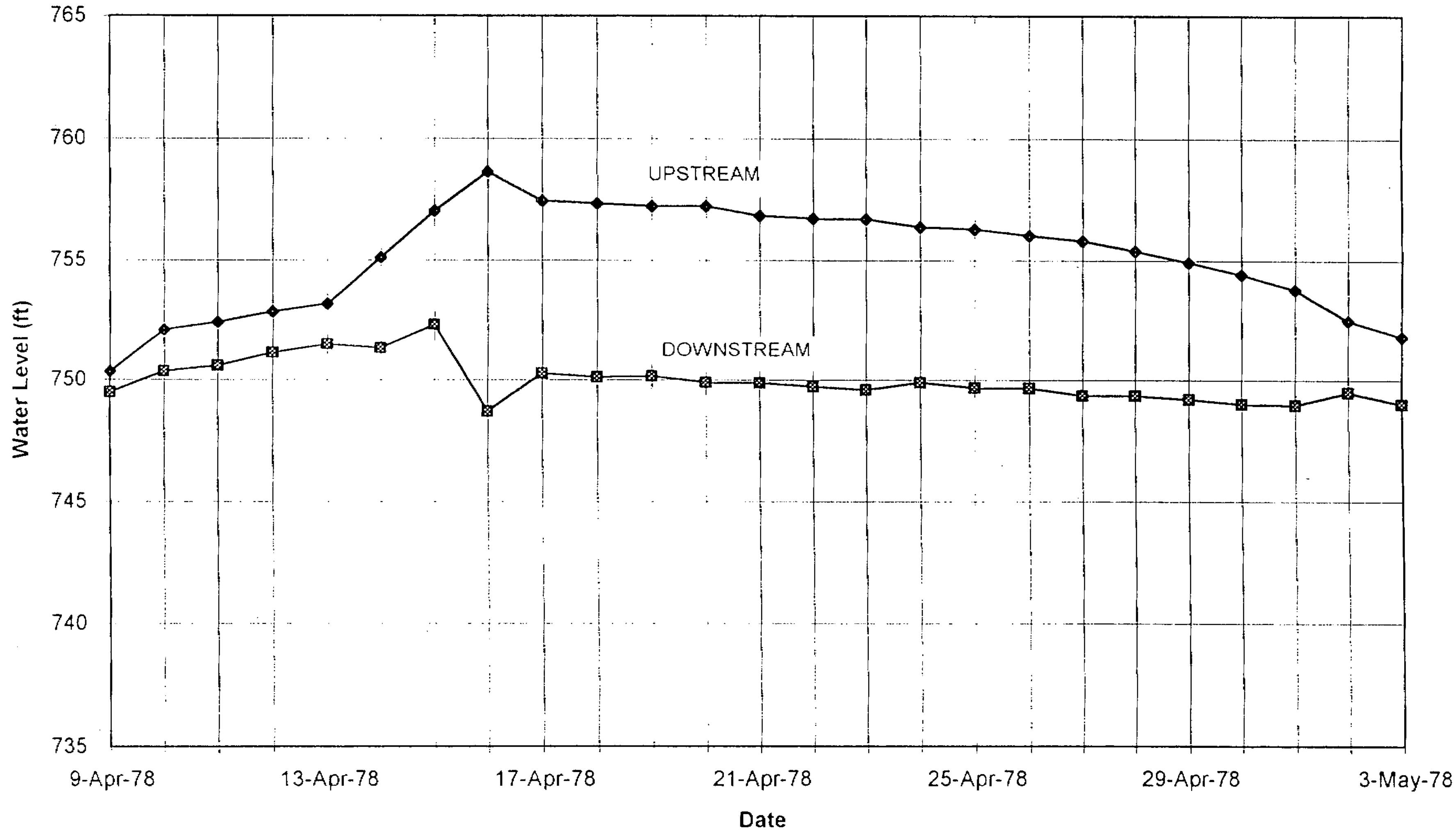


Figure 5.2 (d)

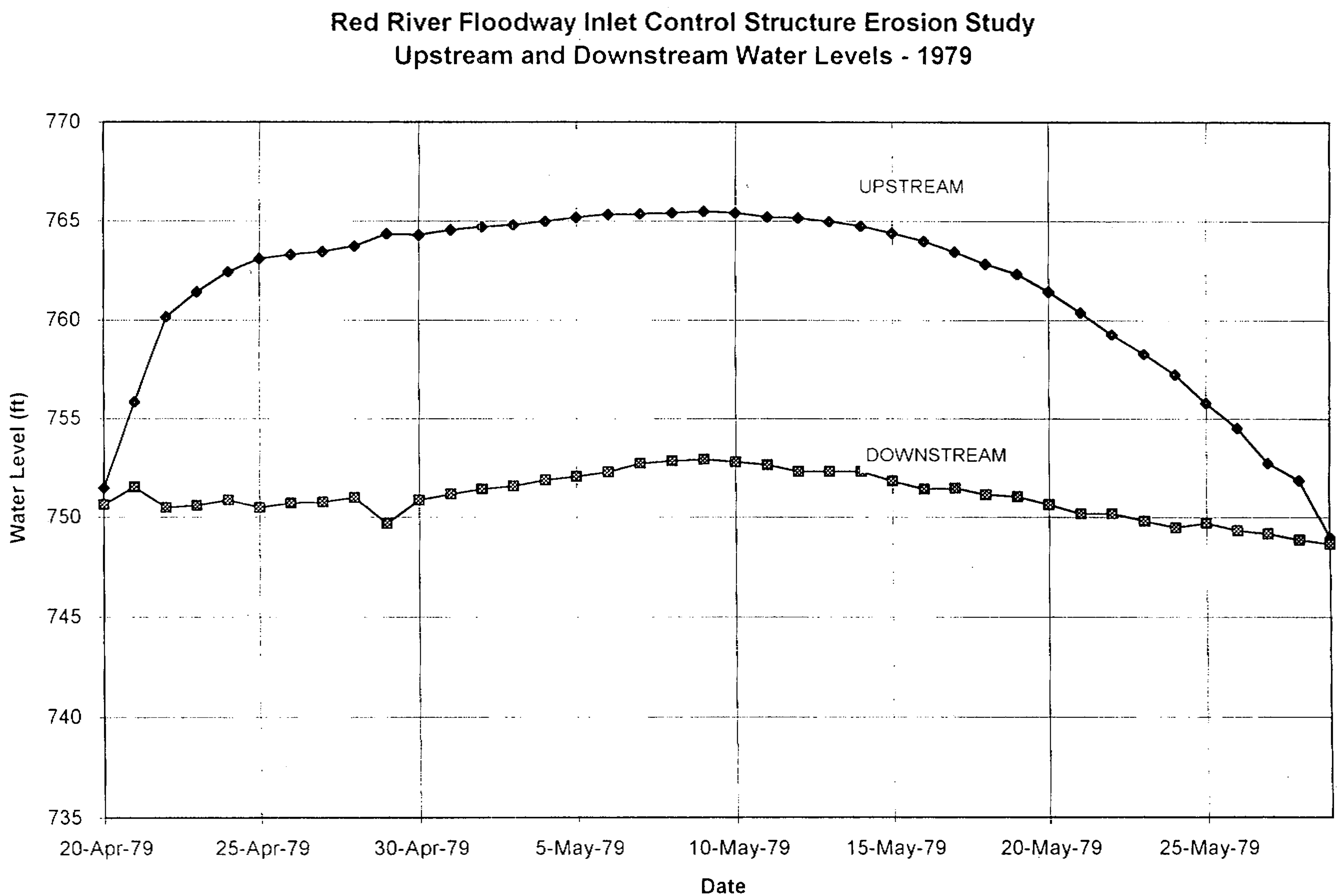

Red River Floodway Inlet Control Structure Erosion Study
Upstream and Downstream Water Levels - 1979
770
765
760
755
750
745
740
735
Water Level (ft)
UPSTREAM
DOWNSTREAM
20-Apr-79
25-Apr-79
30-Apr-79
5-May-79
10-May-79
15-May-79
20-May-79
25-May-79
Date

Figure 5.2 (e)

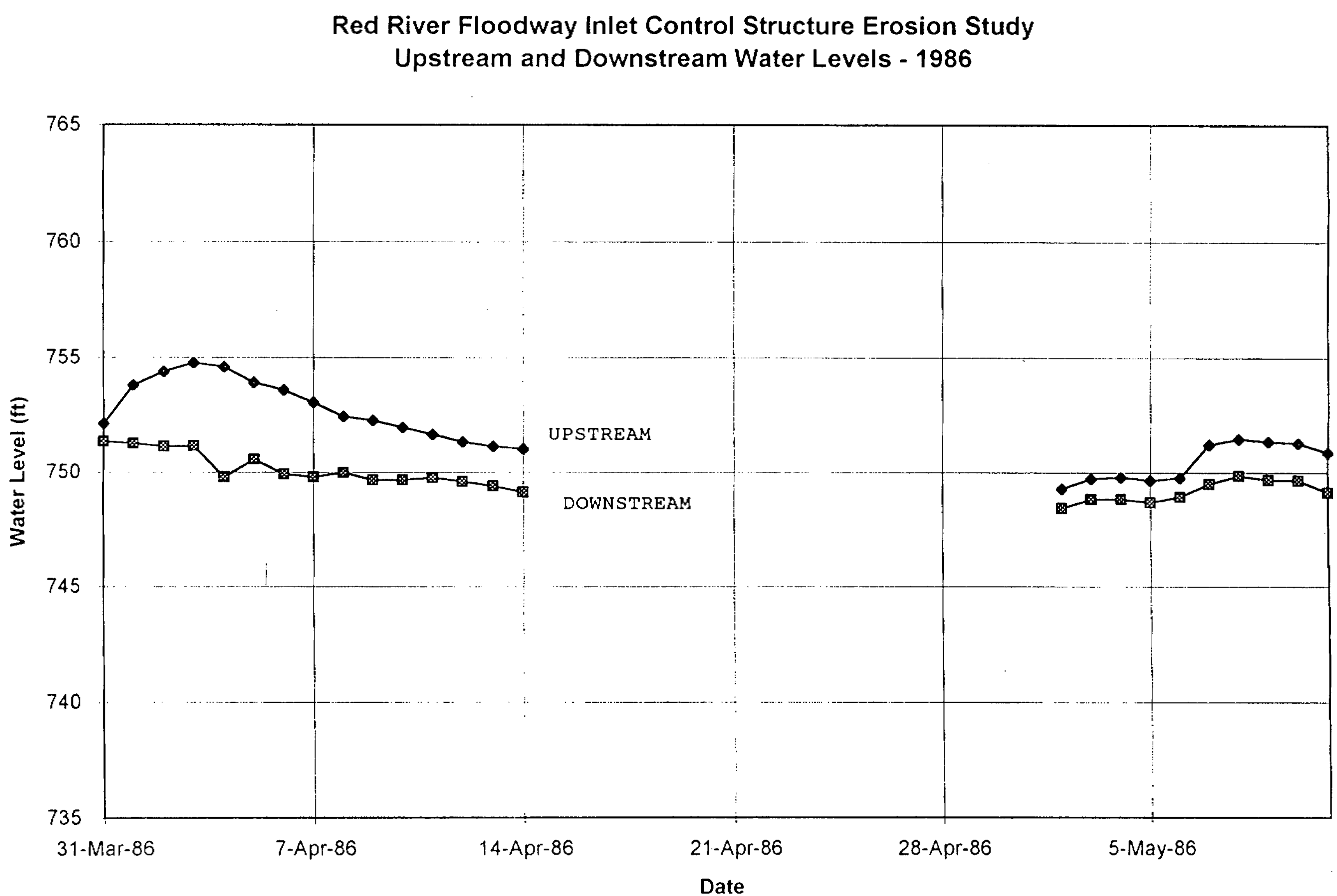

Red River Floodway Inlet Control Structure Erosion Study
Upstream and Downstream Water Levels - 1986
765
760
755
750
745
740
735
Water Level (ft)
UPSTREAM
DOWNSTREAM
31-Mar-86
7-Apr-86
14-Apr-86
21-Apr-86
28-Apr-86
5-May-86
Date

Figure 5.2 (f)

Red River Floodway Inlet Control Structure Erosion Study
Upstream and Downstream Water Levels - 1995

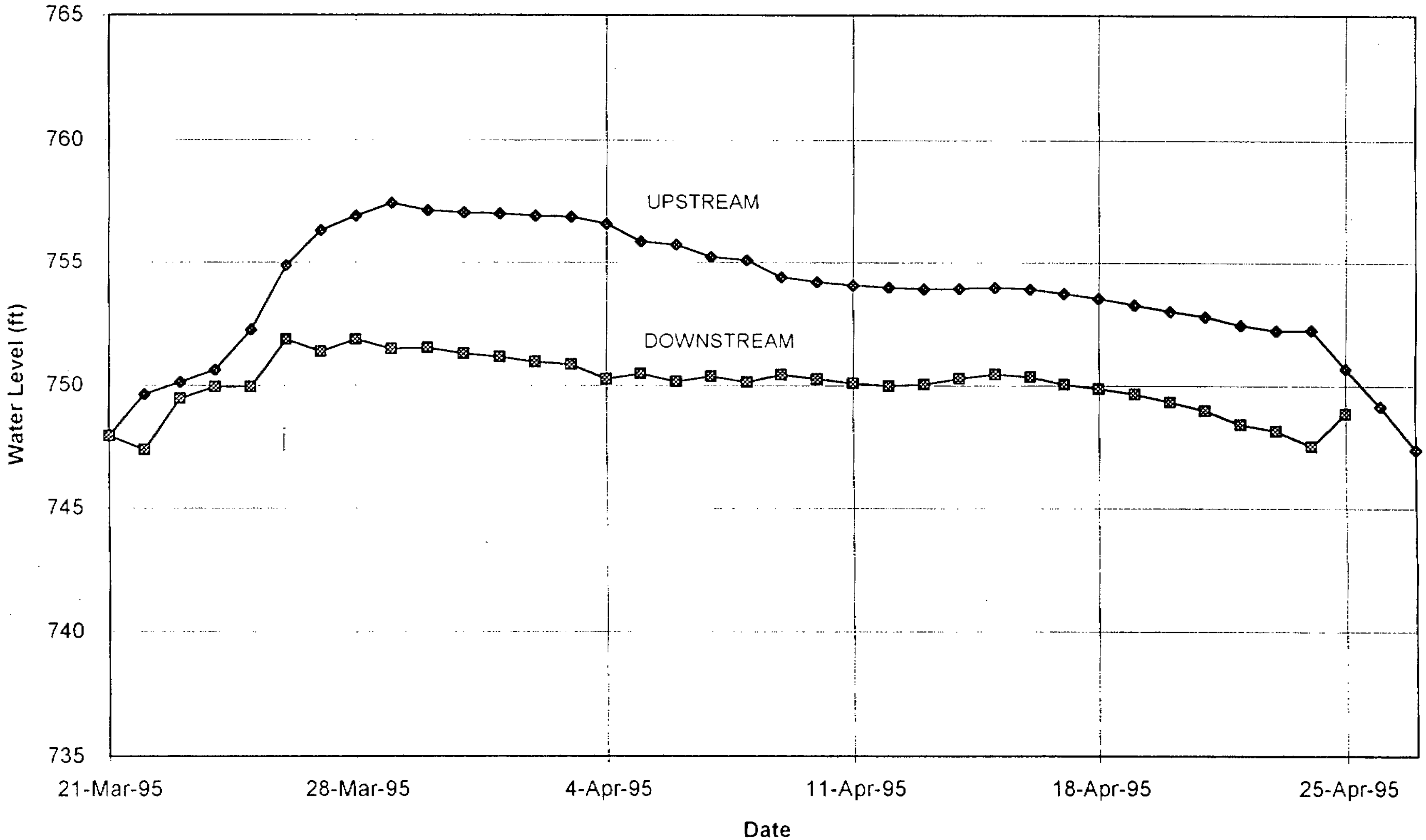


Figure 5.2 (g)

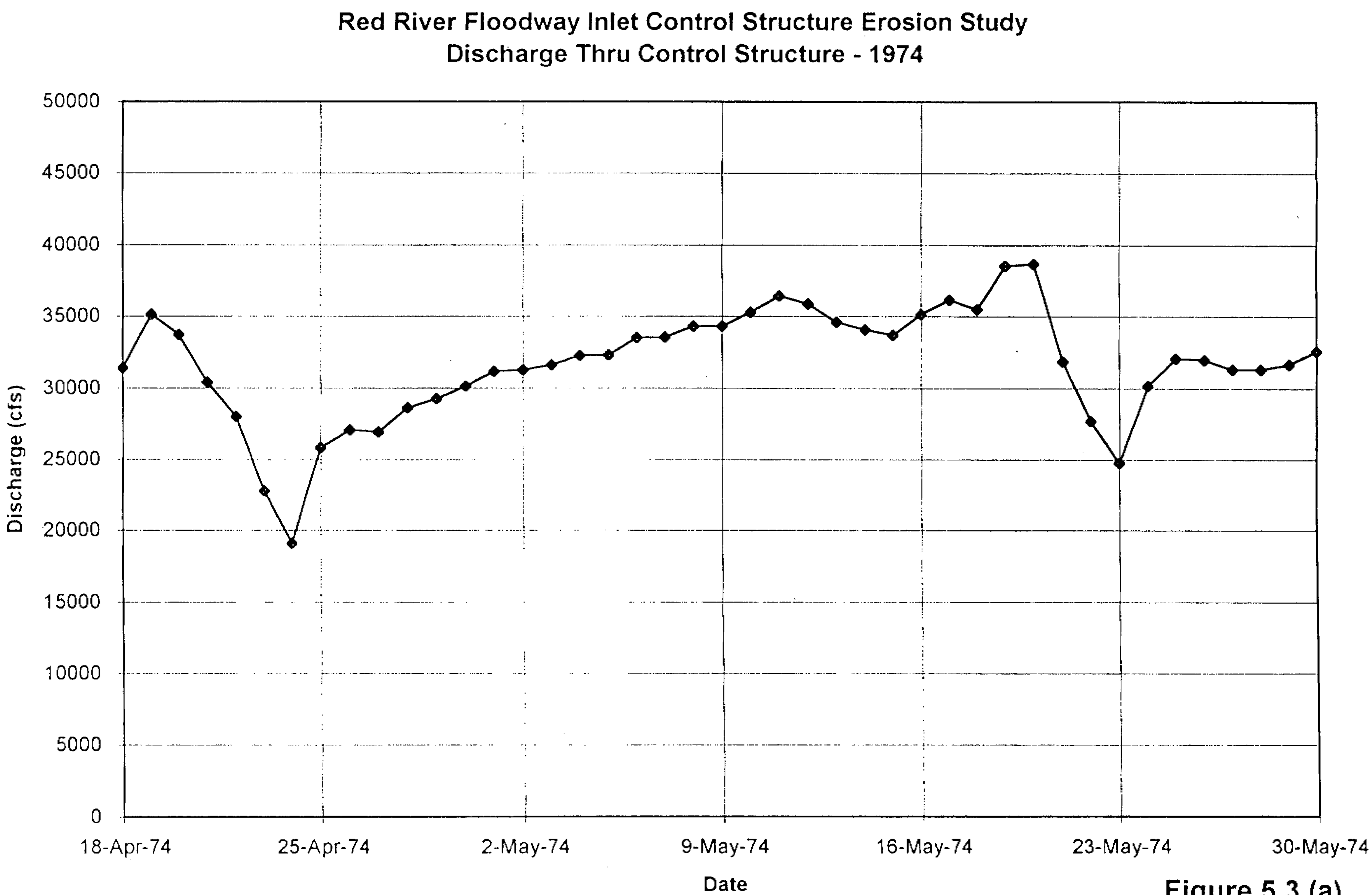

Red River Floodway Inlet Control Structure Erosion Study
Discharge Thru Control Structure - 1974
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
Discharge (cfs)
18-Apr-74
25-Apr-74
2-May-74
9-May-74
16-May-74
23-May-74
30-May-74
Date

Figure 5.3 (a)

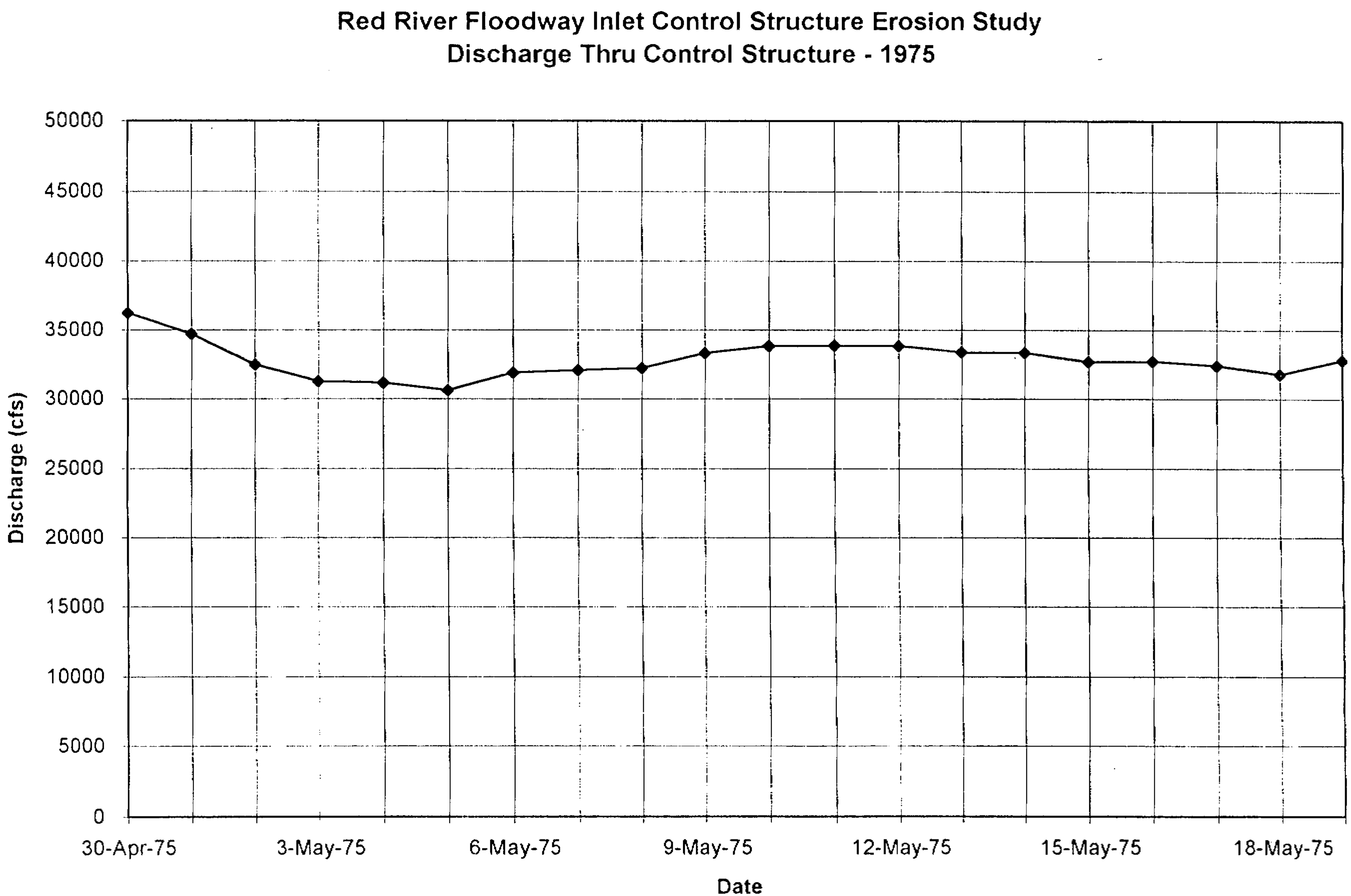

Red River Floodway Inlet Control Structure Erosion Study
Discharge Thru Control Structure - 1975
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
Discharge (cfs)
30-Apr-75
3-May-75
6-May-75
9-May-75
12-May-75
15-May-75
18-May-75
Date

Figure 5.3 (b)

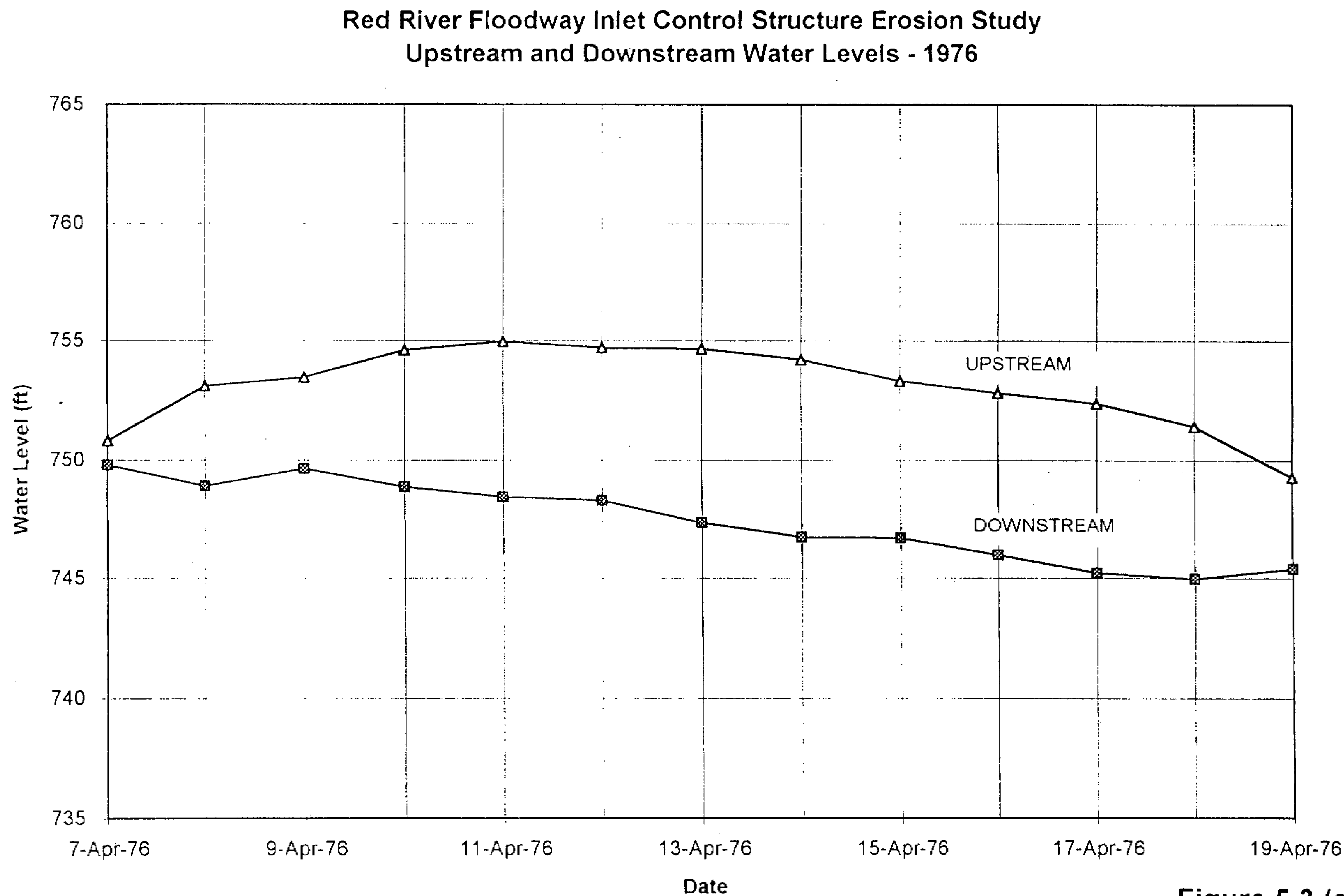
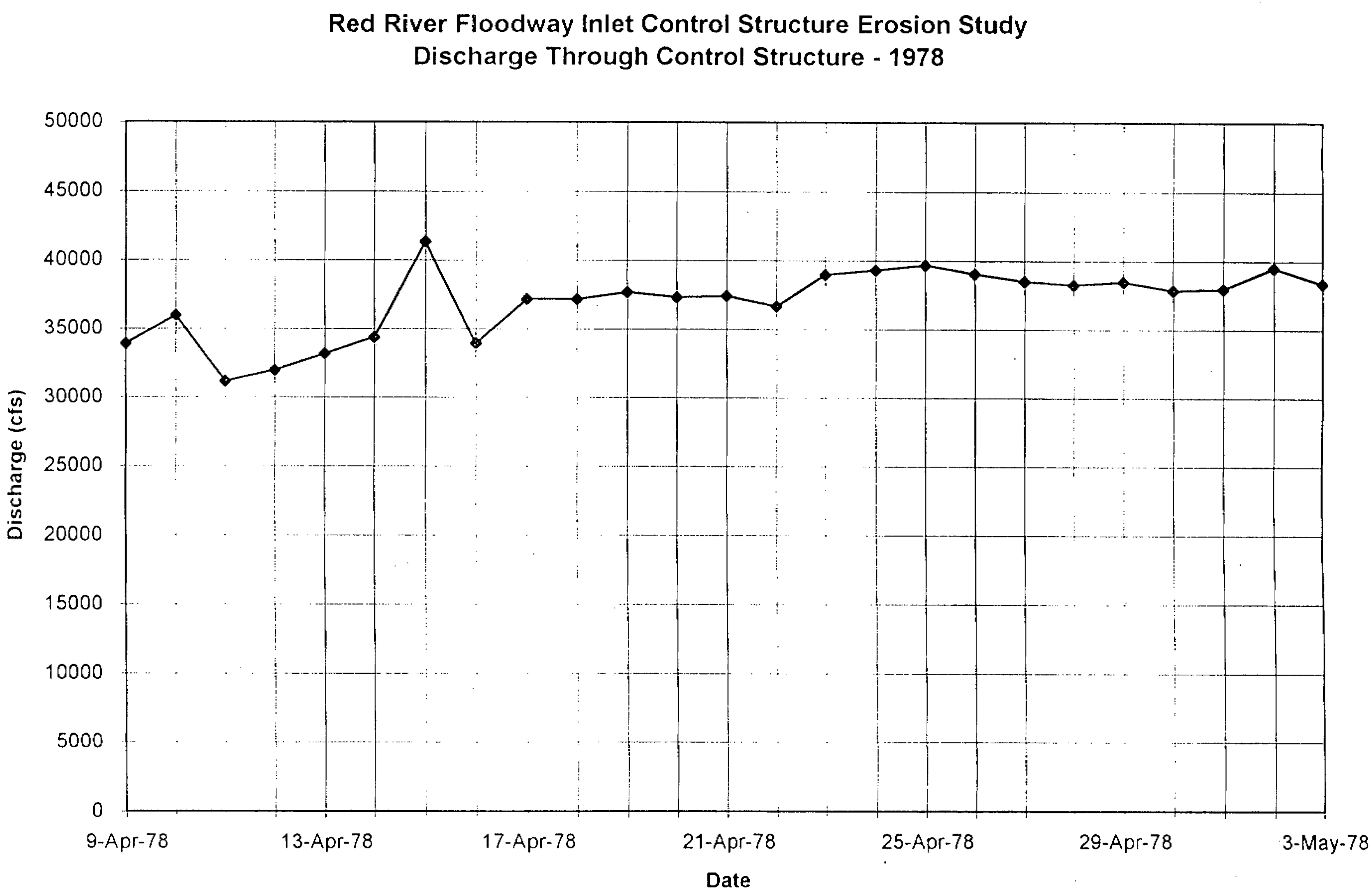

Red River Floodway Inlet Control Structure Erosion Study
Upstream and Downstream Water Levels - 1976
765
760
755
750
745
740
735
Water Level (ft)
UPSTREAM
DOWNSTREAM
7-Apr-76
9-Apr-76
11-Apr-76
13-Apr-76
15-Apr-76
17-Apr-76
19-Apr-76
Date

Figure 5.3 (c)



Red River Floodway Inlet Control Structure Erosion Study
Discharge Through Control Structure - 1978
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
Discharge (cfs)
9-Apr-78
13-Apr-78
17-Apr-78
21-Apr-78
25-Apr-78
29-Apr-78
3-May-78
Date

Figure 5.3 (d)

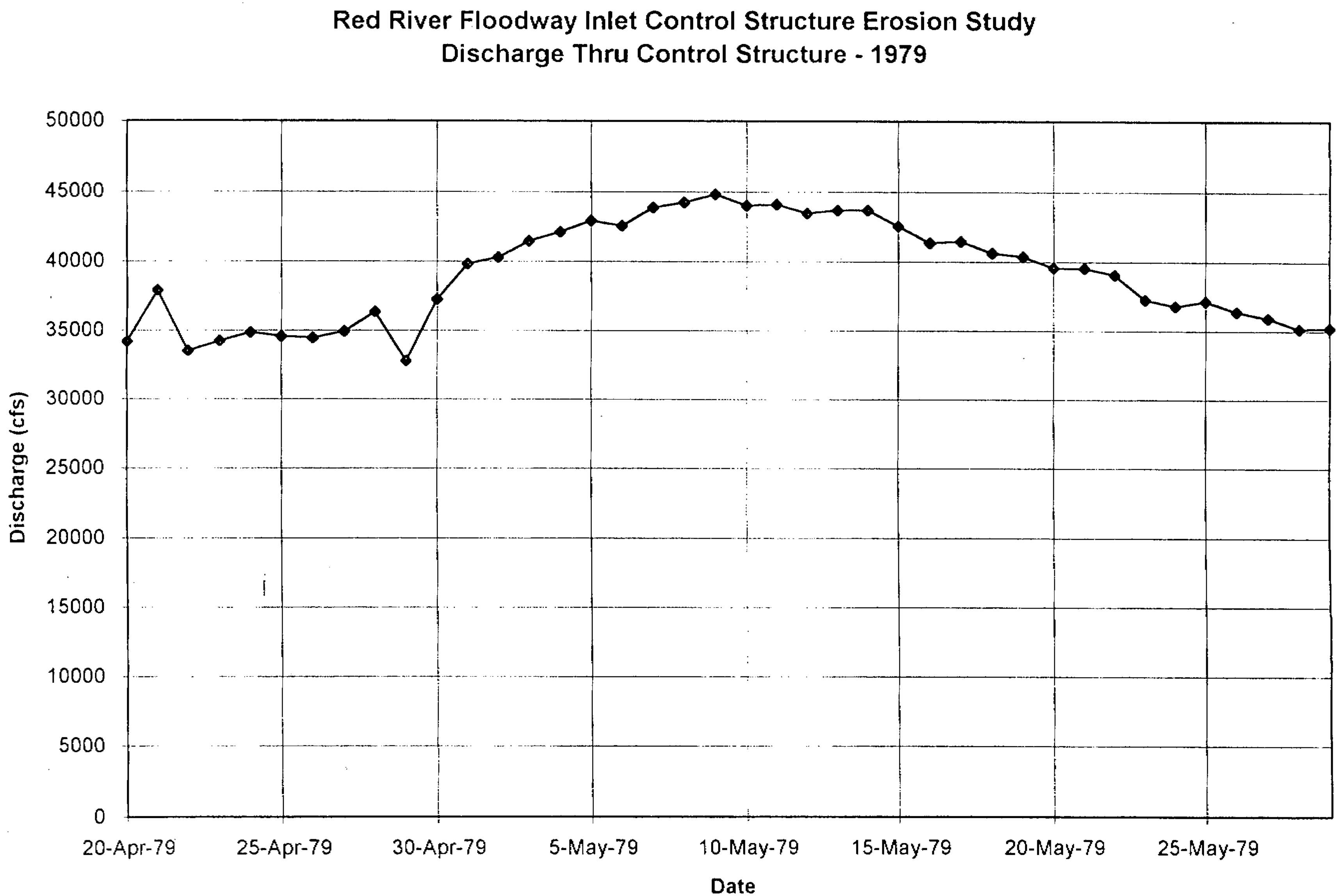
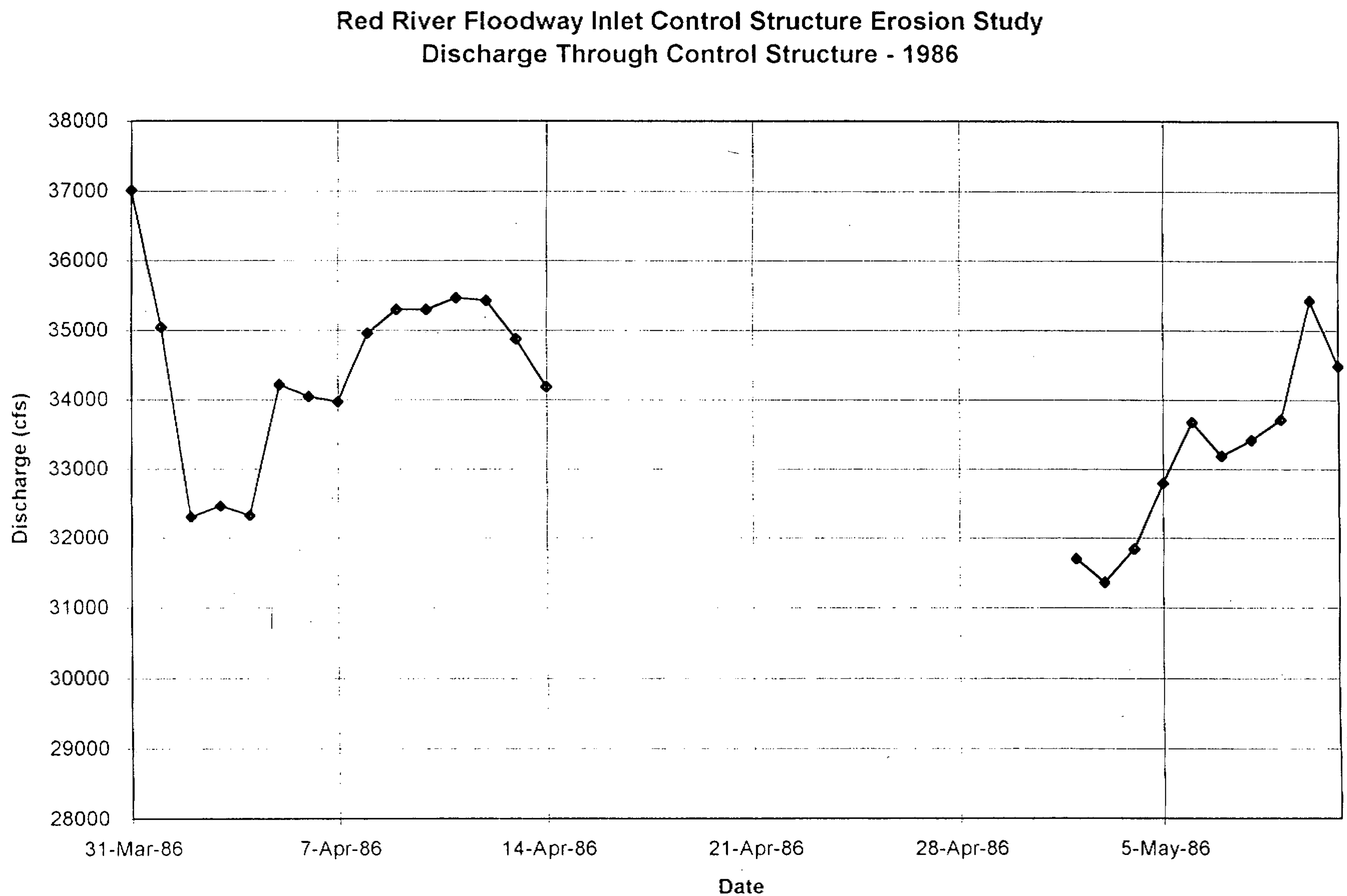

Red River Floodway Inlet Control Structure Erosion Study
Discharge Thru Control Structure - 1979
Discharge (cfs)
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
20-Apr-79
25-Apr-79
30-Apr-79
5-May-79
10-May-79
15-May-79
20-May-79
25-May-79
Date

Figure 5.3 (e)



Red River Floodway Inlet Control Structure Erosion Study
Discharge Through Control Structure - 1986
Discharge (cfs)
38000
37000
36000
35000
34000
33000
32000
31000
30000
29000
28000
31-Mar-86
7-Apr-86
14-Apr-86
21-Apr-86
28-Apr-86
5-May-86
Date

Figure 5.3 (f)

Red River Floodway Inlet Control Structure Erosion Study
Discharge Thru Control Structure - 1995

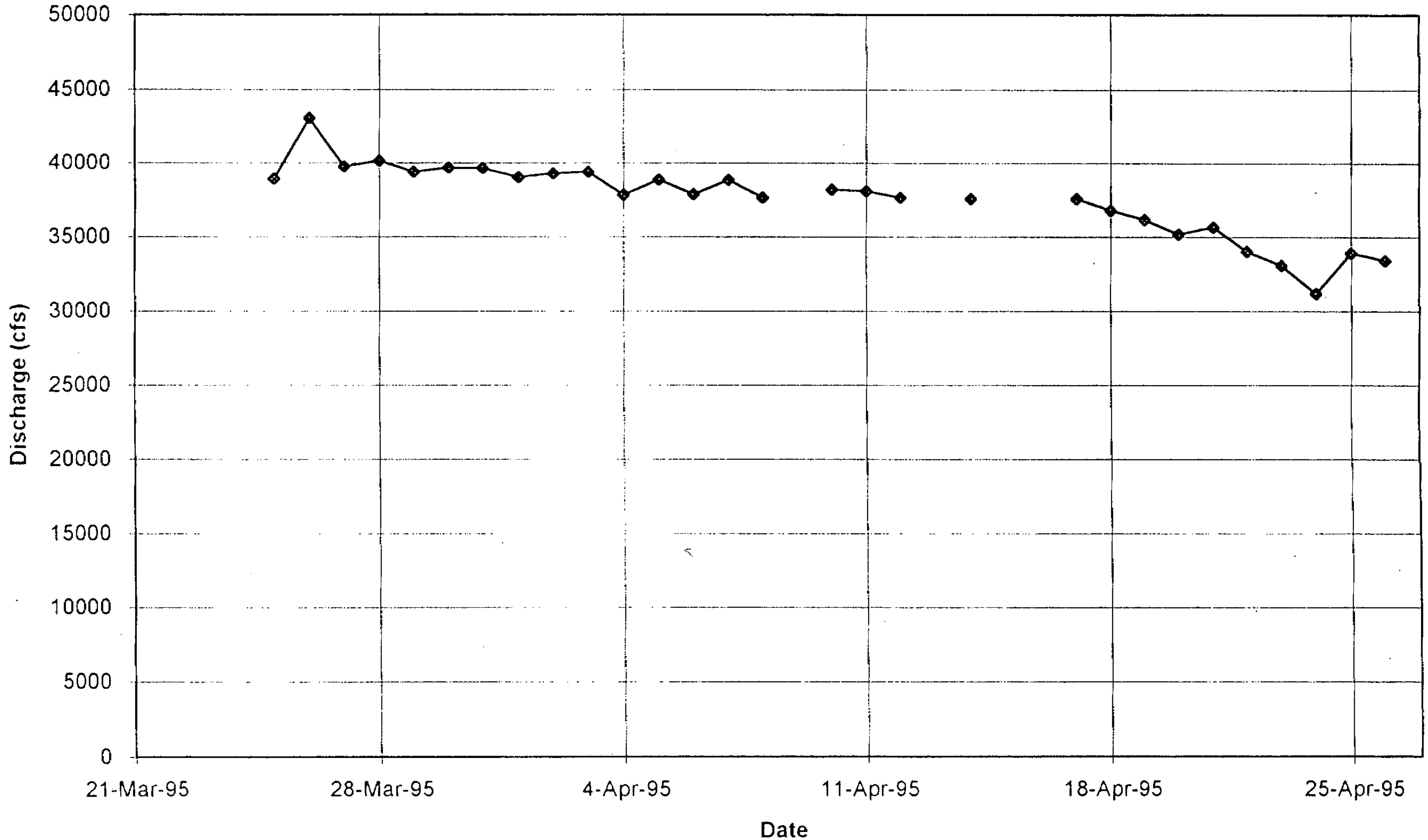
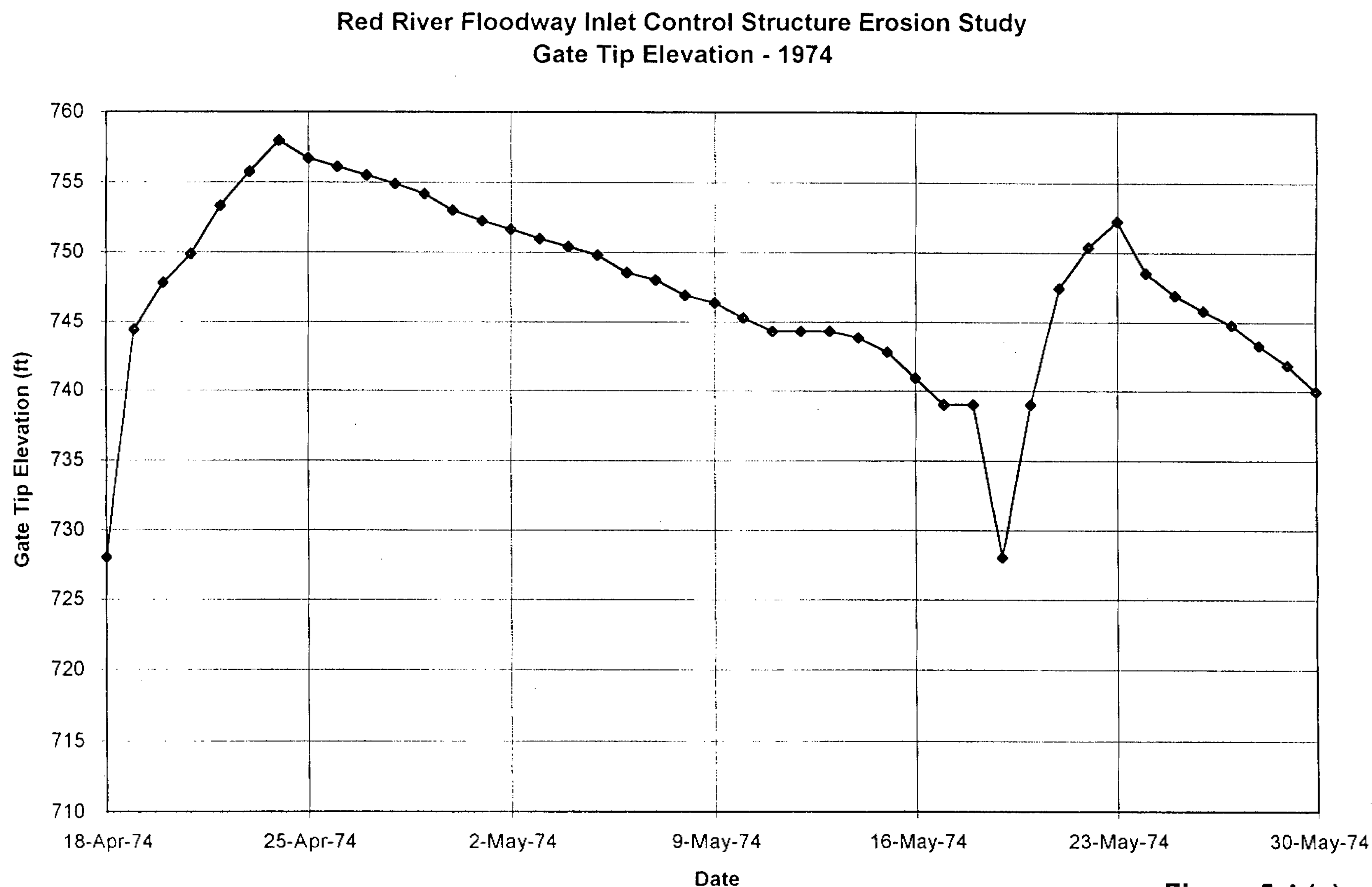


Figure 5.3 (g)



Red River Floodway Inlet Control Structure Erosion Study
Gate Tip Elevation - 1974
Gate Tip Elevation (ft)
760
755
750
745
740
735
730
725
720
715
710
18-Apr-74
25-Apr-74
2-May-74
9-May-74
16-May-74
23-May-74
30-May-74
Date

Figure 5.4 (a)

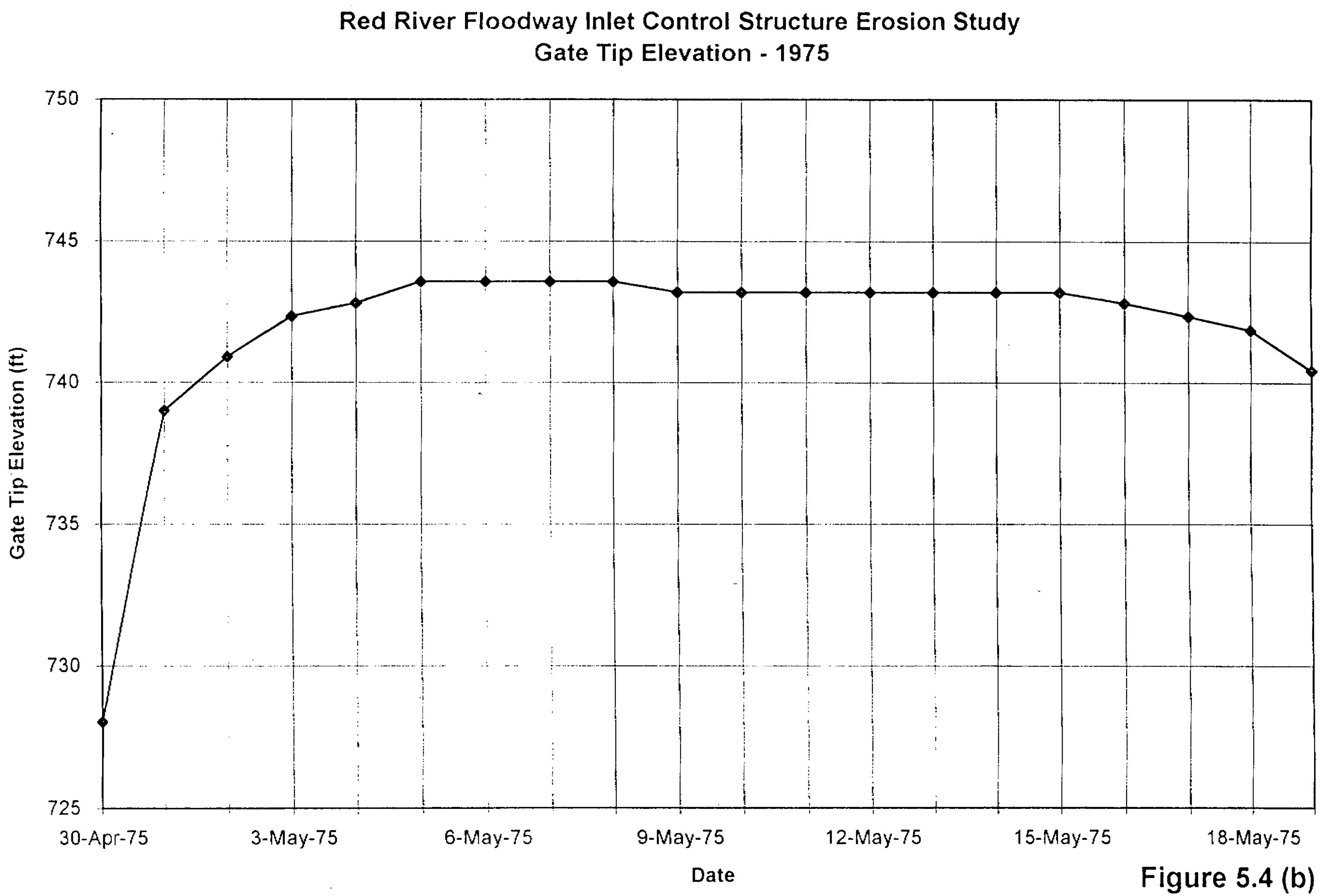

Red River Floodway Inlet Control Structure Erosion Study
Gate Tip Elevation - 1975
750
745
740
735
730
725
Gate Tip Elevation (ft)
30-Apr-75
3-May-75
6-May-75
9-May-75
12-May-75
15-May-75
18-May-75
Date

Figure 5.4 (b)

Red River Floodway Inlet Control Structure Erosion Study
Gate Tip Elevation - 1976

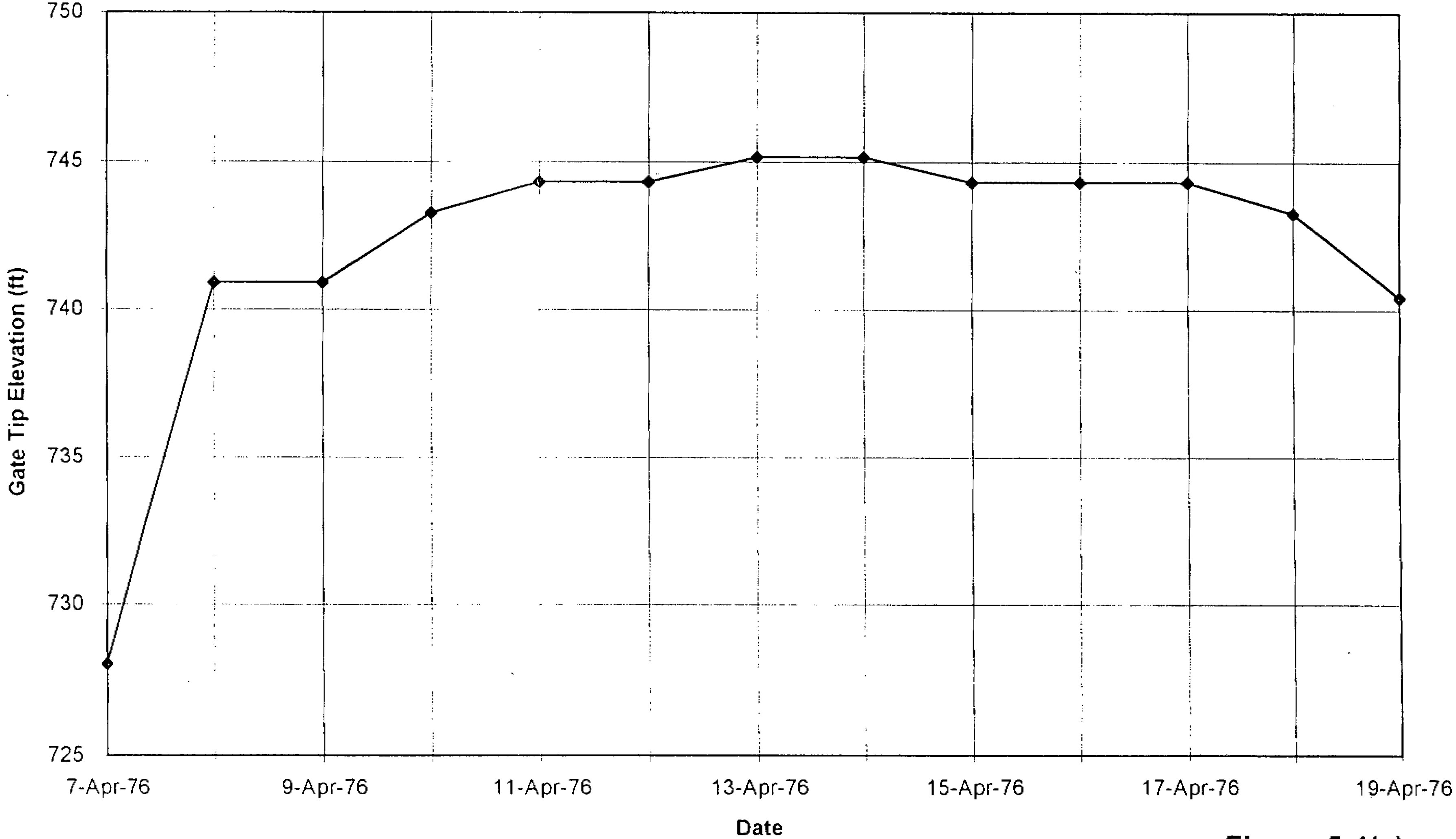


Figure 5.4(c)

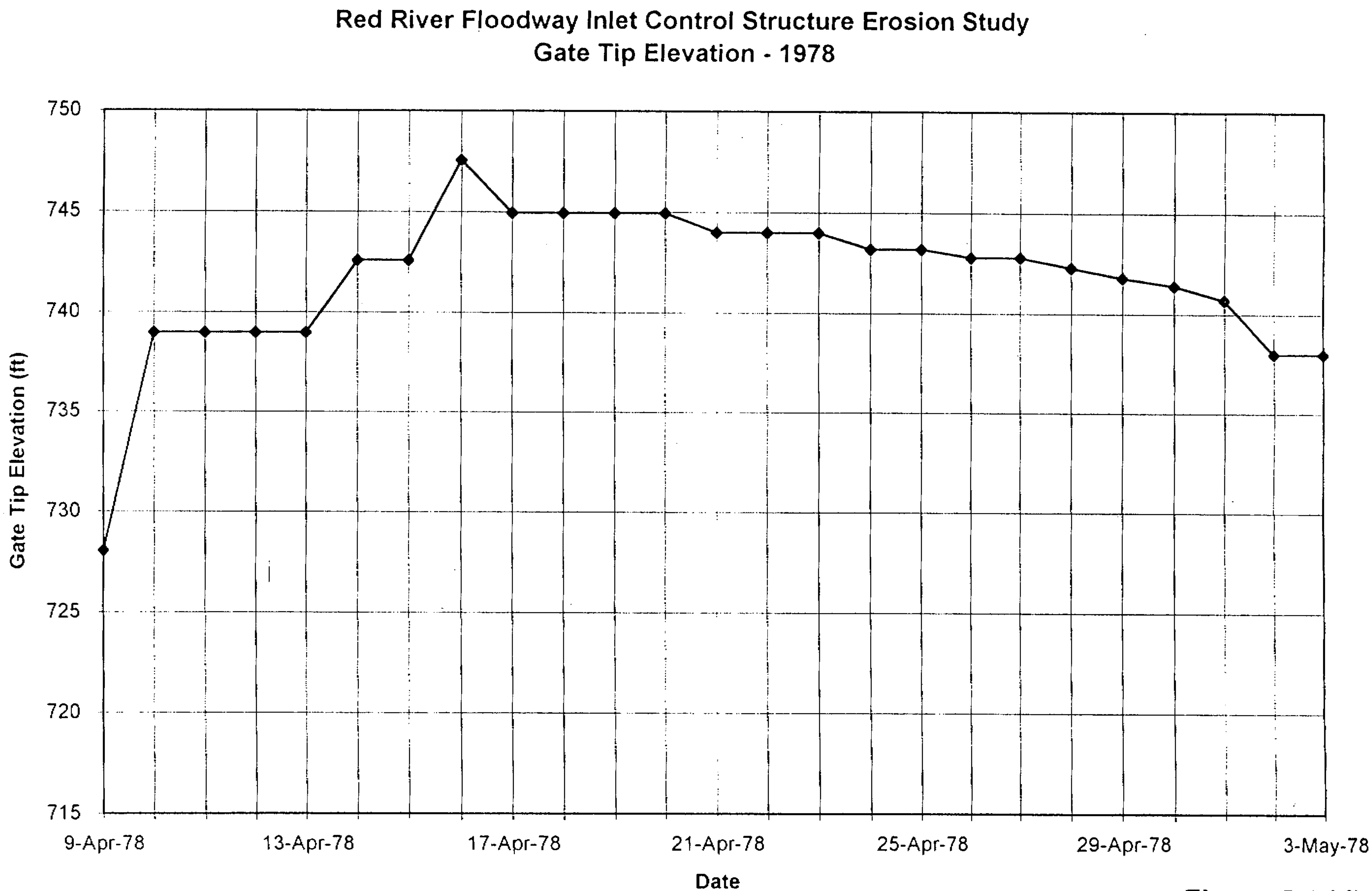

Red River Floodway Inlet Control Structure Erosion Study
Gate Tip Elevation - 1978
Gate Tip Elevation (ft)
750
745
740
735
730
725
720
715
9-Apr-78
13-Apr-78
17-Apr-78
21-Apr-78
25-Apr-78
29-Apr-78
3-May-78
Date

Figure 5.4 (d)

Red River Floodway Inlet Control Structure Erosion Study
Gate Tip Elevation - 1979

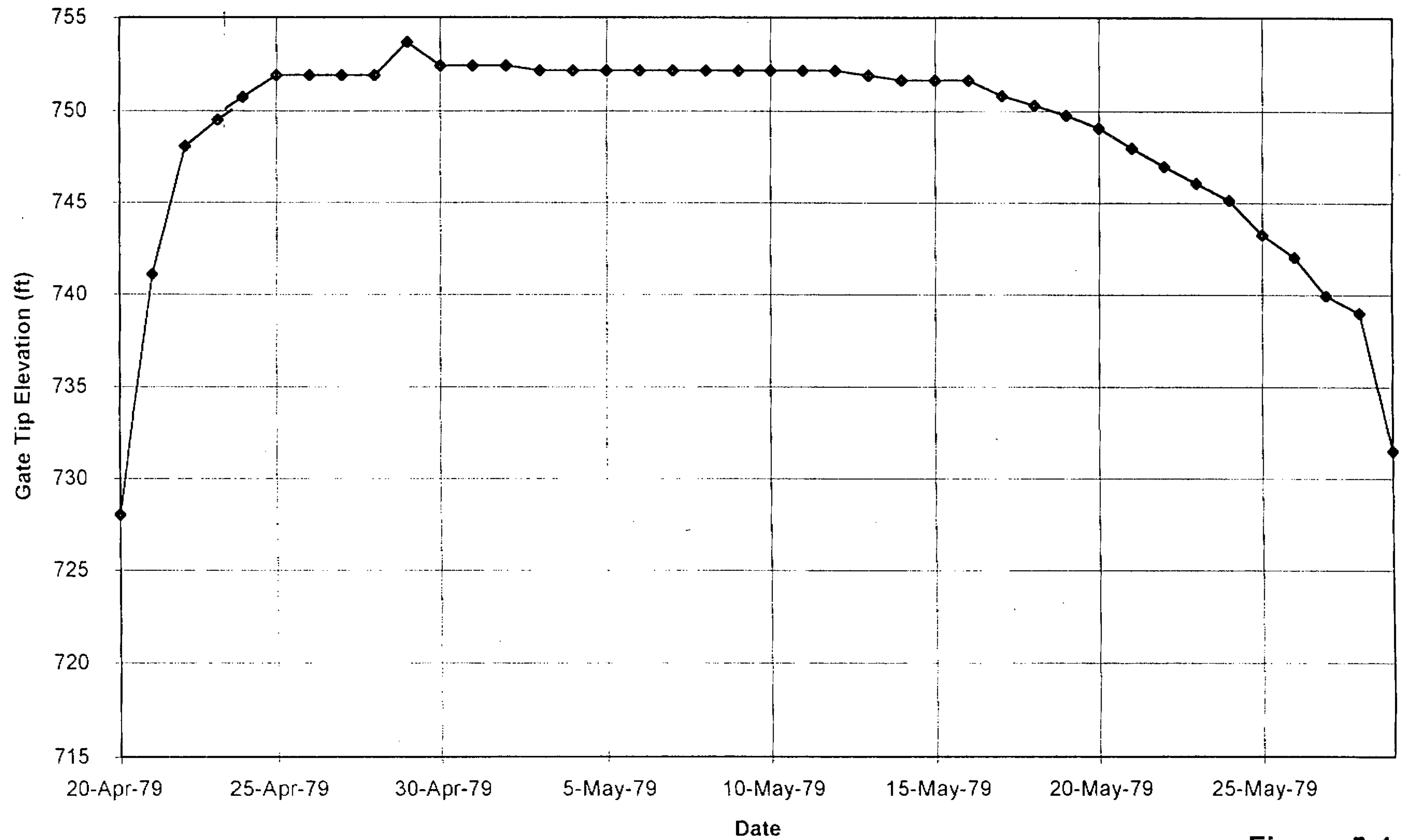


Figure 5.4 (e)

Red River Floodway Inlet Control Structure Erosion Study
Gate Tip Elevation - 1986

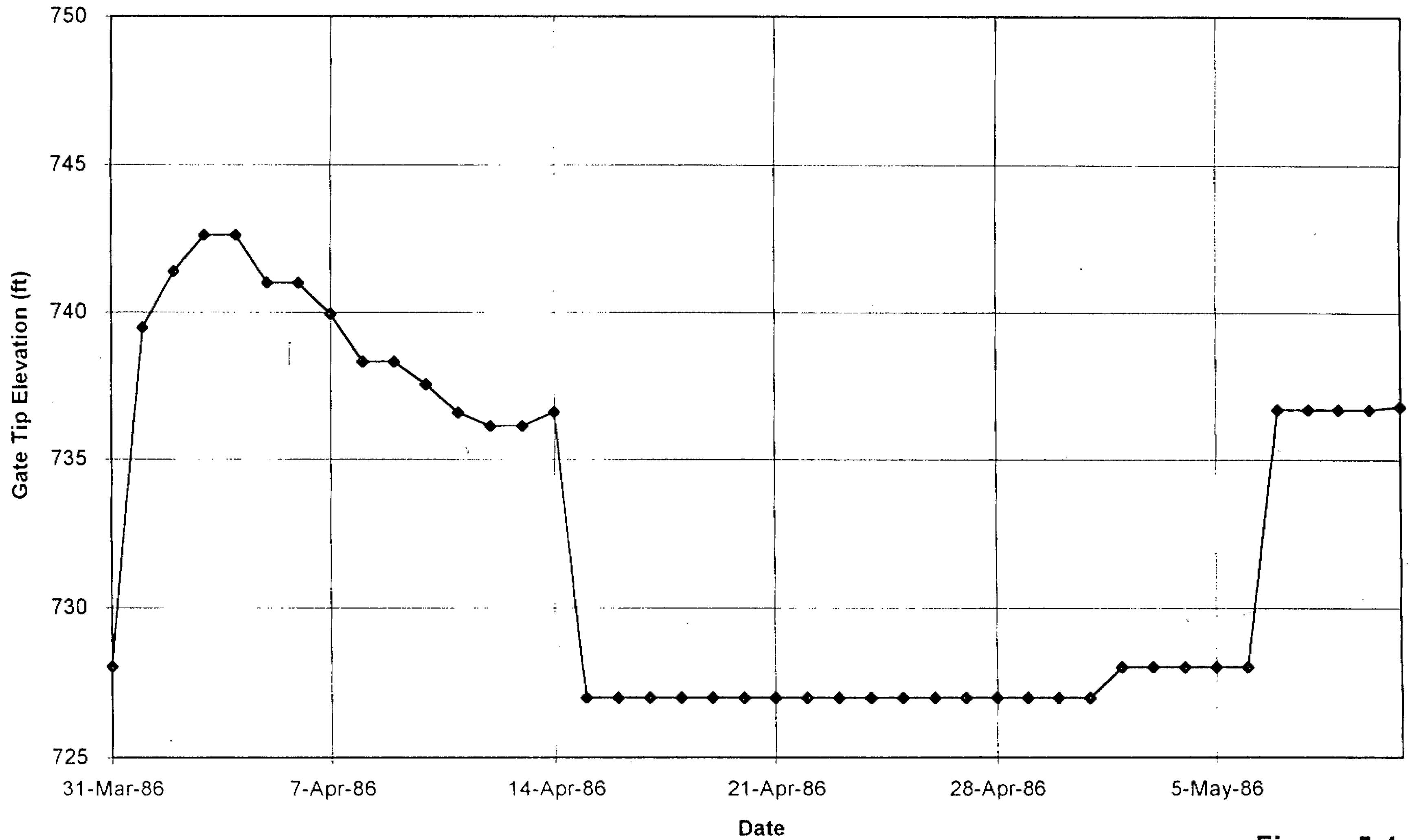


Figure 5.4 (f)

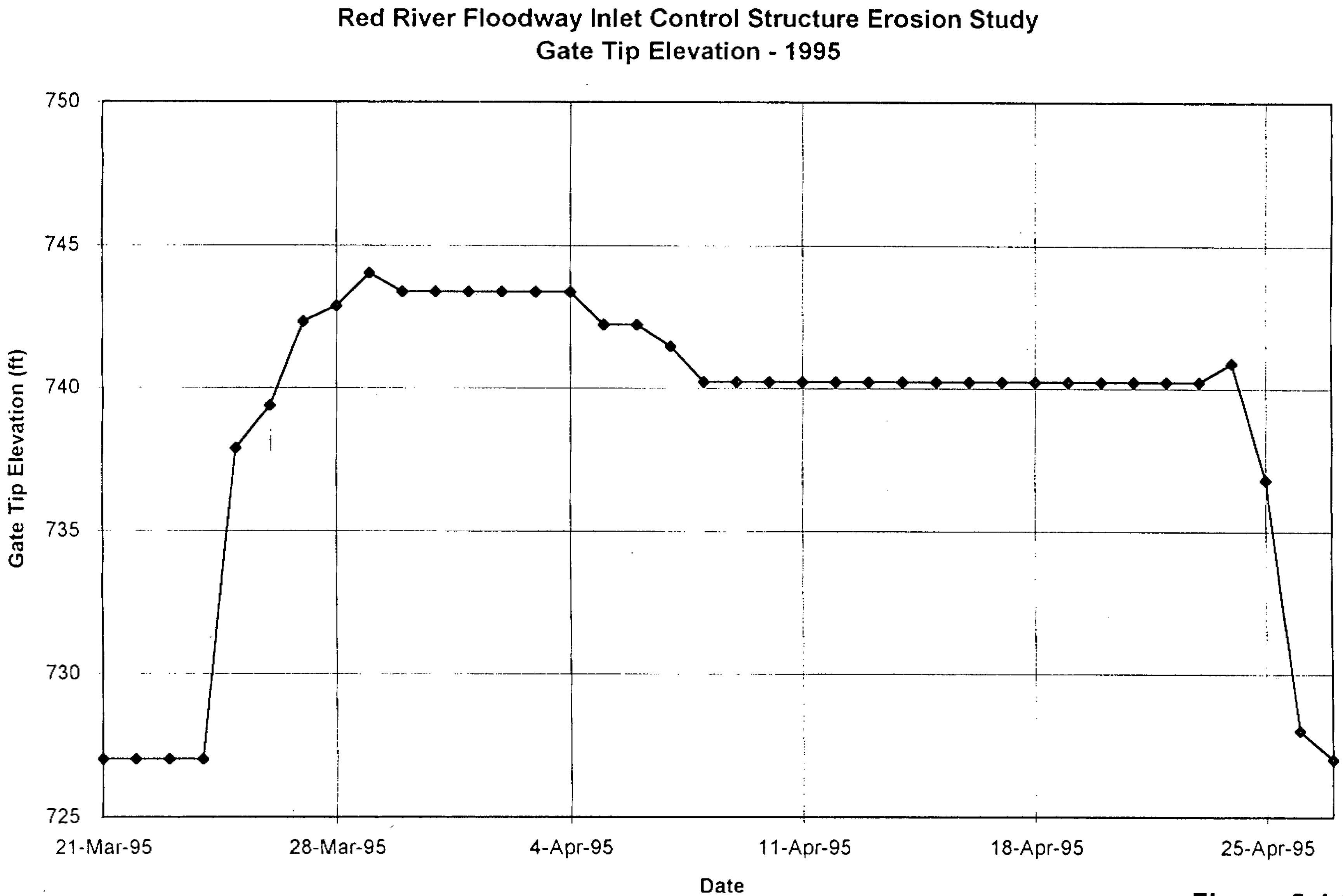

Red River Floodway Inlet Control Structure Erosion Study
Gate Tip Elevation - 1995
Gate Tip Elevation (ft)
750
745
740
735
730
725
21-Mar-95
28-Mar-95
4-Apr-95
11-Apr-95
18-Apr-95
25-Apr-95
Date

Figure 5.4 (g)

RED RIVER FLOODWAY INLET CONTROL STRUCTURE EROSION STUDY
Maximum Flow Velocity Downstream of Inlet Structure

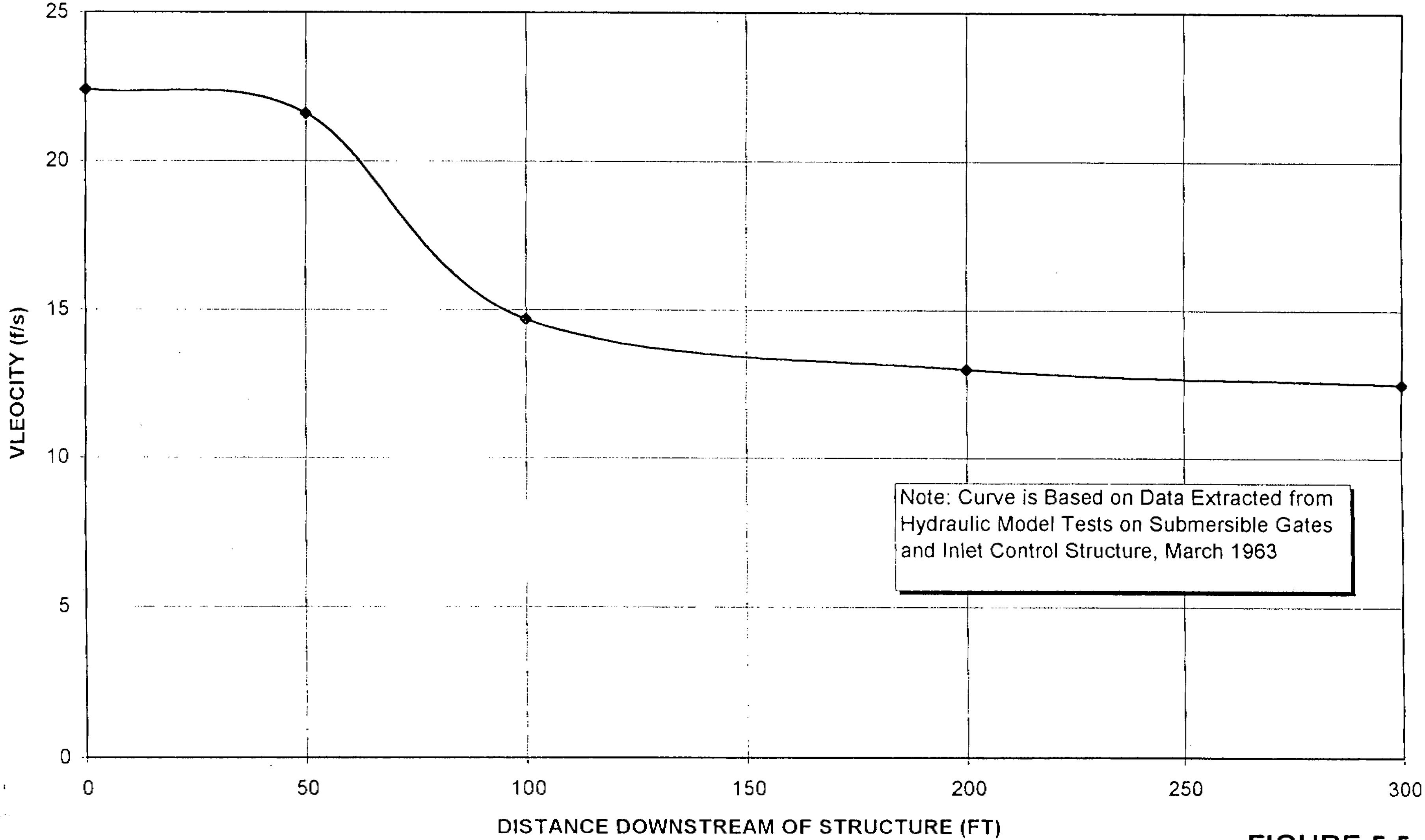


FIGURE 5.5

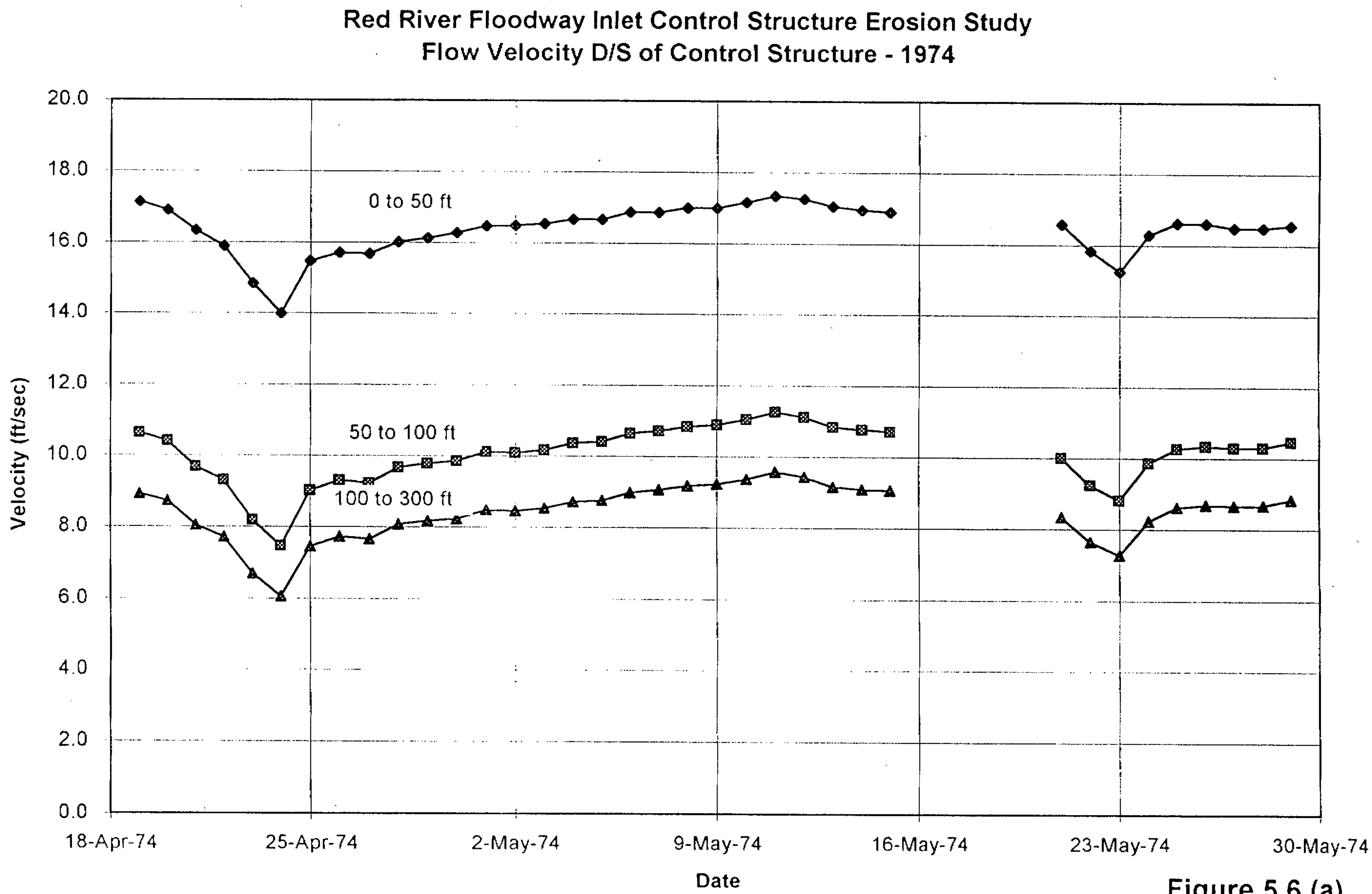

Red River Floodway Inlet Control Structure Erosion Study
Flow Velocity D/S of Control Structure - 1974
Velocity (ft/sec)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0 to 50 ft
50 to 100 ft
100 to 300 ft
18-Apr-74
25-Apr-74
2-May-74
9-May-74
16-May-74
23-May-74
30-May-74
Date

Figure 5.6 (a)

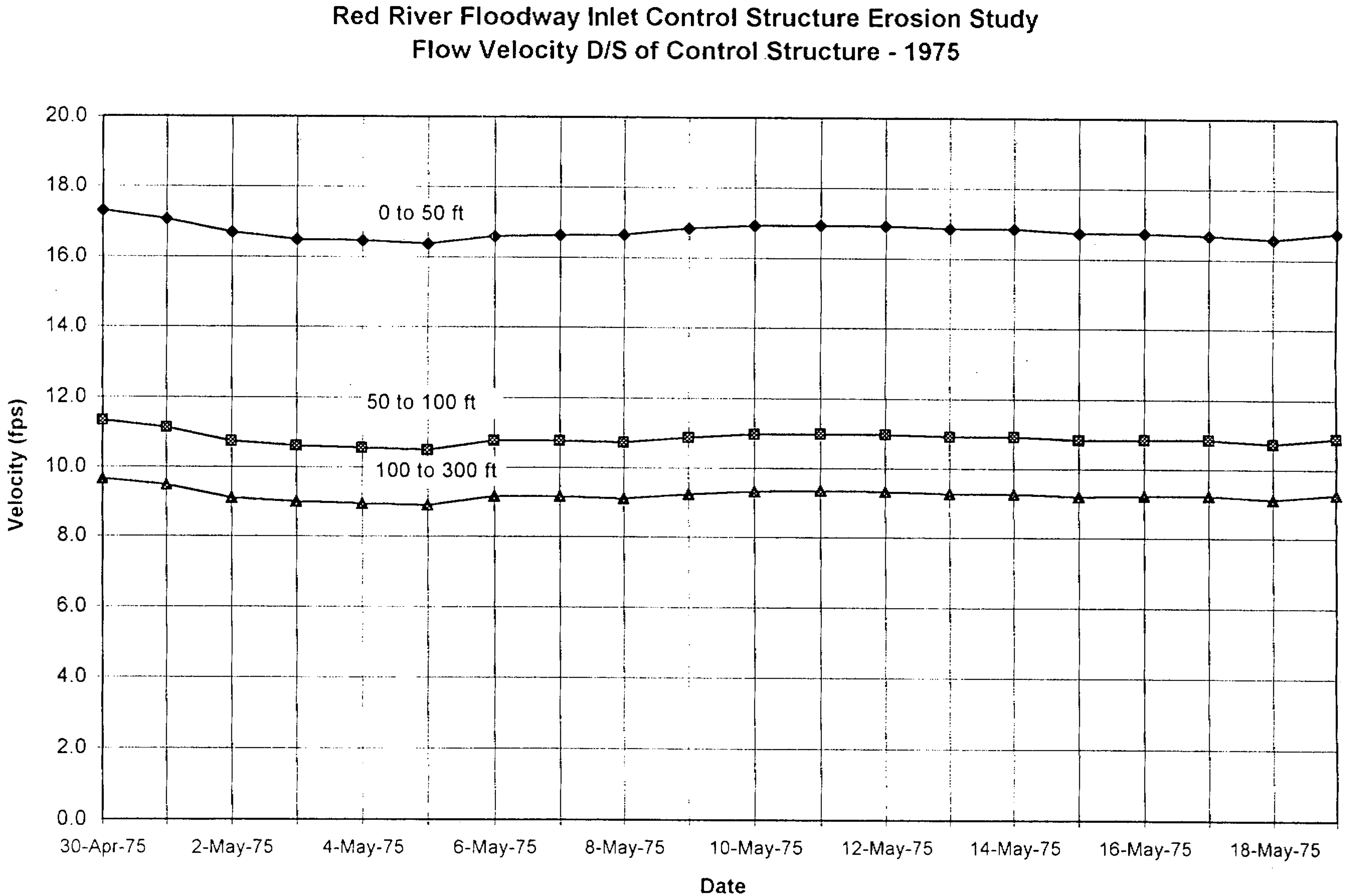
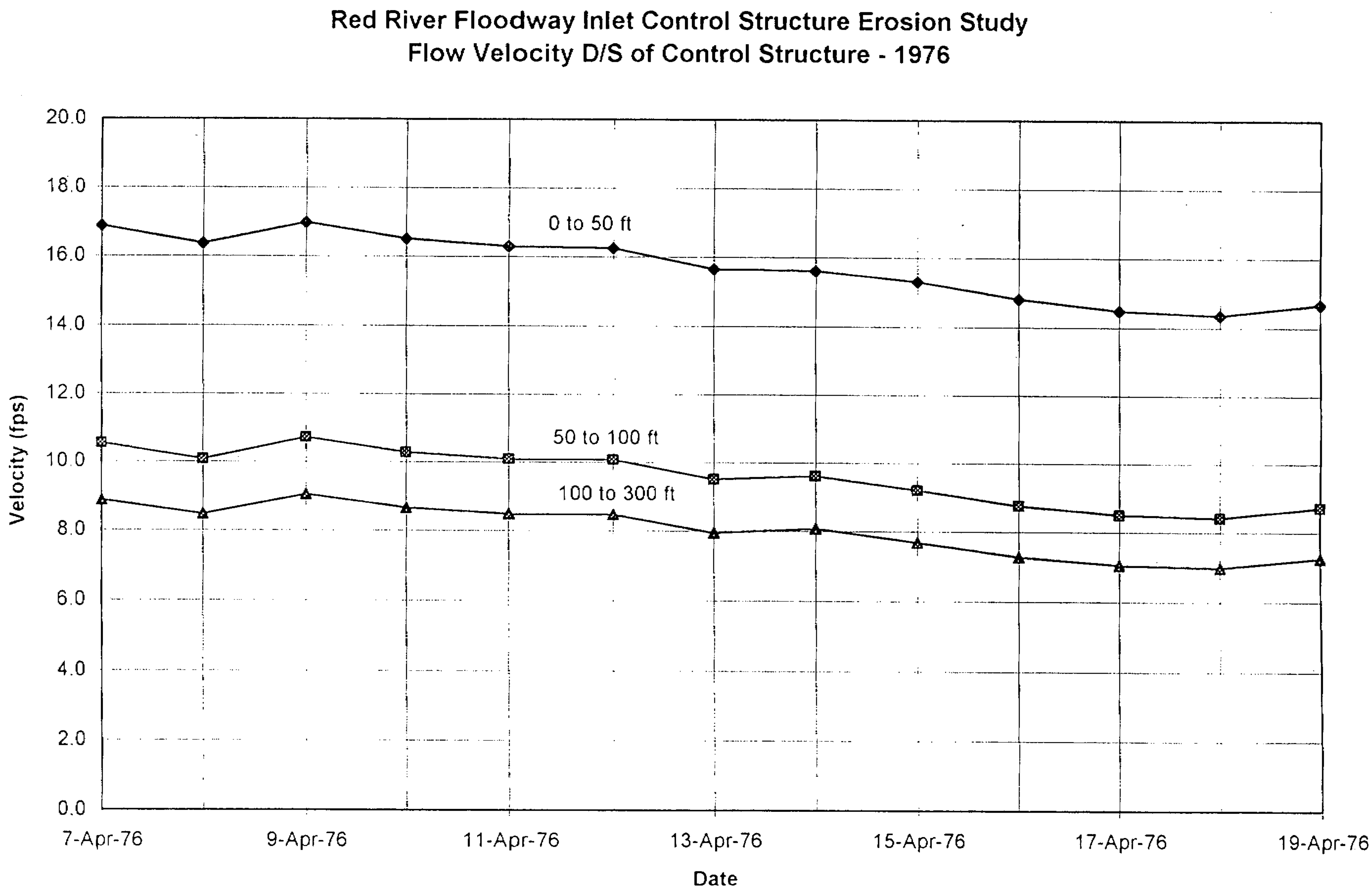

Red River Floodway Inlet Control Structure Erosion Study
Flow Velocity D/S of Control Structure - 1975
0 to 50 ft
50 to 100 ft
100 to 300 ft
Velocity (fps)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
30-Apr-75
2-May-75
4-May-75
6-May-75
8-May-75
10-May-75
12-May-75
14-May-75
16-May-75
18-May-75
Date

Figure 5.6 (b)



Red River Floodway Inlet Control Structure Erosion Study
Flow Velocity D/S of Control Structure - 1976
Velocity (fps)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0 to 50 ft
50 to 100 ft
100 to 300 ft
7-Apr-76
9-Apr-76
11-Apr-76
13-Apr-76
15-Apr-76
17-Apr-76
19-Apr-76
Date

Figure 5.6 (c)

Red River Floodway Inlet Control Structure Erosion Study
Flow Velocity D/S of Control Structure - 1978

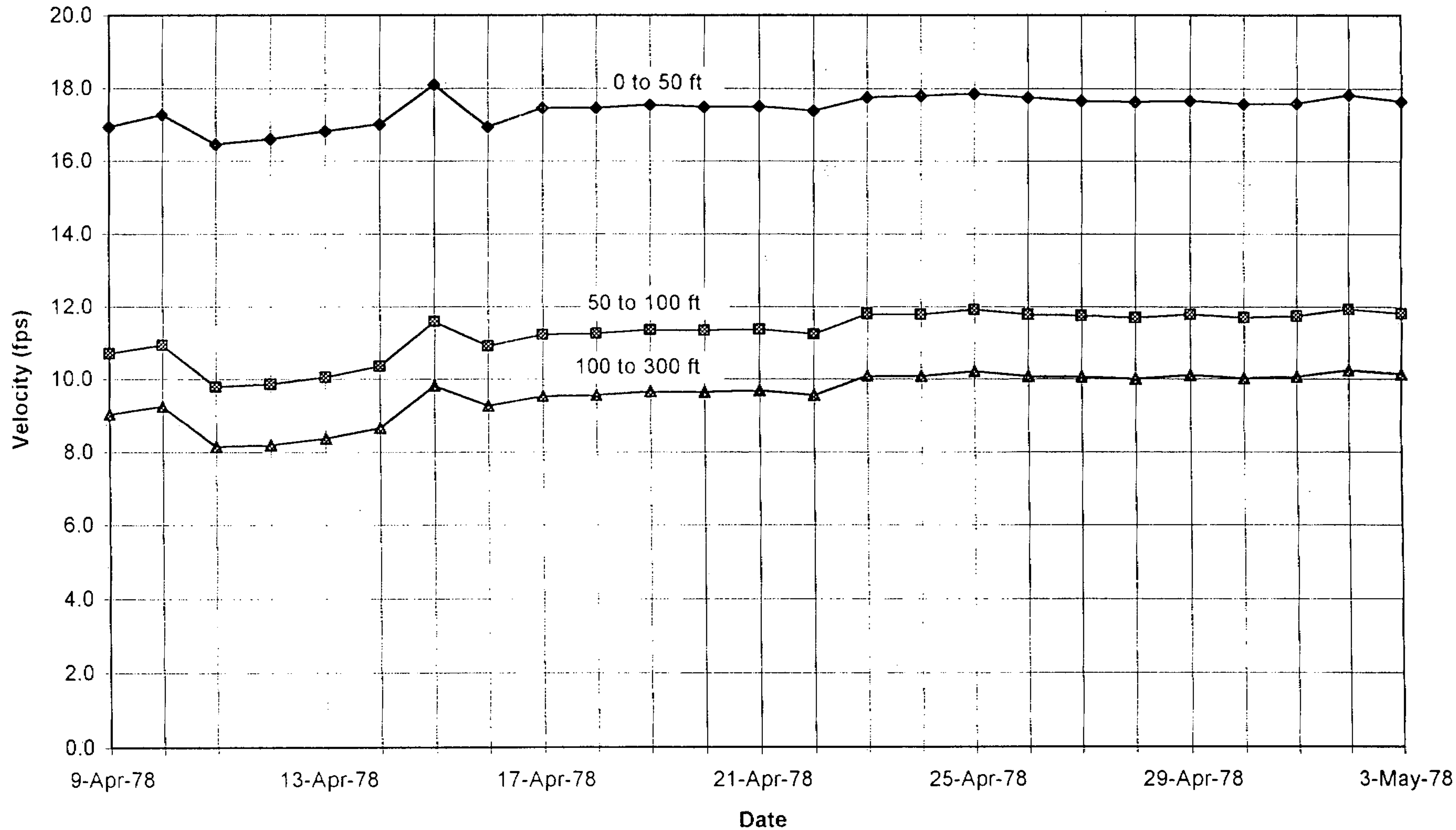
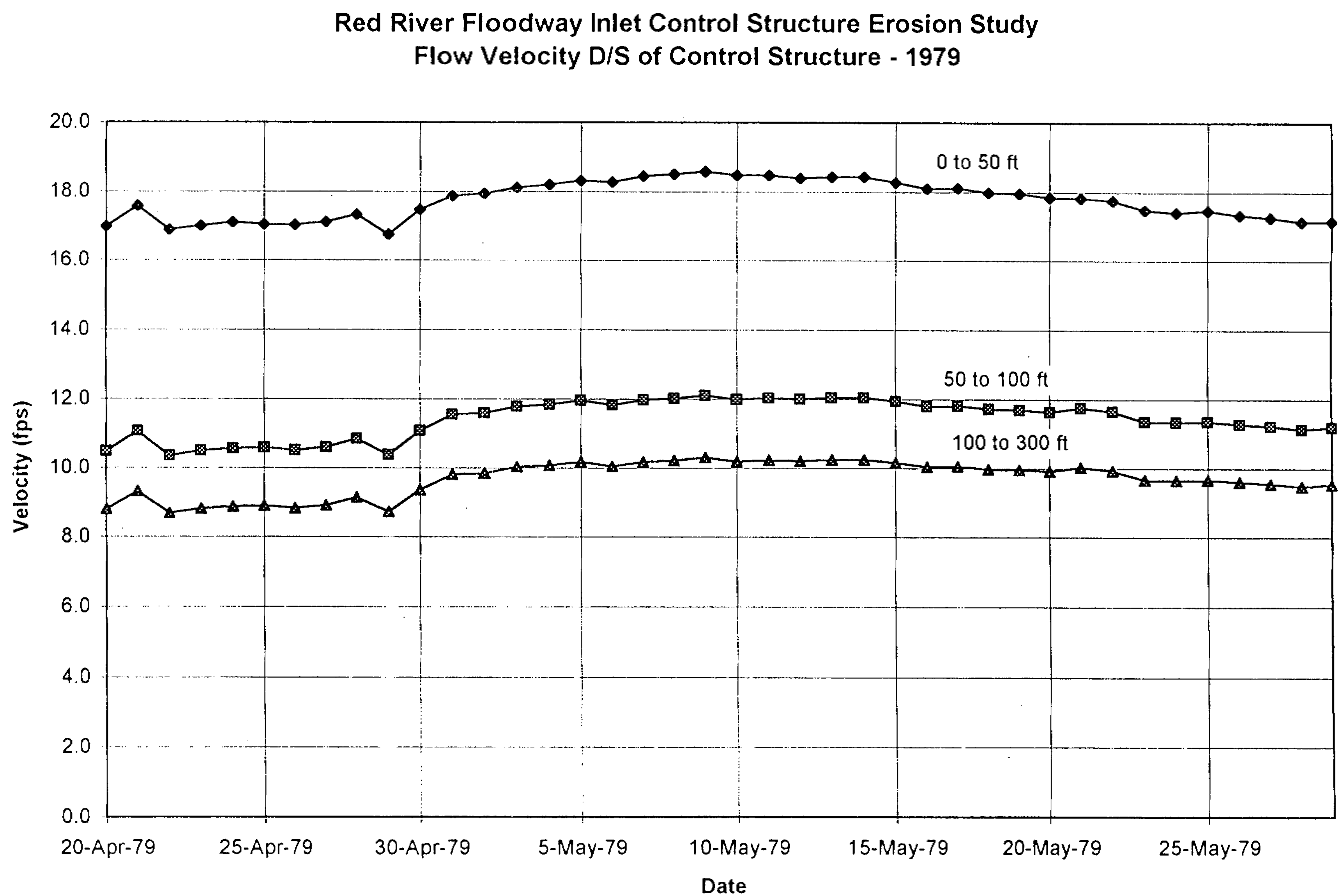


Figure 5.6 (d)



Red River Floodway Inlet Control Structure Erosion Study
Flow Velocity D/S of Control Structure - 1979
Velocity (fps)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0 to 50 ft
50 to 100 ft
100 to 300 ft
20-Apr-79
25-Apr-79
30-Apr-79
5-May-79
10-May-79
15-May-79
20-May-79
25-May-79
Date

Figure 5.6 (e)

Red River Floodway Inlet Control Structure Erosion Study
Flow Velocity D/S of Control Structure - 1986

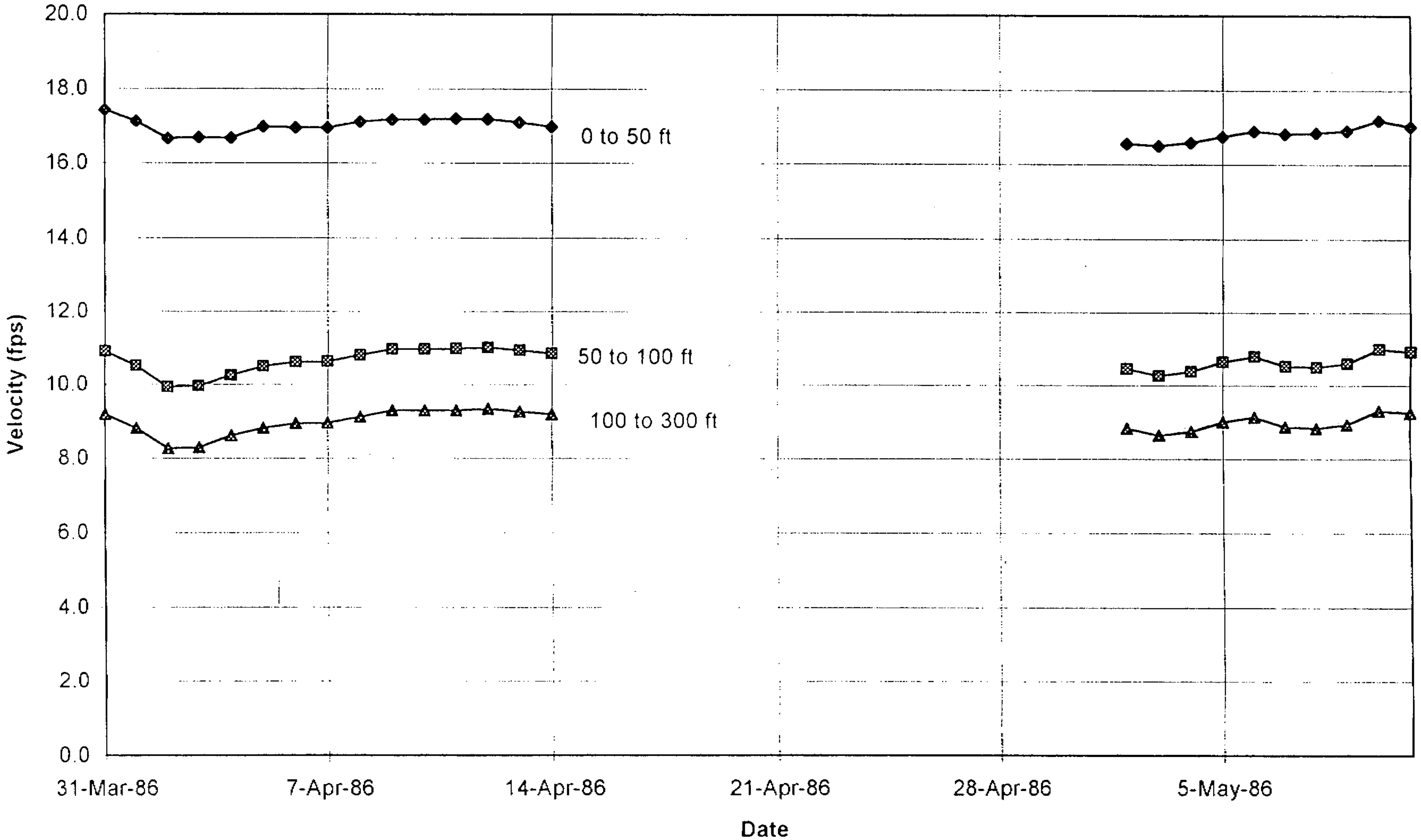
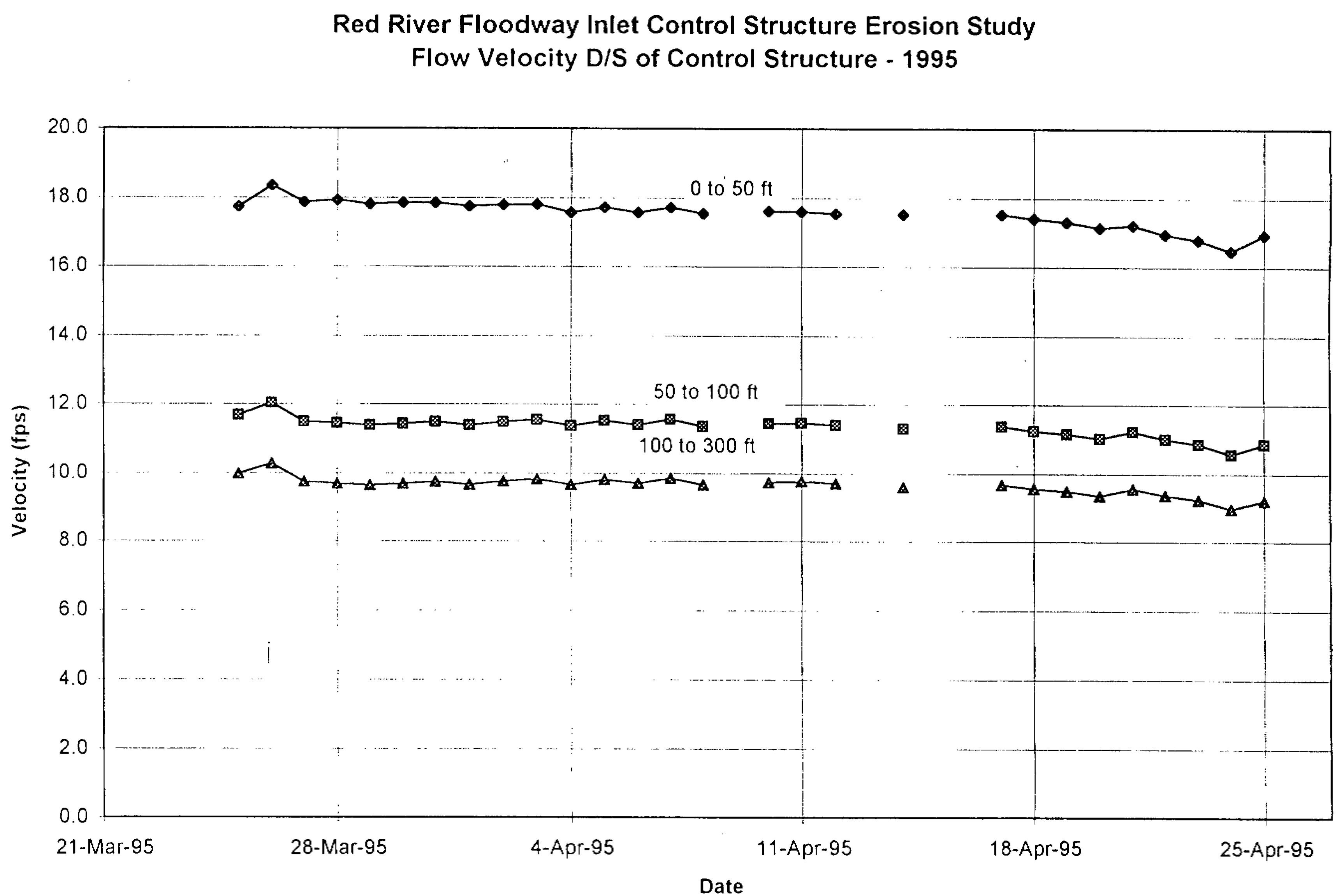


Figure 5.6 (f)



Red River Floodway Inlet Control Structure Erosion Study
Flow Velocity D/S of Control Structure - 1995
Velocity (fps)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0 to 50 ft
50 to 100 ft
100 to 300 ft
21-Mar-95
28-Mar-95
4-Apr-95
11-Apr-95
18-Apr-95
25-Apr-95
Date

Figure 5.6 (g)

Red River Floodway Inlet Control Structure Erosion Study
Corps of Engineers Rip Rap Size

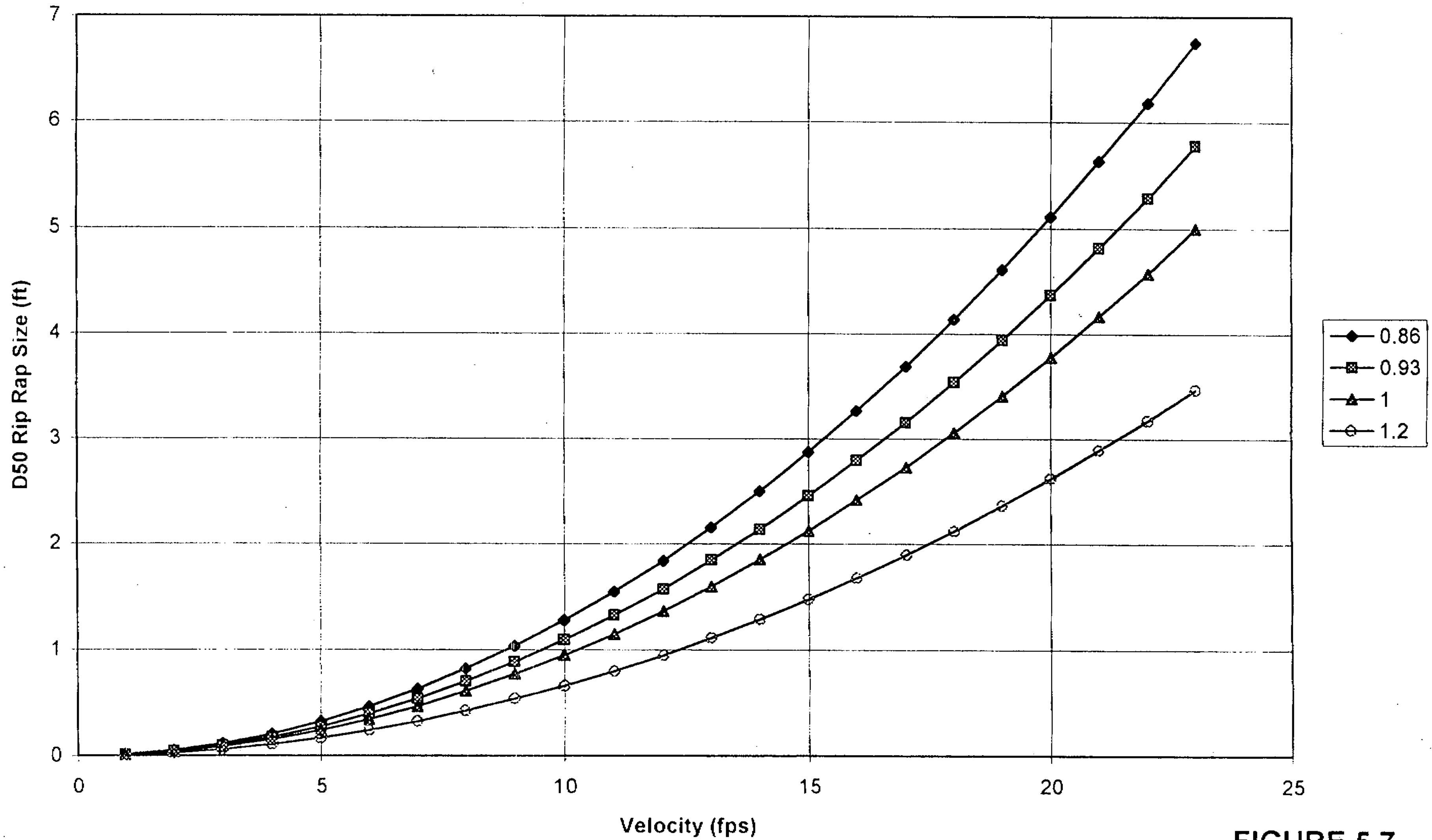
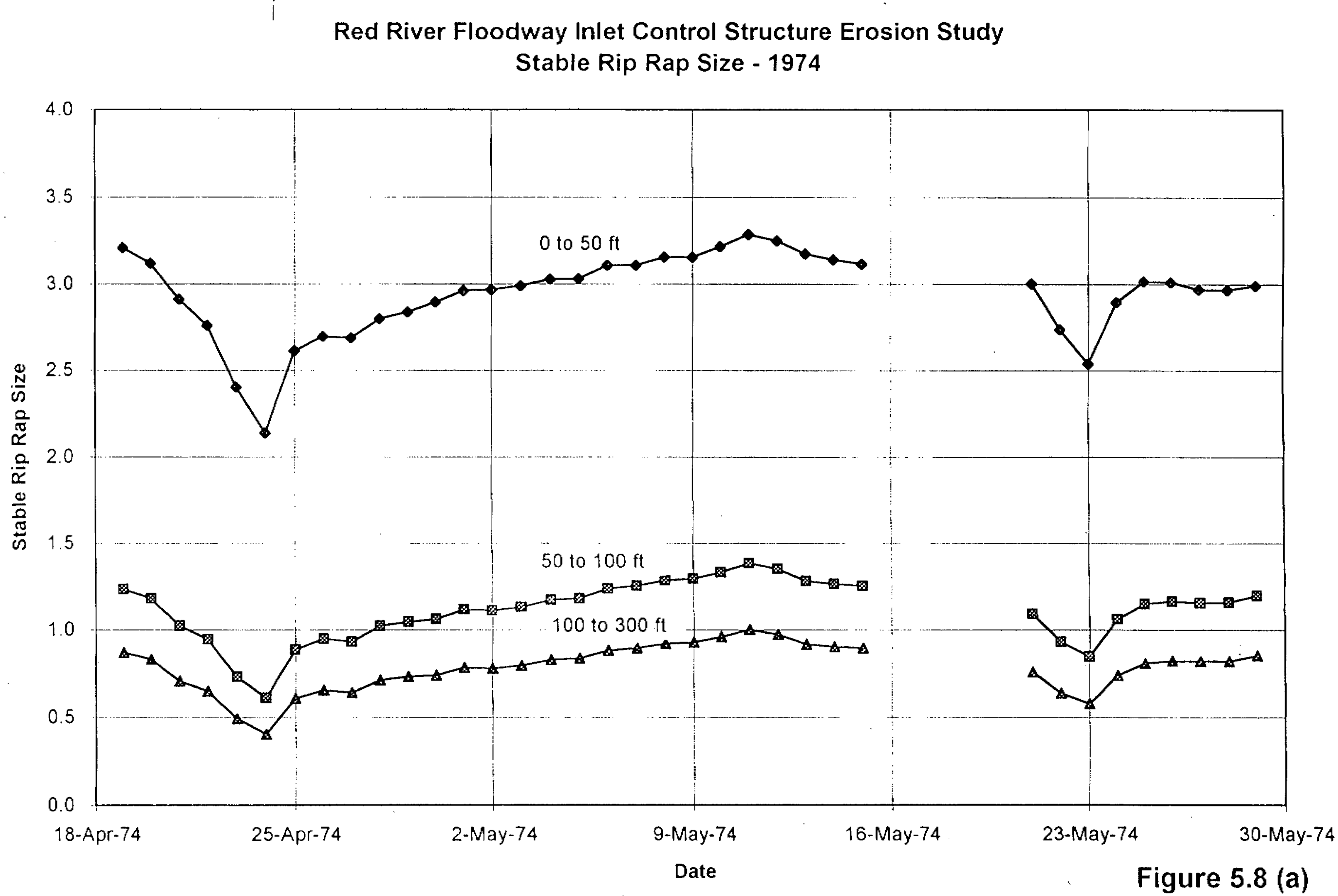


FIGURE 5.7



Red River Floodway Inlet Control Structure Erosion Study
Stable Rip Rap Size - 1974
Stable Rip Rap Size
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0 to 50 ft
50 to 100 ft
100 to 300 ft
18-Apr-74
25-Apr-74
2-May-74
9-May-74
16-May-74
23-May-74
30-May-74
Date

Figure 5.8 (a)

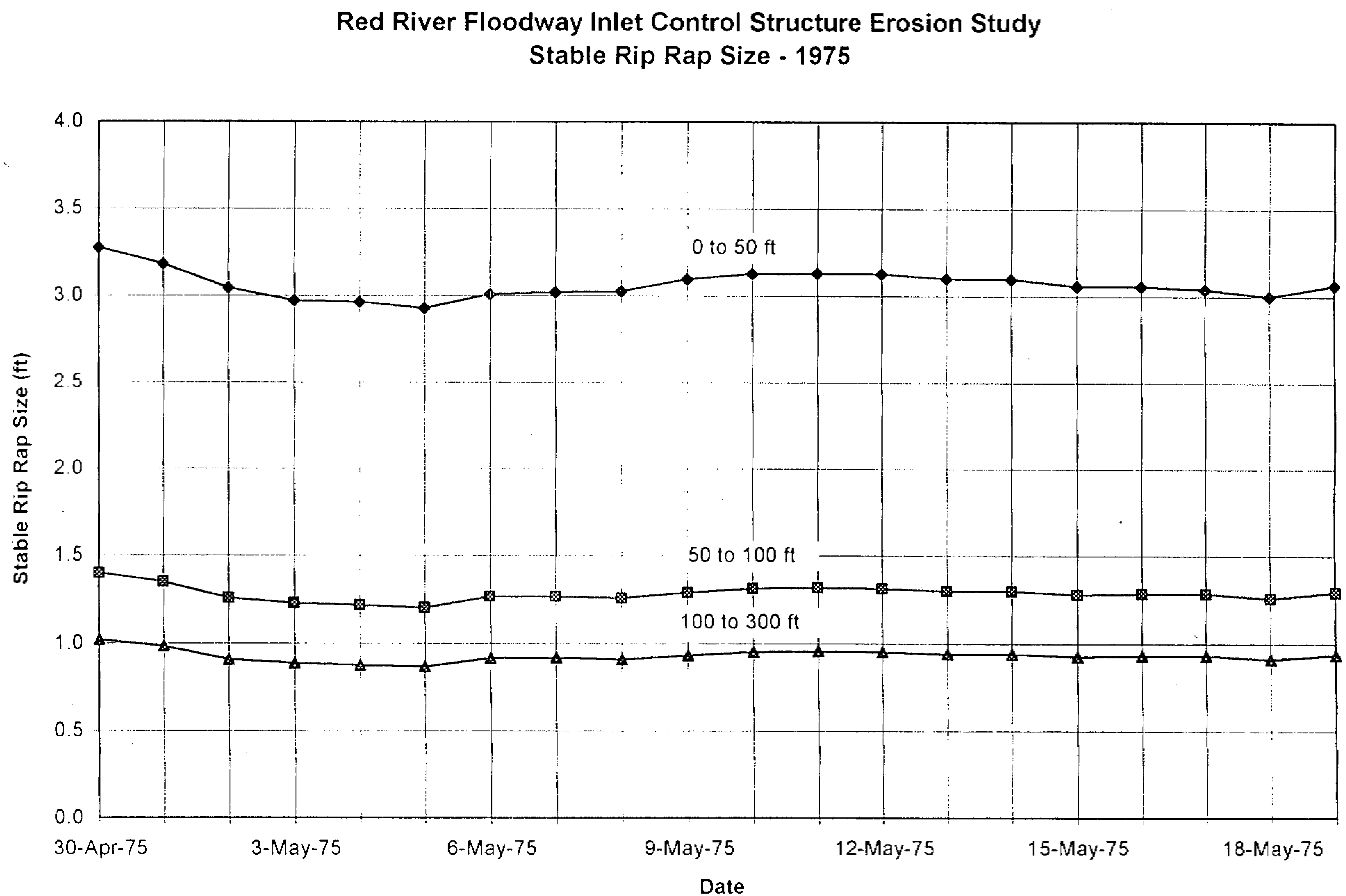

Red River Floodway Inlet Control Structure Erosion Study
Stable Rip Rap Size - 1975
0 to 50 ft
50 to 100 ft
100 to 300 ft
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Stable Rip Rap Size (ft)
30-Apr-75
3-May-75
6-May-75
9-May-75
12-May-75
15-May-75
18-May-75
Date

Figure 5.8 (b)

Red River Floodway Inlet Control Structure Erosion Study
Stable Rip Rap Size - 1976

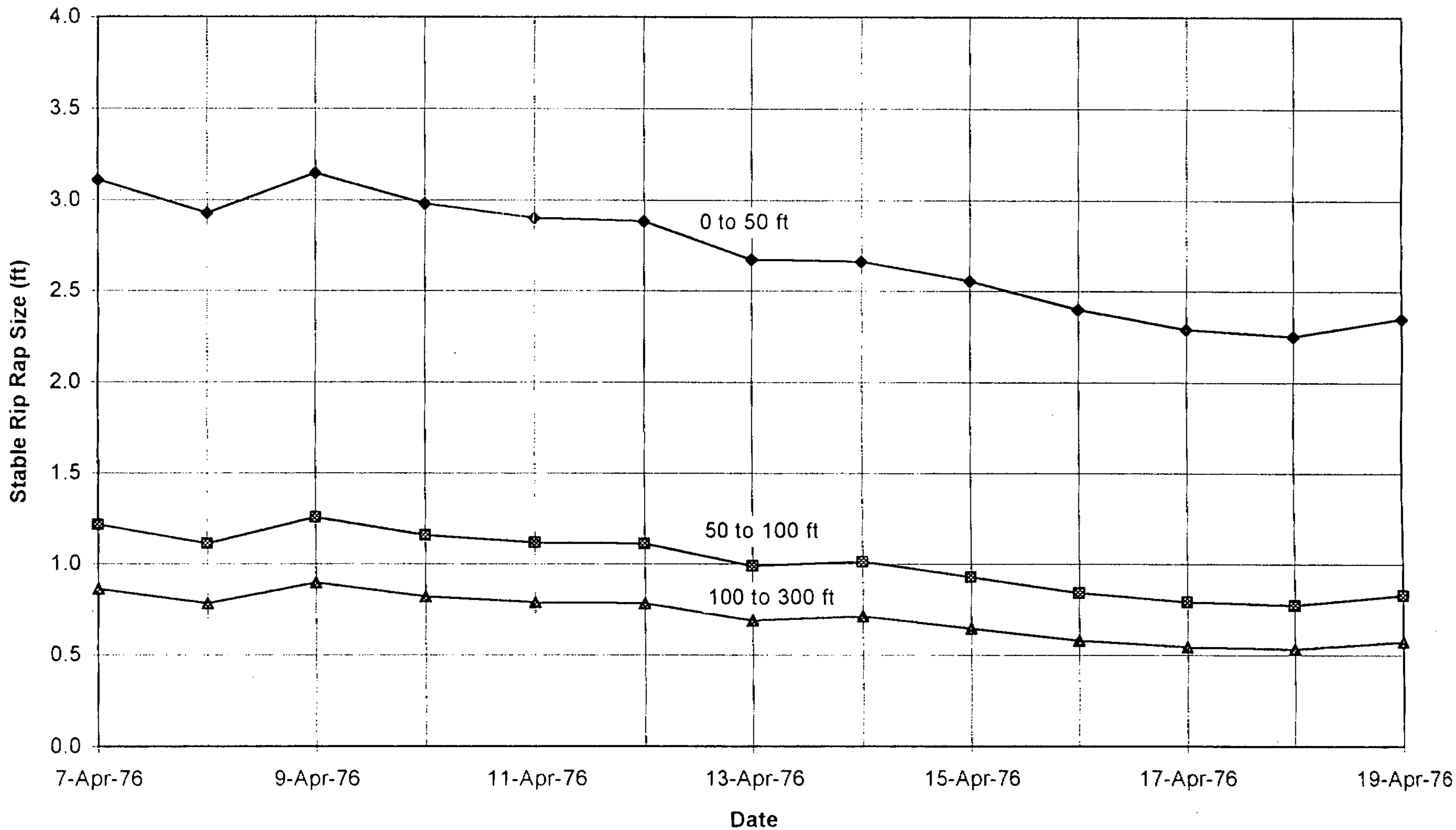


Figure 5.8 (c)

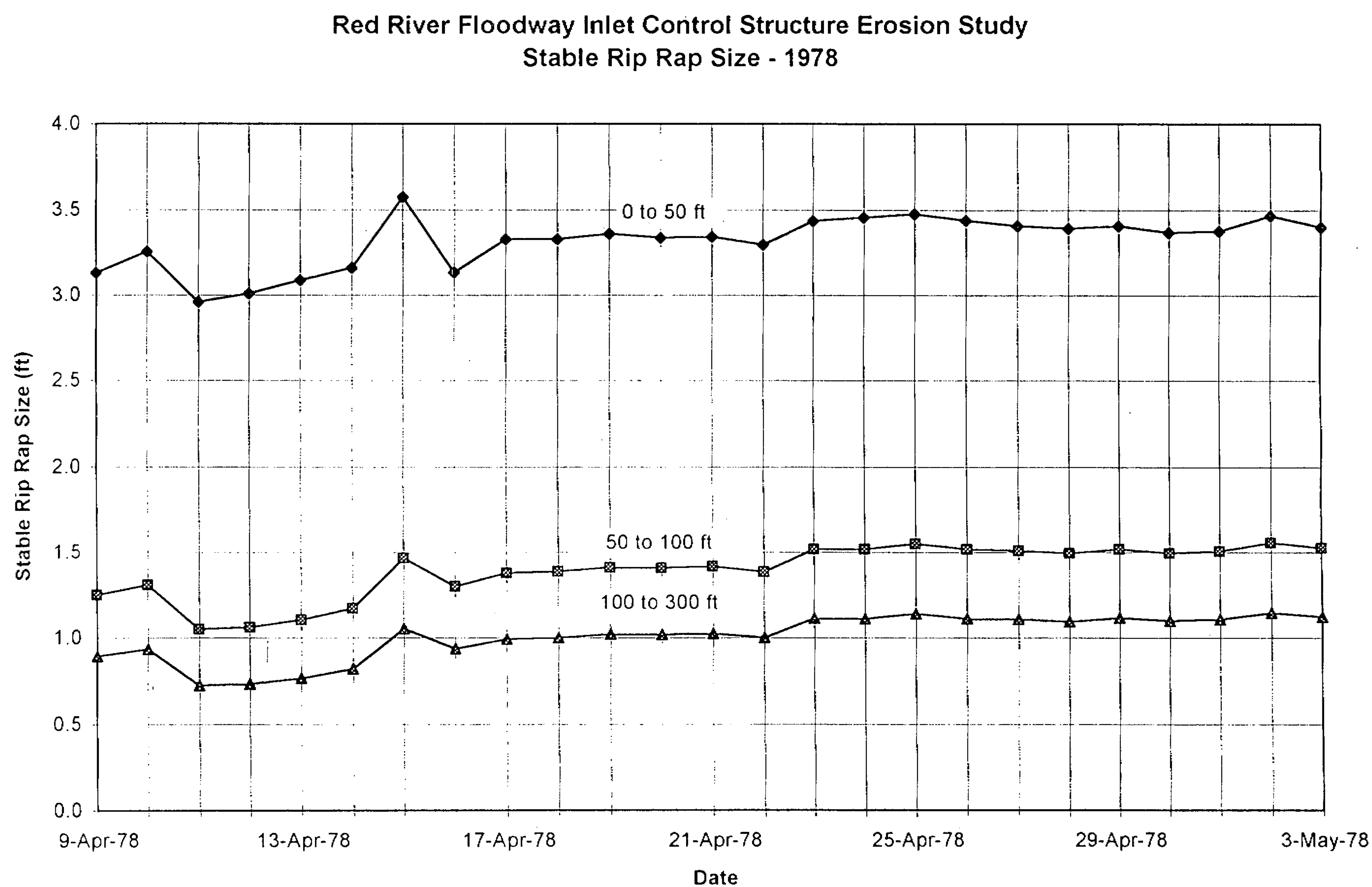
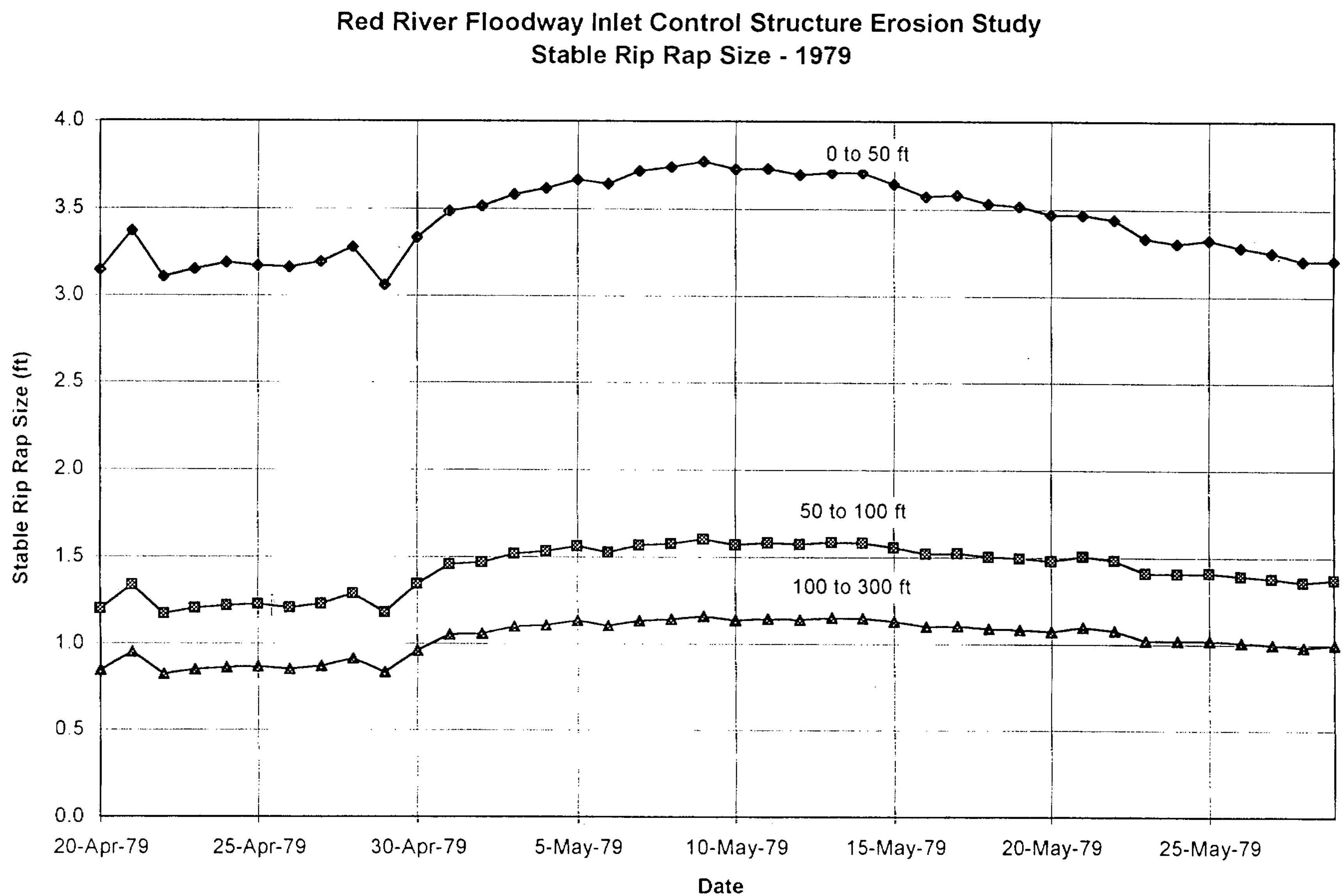

Red River Floodway Inlet Control Structure Erosion Study
Stable Rip Rap Size - 1978
0 to 50 ft
50 to 100 ft
100 to 300 ft
Stable Rip Rap Size (ft)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
9-Apr-78
13-Apr-78
17-Apr-78
21-Apr-78
25-Apr-78
29-Apr-78
3-May-78
Date

Figure 5.8 (d)



Red River Floodway Inlet Control Structure Erosion Study
Stable Rip Rap Size - 1979
Stable Rip Rap Size (ft)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0 to 50 ft
50 to 100 ft
100 to 300 ft
20-Apr-79
25-Apr-79
30-Apr-79
5-May-79
10-May-79
15-May-79
20-May-79
25-May-79
Date

Figure 5.8 (e)

Red River Floodway Inlet Control Structure Erosion Study
Stable Rip Rap Size - 1986

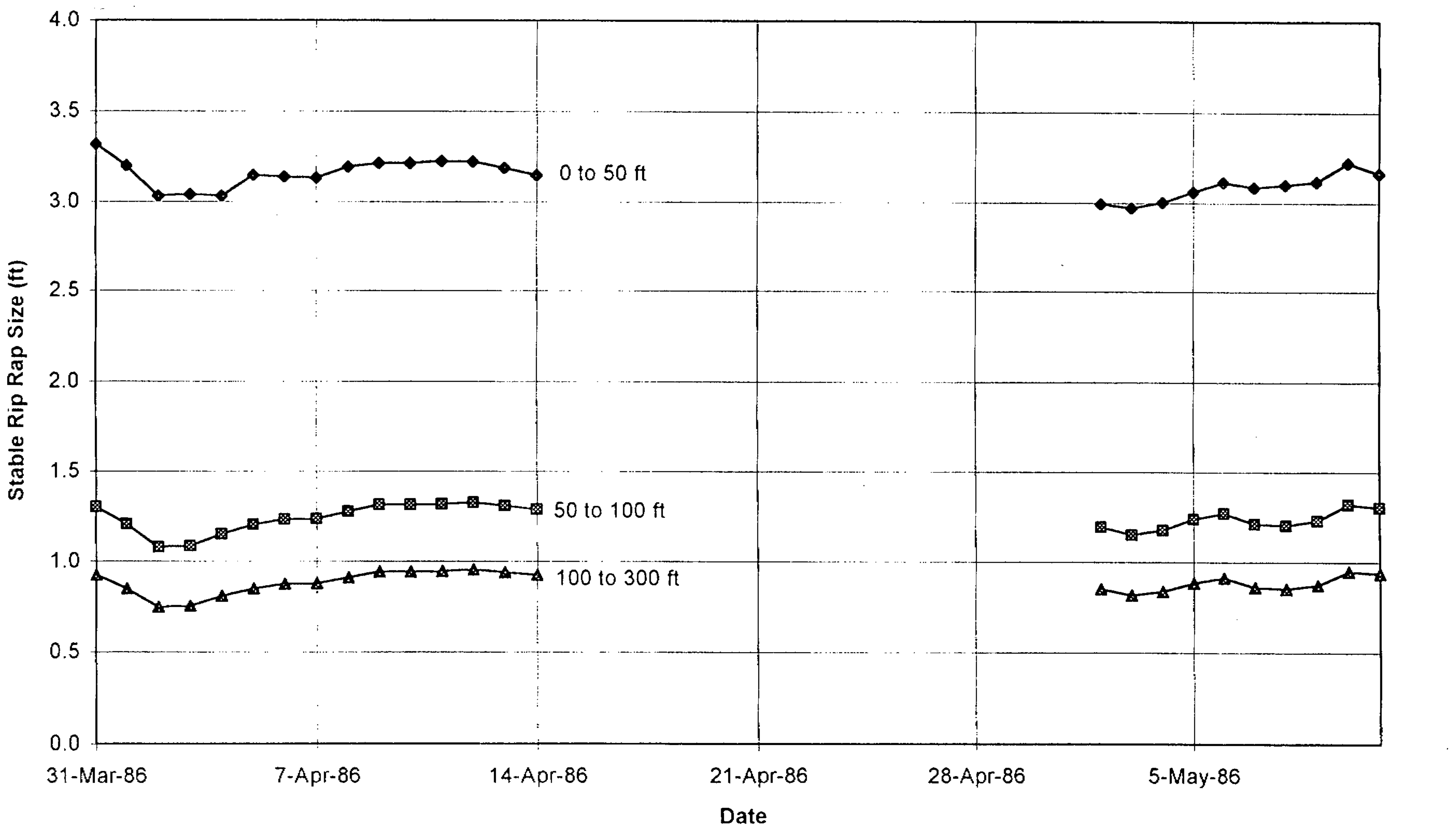
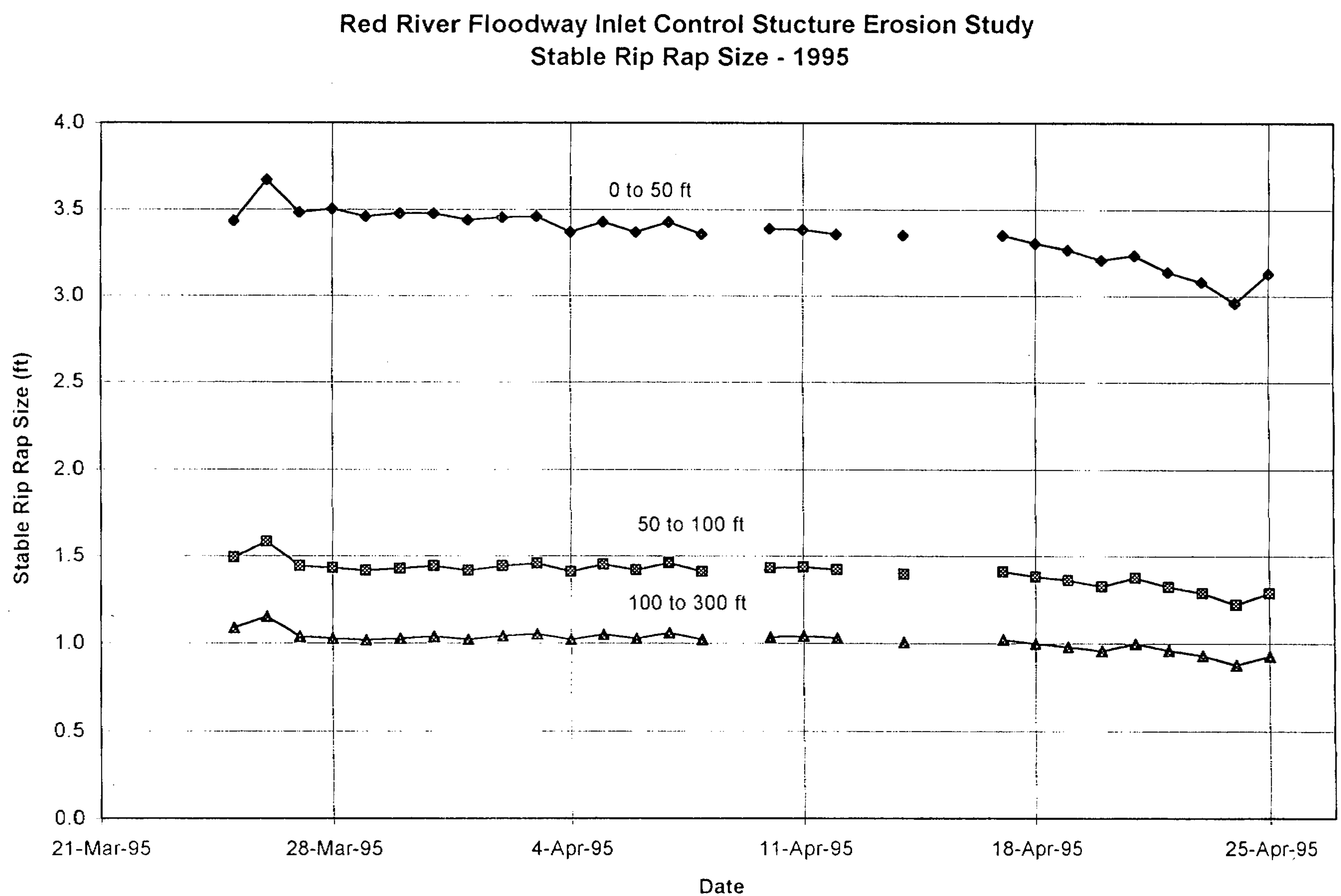


Figure 5.8 (f)



Red River Floodway Inlet Control Stucture Erosion Study
Stable Rip Rap Size - 1995
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Stable Rip Rap Size (ft)
0 to 50 ft
50 to 100 ft
100 to 300 ft
21-Mar-95
28-Mar-95
4-Apr-95
11-Apr-95
18-Apr-95
25-Apr-95
Date

Figure 5.8 (g)

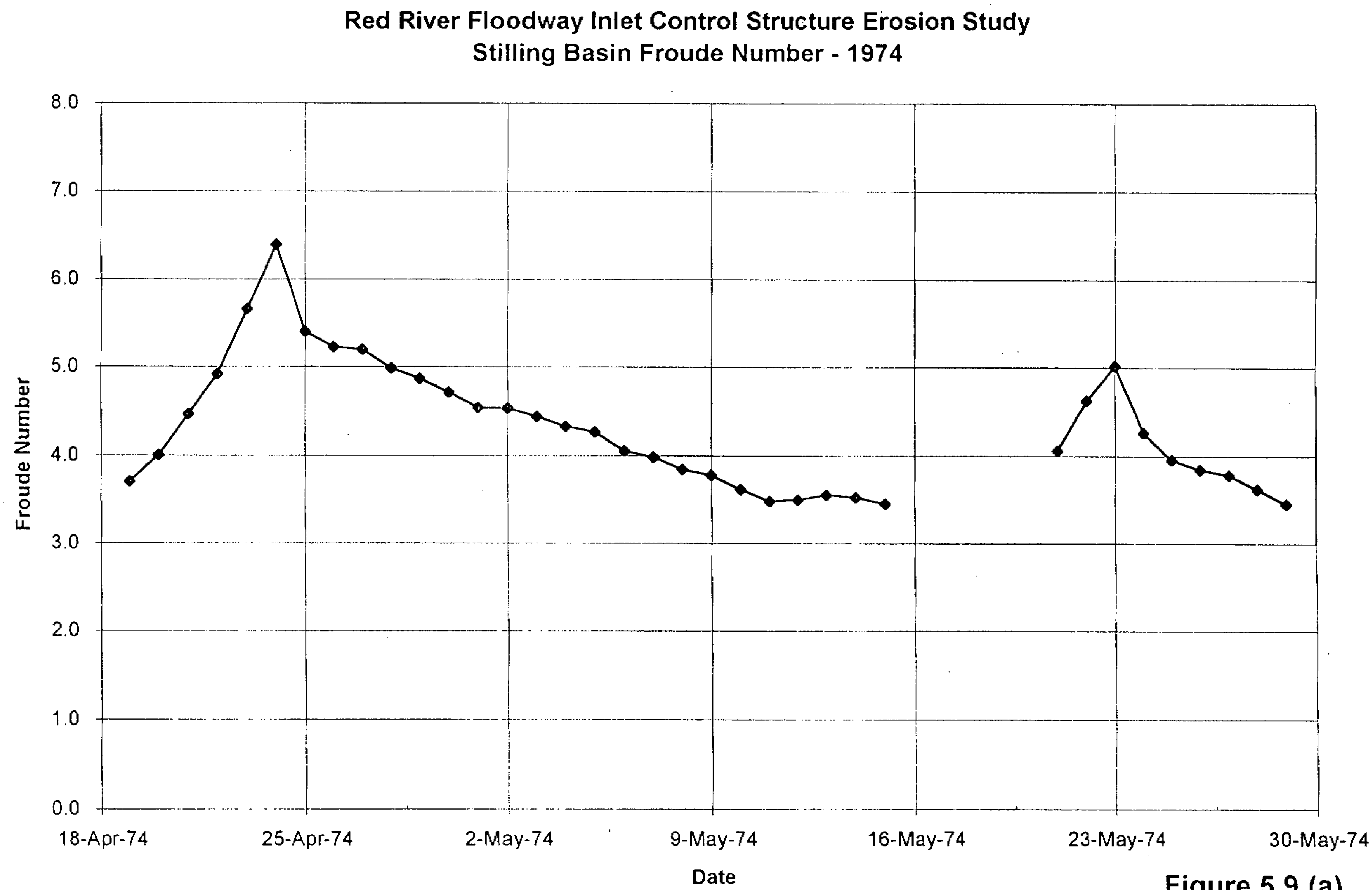

Red River Floodway Inlet Control Structure Erosion Study
Stilling Basin Froude Number - 1974
Froude Number
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
18-Apr-74
25-Apr-74
2-May-74
9-May-74
16-May-74
23-May-74
30-May-74
Date

Figure 5.9 (a)

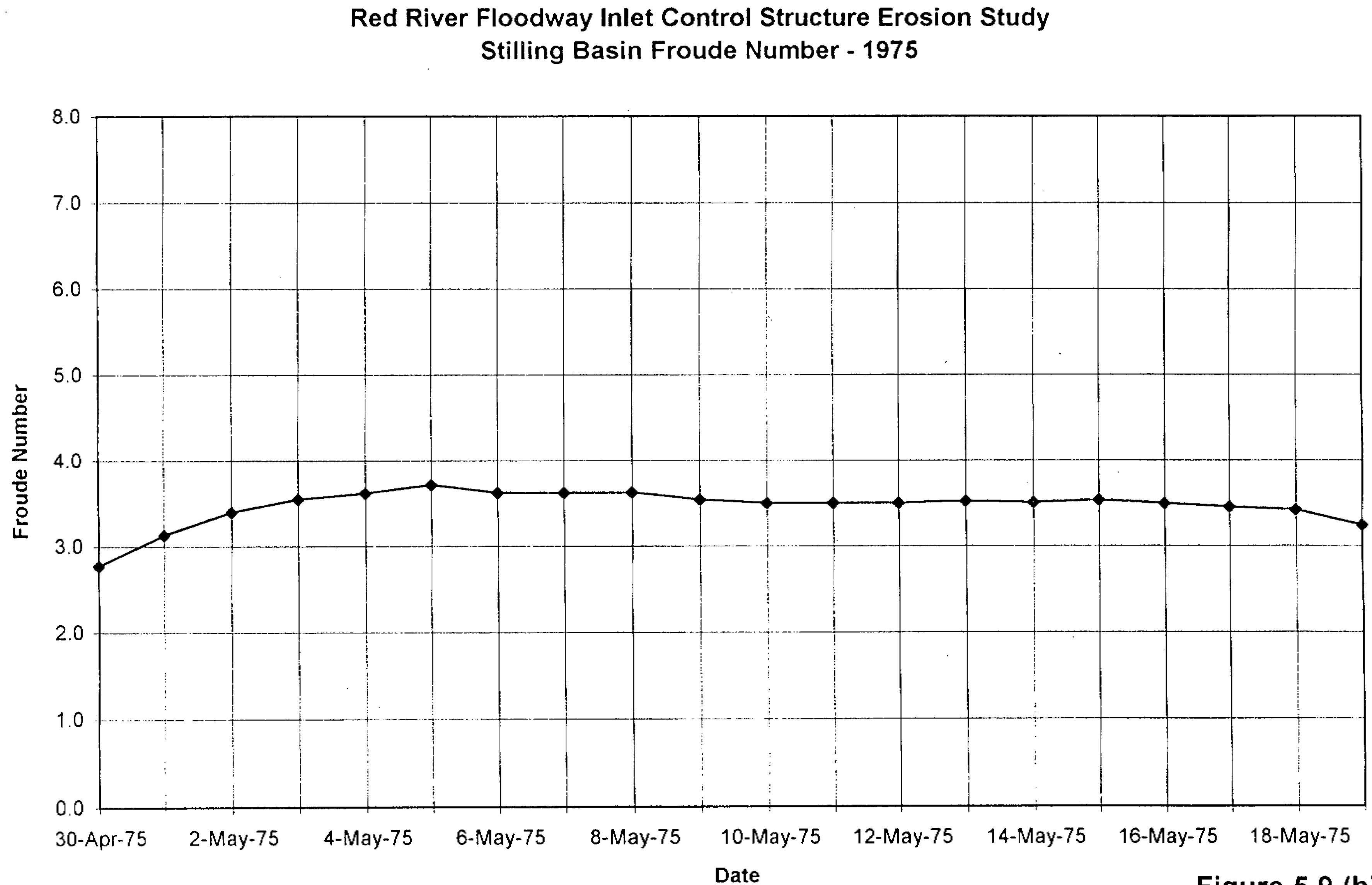
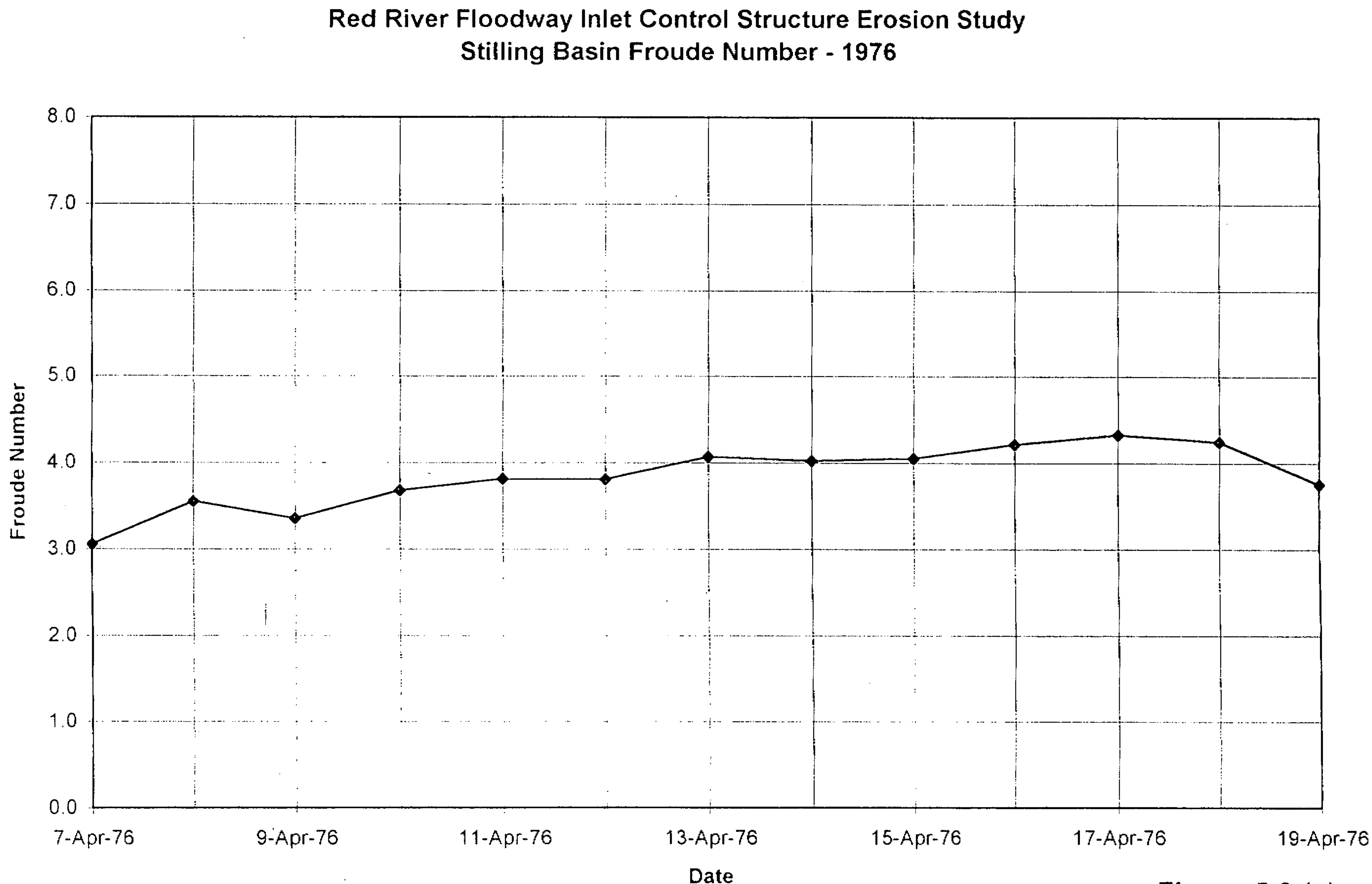

Red River Floodway Inlet Control Structure Erosion Study
Stilling Basin Froude Number - 1975
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Froude Number
30-Apr-75
2-May-75
4-May-75
6-May-75
8-May-75
10-May-75
12-May-75
14-May-75
16-May-75
18-May-75
Date

Figure 5.9 (b)



Red River Floodway Inlet Control Structure Erosion Study
Stilling Basin Froude Number - 1976
Froude Number
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
7-Apr-76
9-Apr-76
11-Apr-76
13-Apr-76
15-Apr-76
17-Apr-76
19-Apr-76
Date

Figure 5.9 (c)

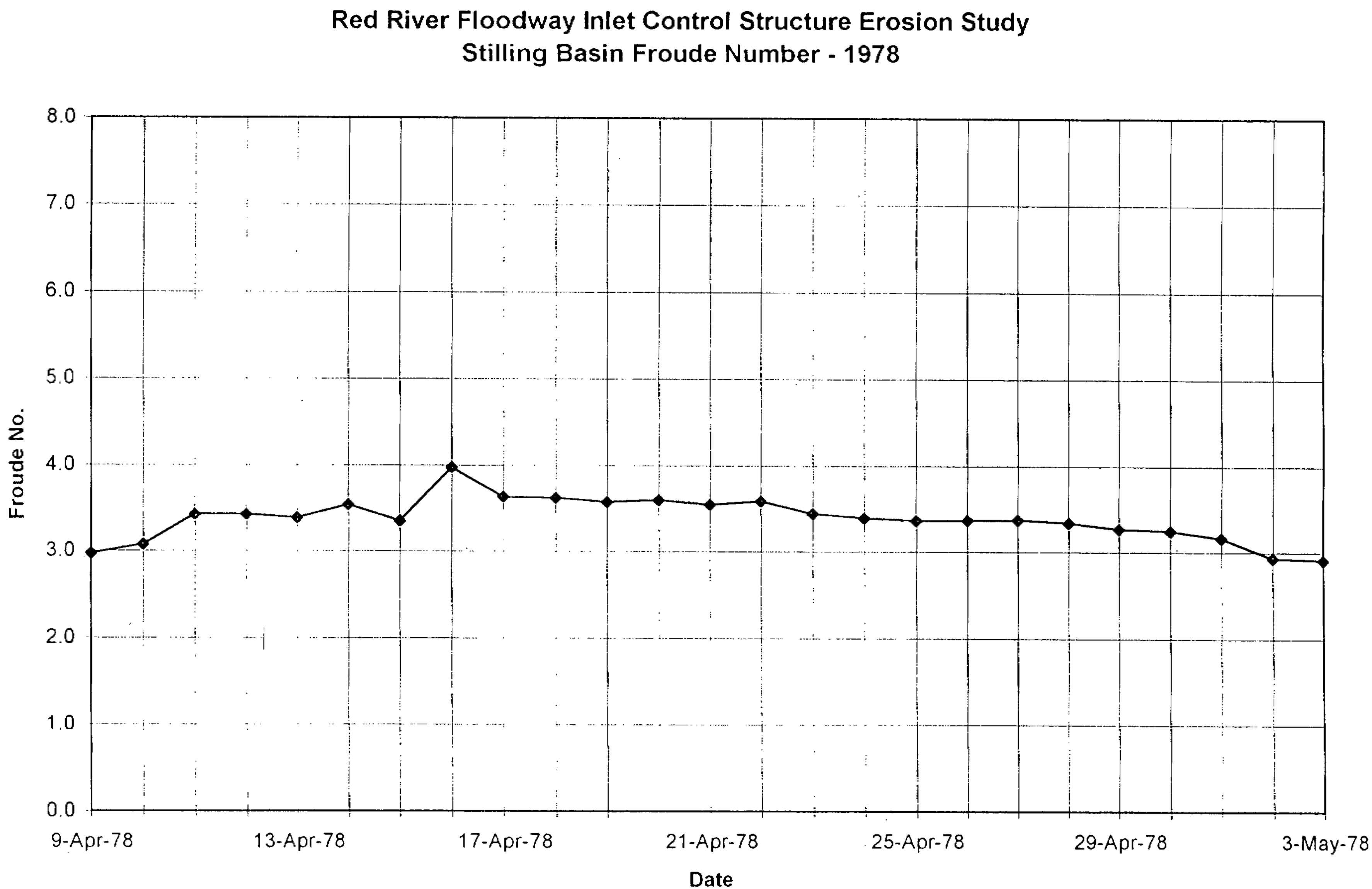

Red River Floodway Inlet Control Structure Erosion Study
Stilling Basin Froude Number - 1978
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Froude No.
9-Apr-78
13-Apr-78
17-Apr-78
21-Apr-78
25-Apr-78
29-Apr-78
3-May-78
Date

Figure 5.9 (d)

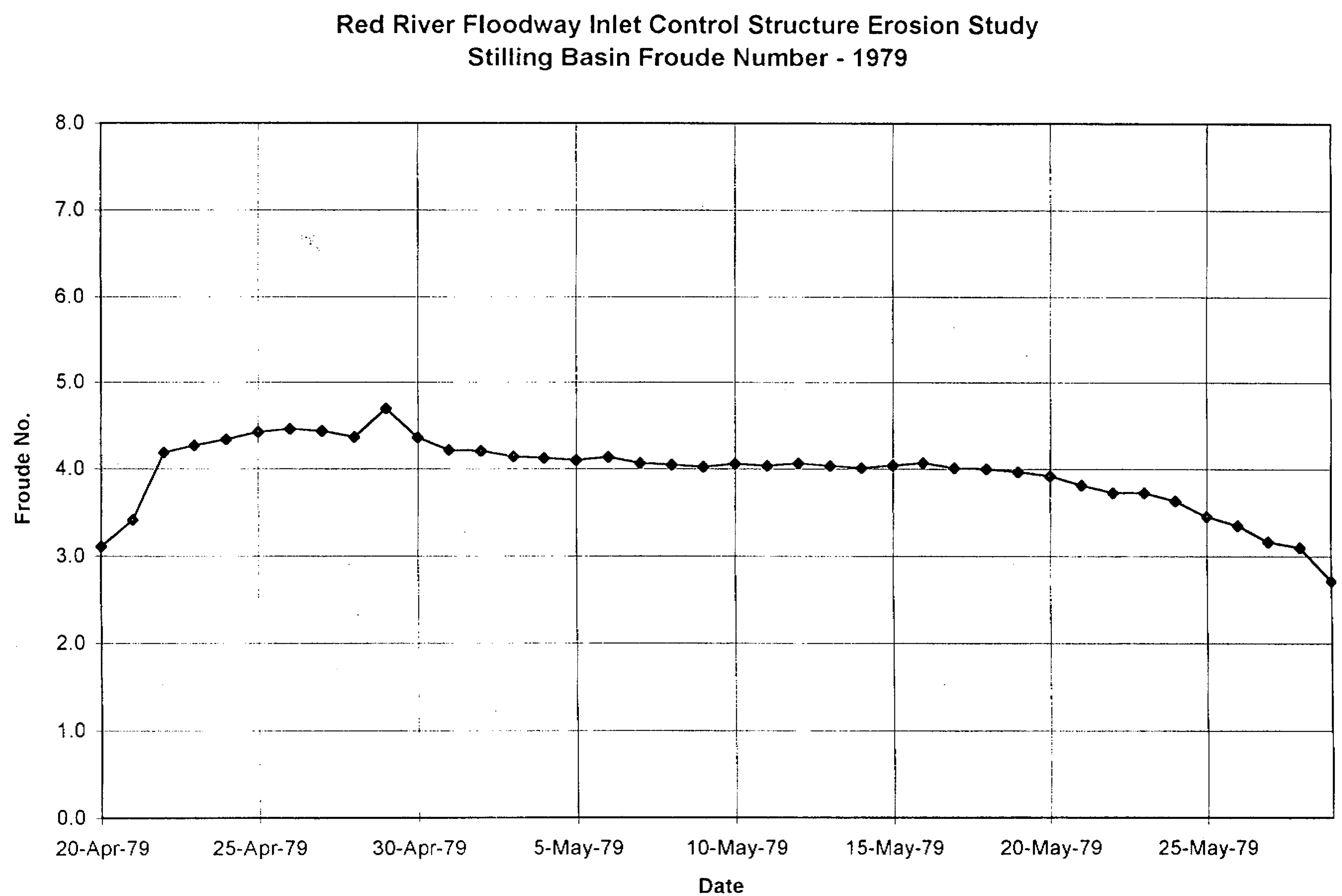

Red River Floodway Inlet Control Structure Erosion Study
Stilling Basin Froude Number - 1979
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Froude No.
20-Apr-79
25-Apr-79
30-Apr-79
5-May-79
10-May-79
15-May-79
20-May-79
25-May-79
Date

Figure 5.9 (e)

Red River Floodway Inlet Control Structure Erosion Study
Stilling Basin Froude Number - 1986

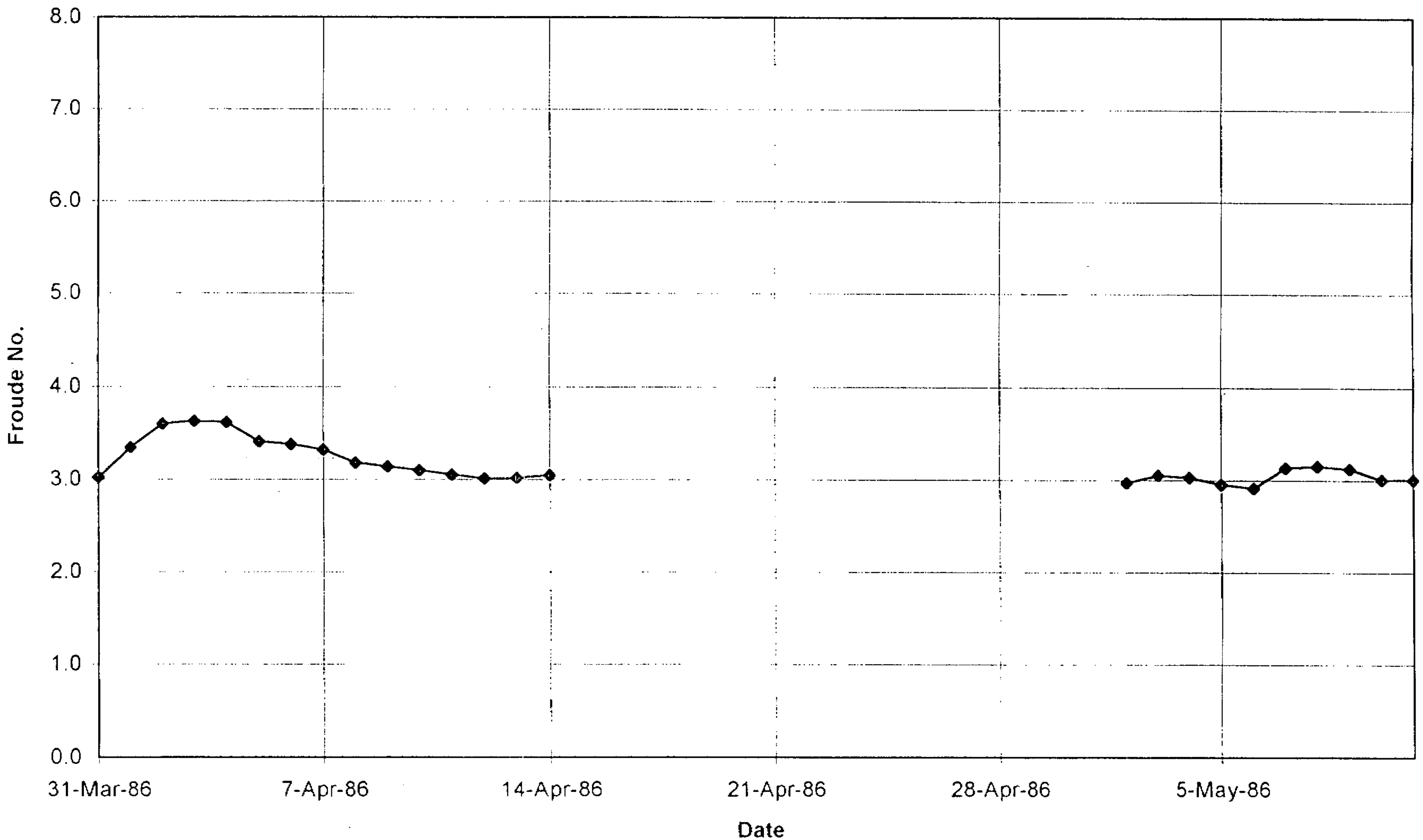


Figure 5.9 (f)

Red River Floodway Inlet Control Structure Erosion Study
Stilling Basin Froude Number - 1995

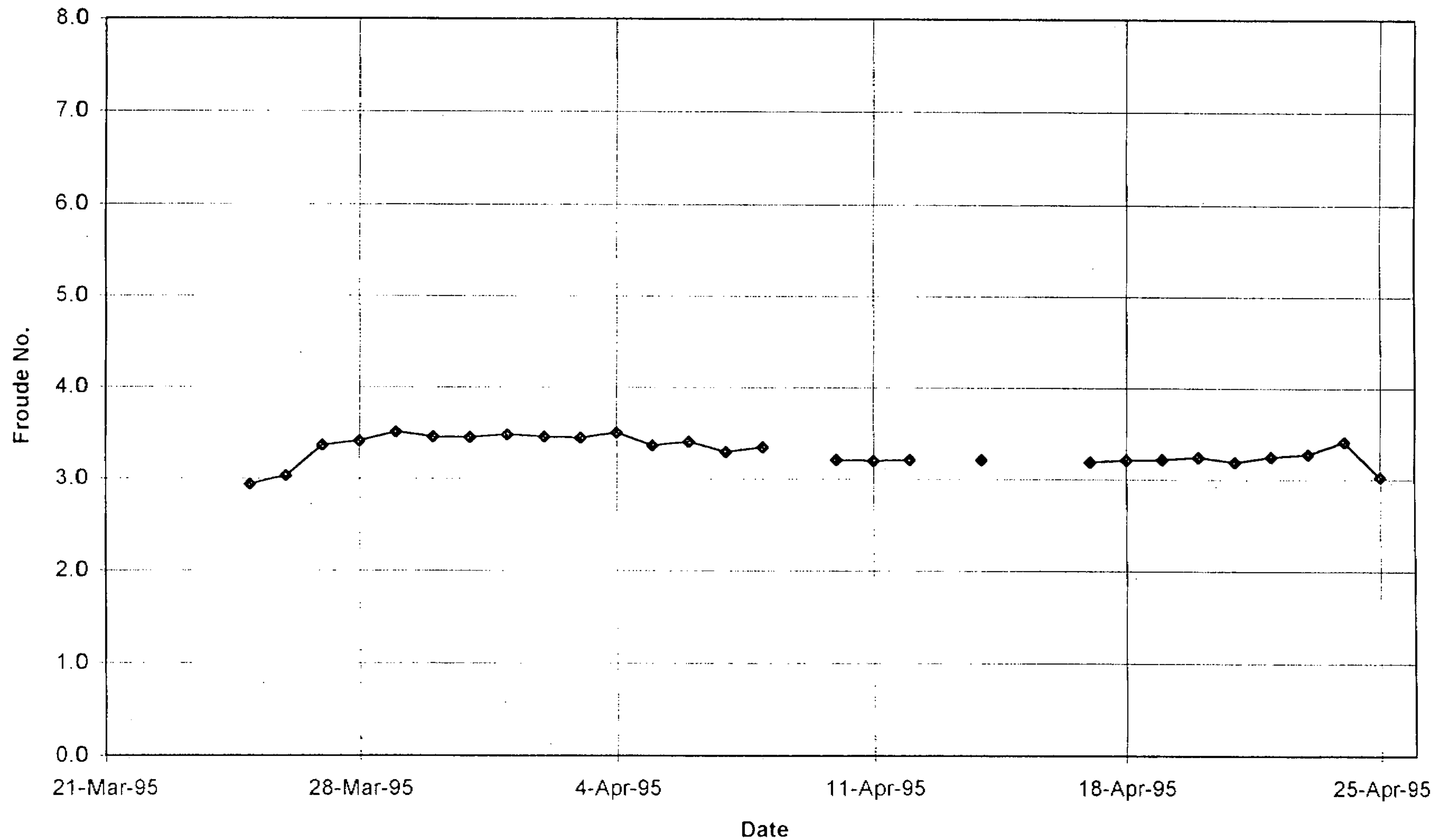


Figure 5.9 (g)

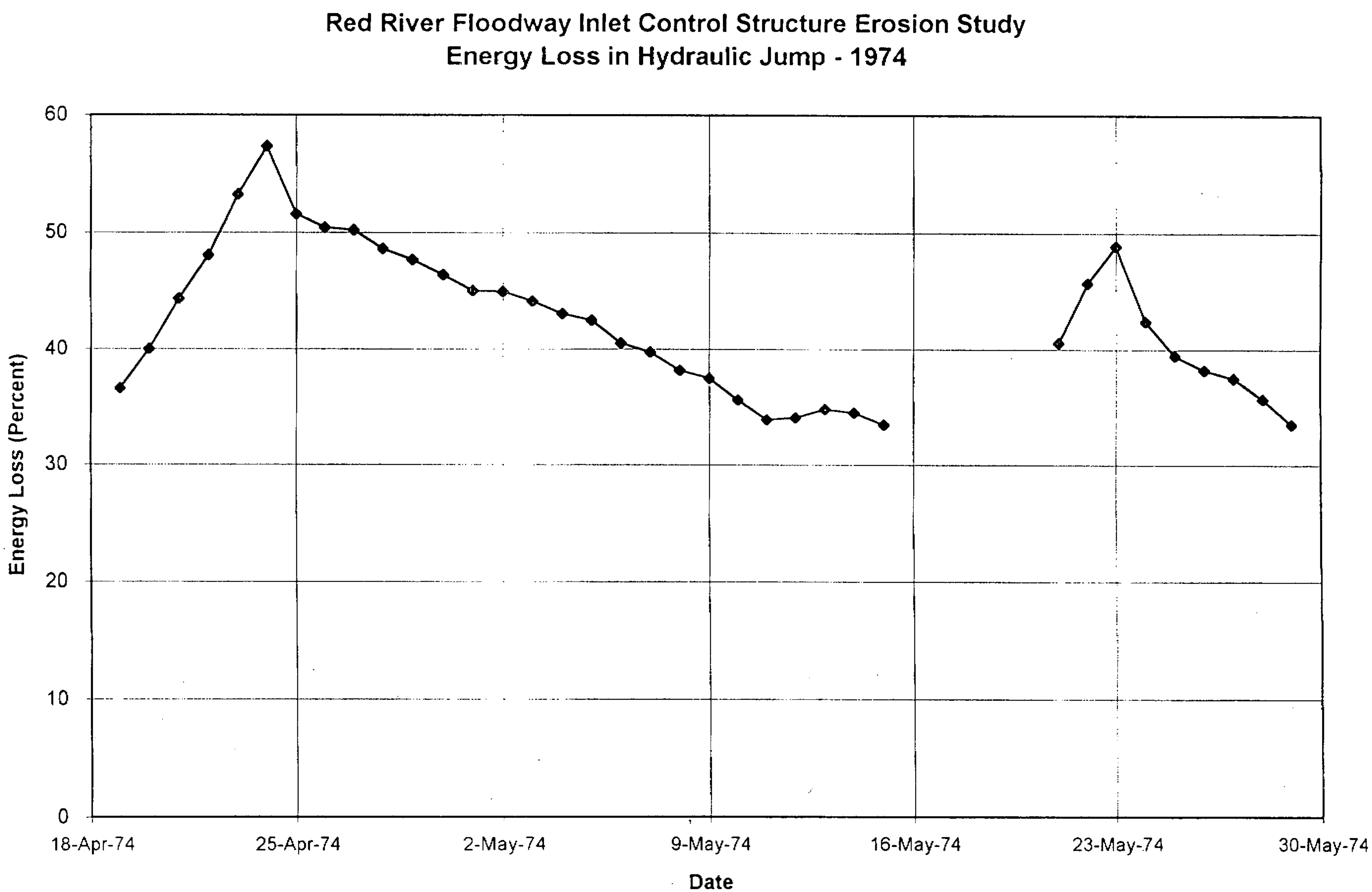

Red River Floodway Inlet Control Structure Erosion Study
Energy Loss in Hydraulic Jump - 1974
Energy Loss (Percent)
0
10
20
30
40
50
60
18-Apr-74
25-Apr-74
2-May-74
9-May-74
16-May-74
23-May-74
30-May-74
Date

Figure 5.10 (a)

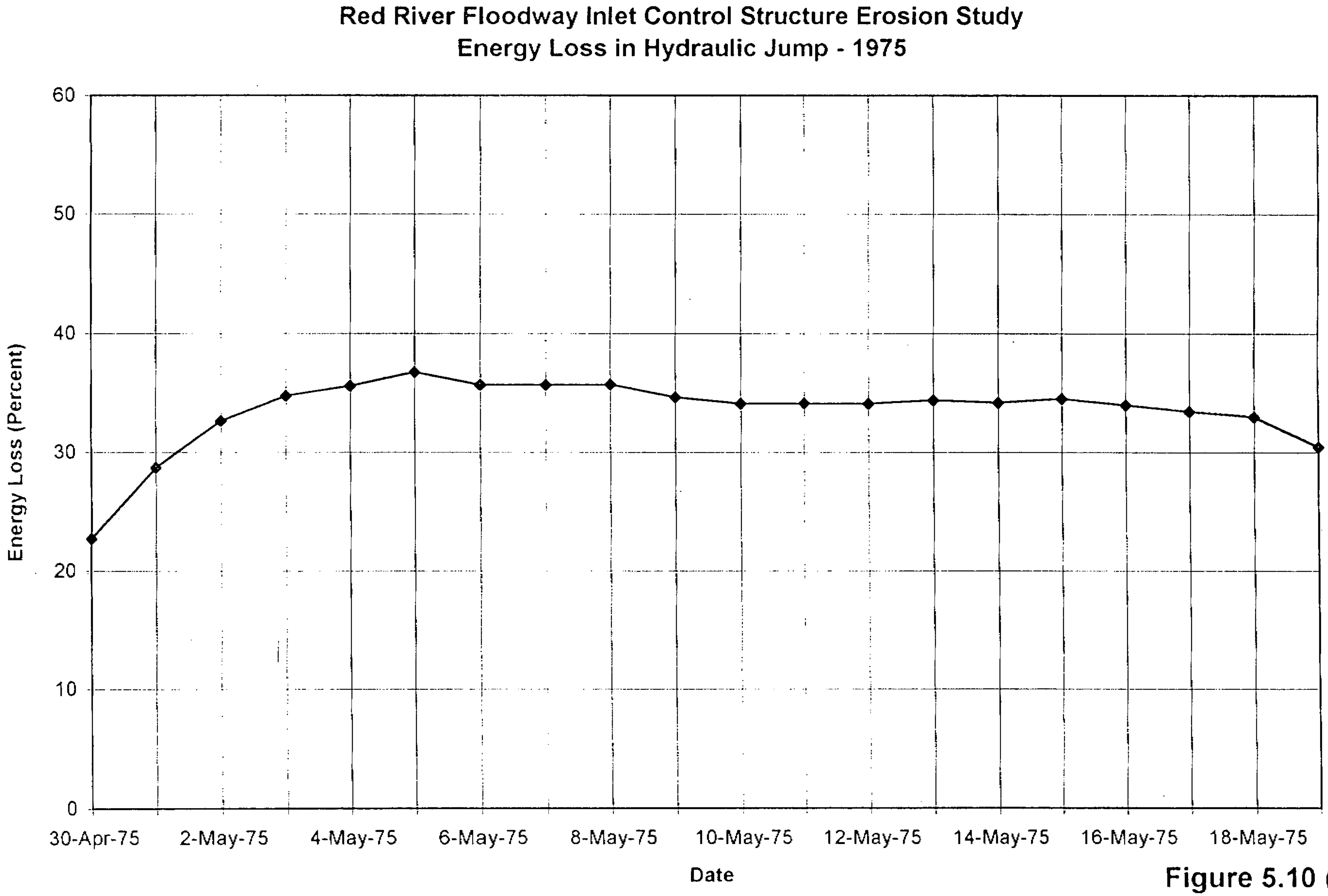
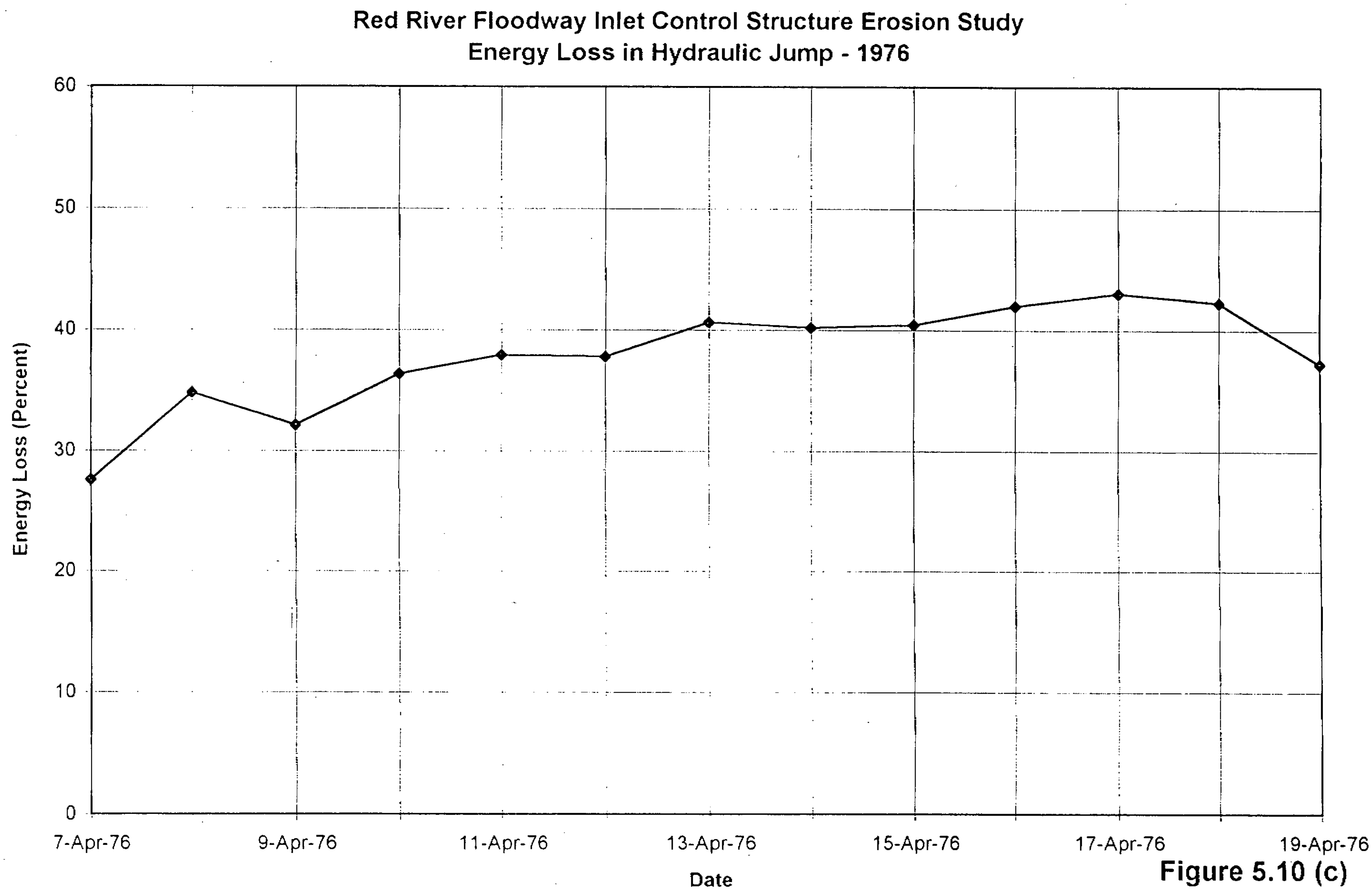

Red River Floodway Inlet Control Structure Erosion Study
Energy Loss in Hydraulic Jump - 1975
Energy Loss (Percent)
60
50
40
30
20
10
0
30-Apr-75
2-May-75
4-May-75
6-May-75
8-May-75
10-May-75
12-May-75
14-May-75
16-May-75
18-May-75
Date

Figure 5.10 (b)



Red River Floodway Inlet Control Structure Erosion Study
Energy Loss in Hydraulic Jump - 1976
Energy Loss (Percent)
60
50
40
30
20
10
0
7-Apr-76
9-Apr-76
11-Apr-76
13-Apr-76
15-Apr-76
17-Apr-76
19-Apr-76
Date

Figure 5.10 (c)

Red River Floodway Inlet Control Structure Erosion Study
Energy Loss in Hydraulic Jump - 1978

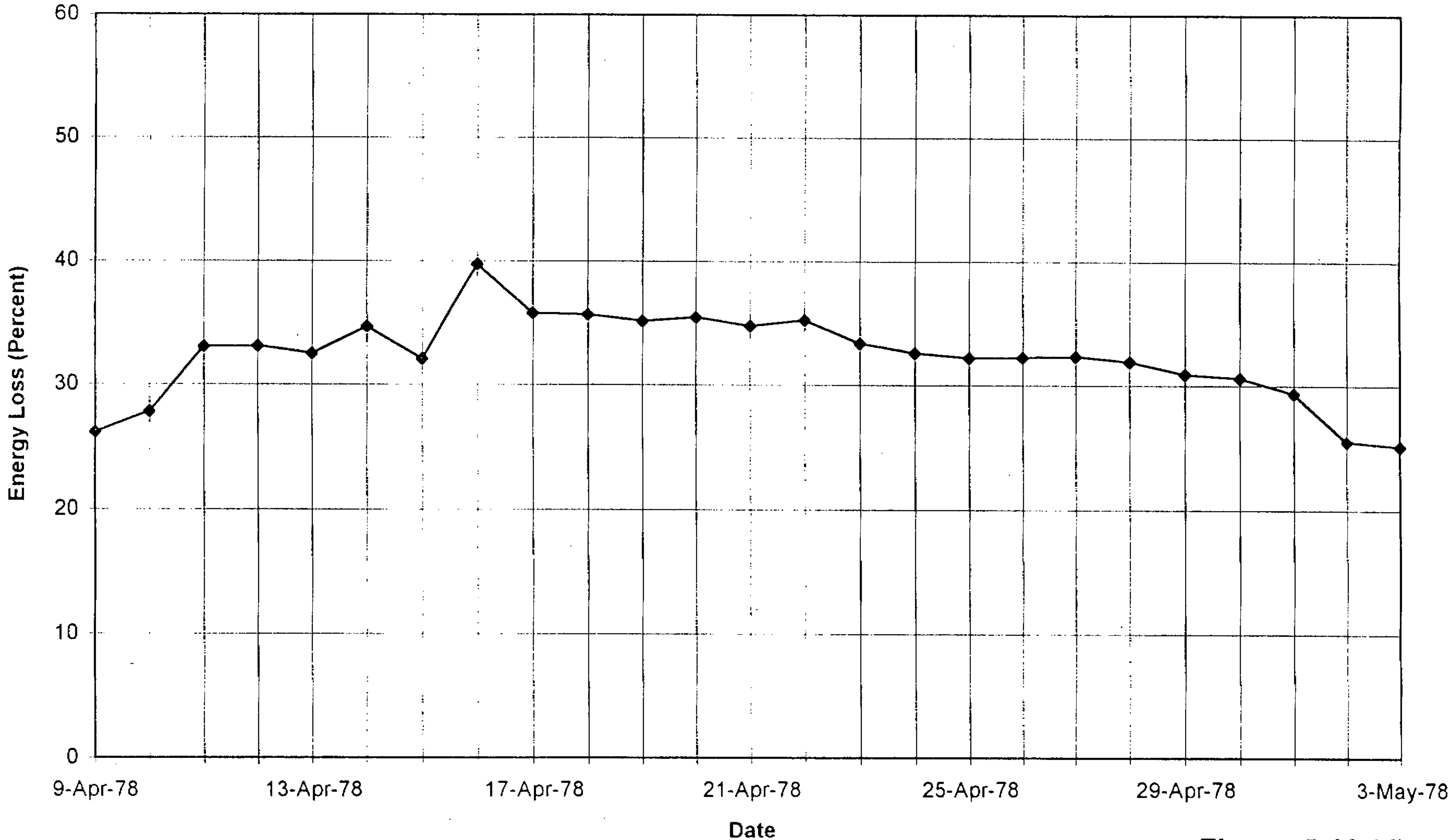


Figure 5.10 (d)

Red River Floodway Inlet Control Structure Erosion Study
Energy Loss in Hydraulic Jump - 1979

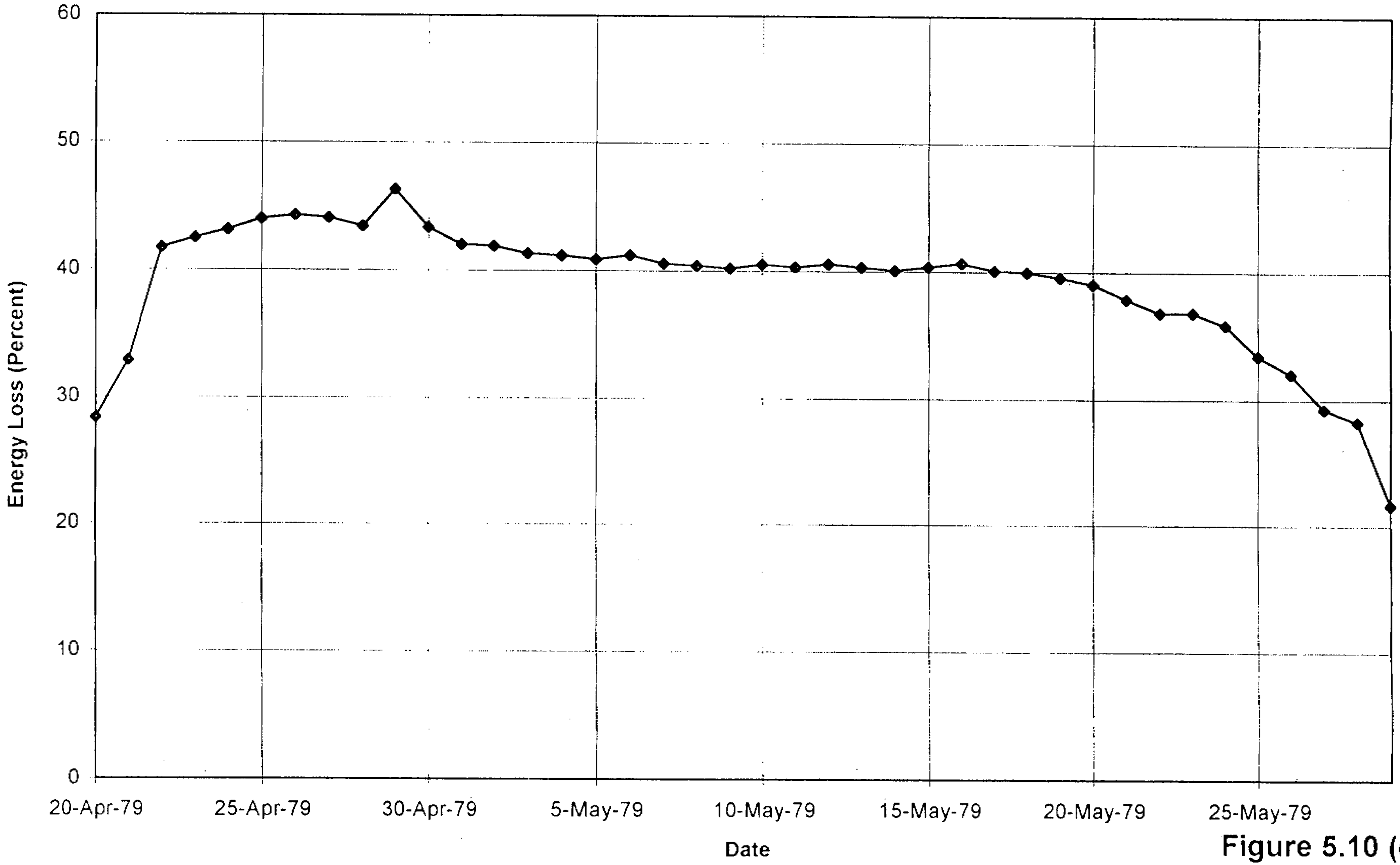


Figure 5.10 (e)

Red River Floodway Inlet Control Structure Erosion Study
Energy Loss in Hydraulic Jump - 1986

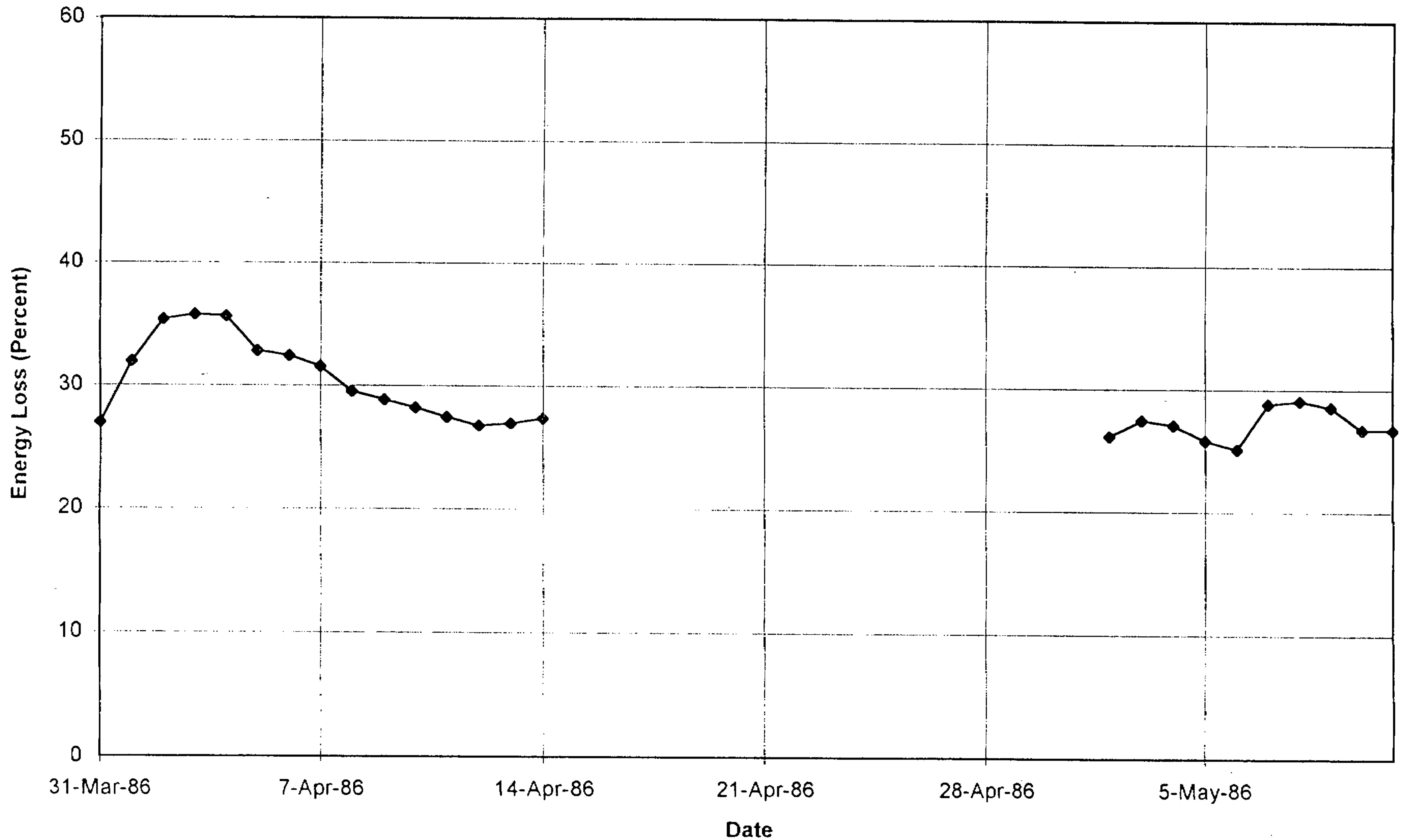
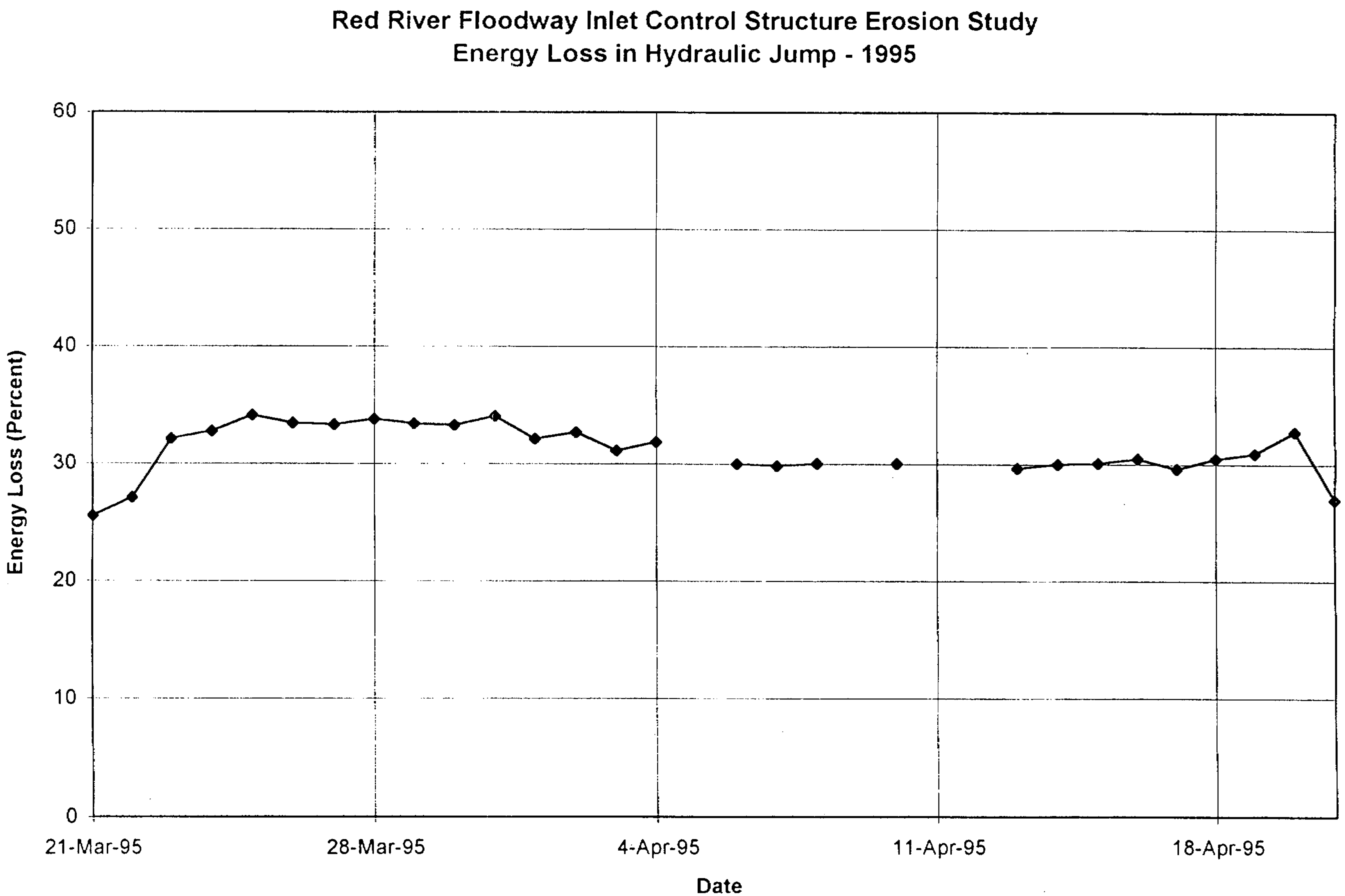


Figure 5.10(f)



Red River Floodway Inlet Control Structure Erosion Study
Energy Loss in Hydraulic Jump - 1995
60
50
40
30
20
10
0
Energy Loss (Percent)
21-Mar-95
28-Mar-95
4-Apr-95
11-Apr-95
18-Apr-95
Date

Figure 5.10 (g)

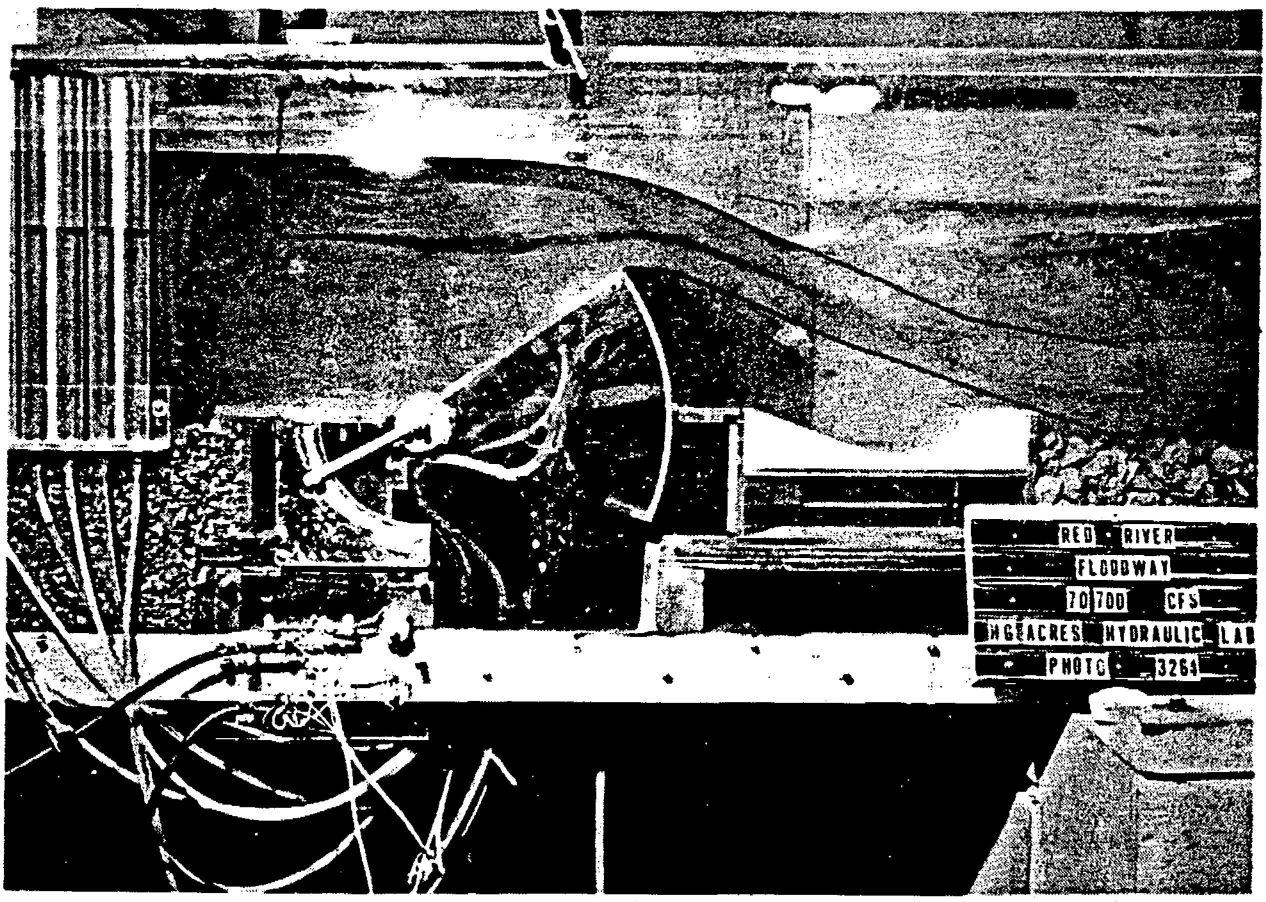


FIGURE 5.11 1:60 scale hydraulic model of control structure gate and stilling basin. Note that the jet plunges past the end of the stilling basin

RED RIVER FLOODWAY INLET CONTROL STRUCTURE EROSION STUDY

APPENDIX A

1995 UNDERWATER EROSION SURVEY

FLOODWAY GATES

EROSION SURVEY

RED RIVER FLOODWAY

JUNE 28, 1995

DOMINION DIVERS LTD.
19 ARCHIBALD ST.
WINNIPEG, MAN. R2J 0V7

On June 26, and June 27 1995 Dominion Divers Ltd. conducted an inspection down stream of the inlet gates located at the south end of the City of Winnipeg. The inspection was conducted under the direction of Mr. Brian Bodnaruk with the KGS Group.

The purpose of the inspection was to ascertain the type of bottom and elevation of same in an area across the face of the floodway structure and downstream for a distance of about 90 meters. A grid map 10 meters by 10 meters was provided as a pattern to follow for the inspection. The grid was set up using lines perpendicular to the structure and 10 meters apart beginning with line 1 at the west wing wall of the structure and proceeding east to line 4. The interval after line 4 was changed to 15.84 meters to reflect the center of the structure where there is no water flow. Line 5 through line 8 were again spaced at 10 meter intervals with line 8 being at the east wing wall of the structure. The sections running parallel to the face were also spaced at 10 meter intervals and lettered A to H. A survey crew with KGS staked the shorelines on both the East and West sides beginning 10 meters down stream of the structure with line A and finishing 90 meters downstream with line H.

Dominion Divers Ltd. made use of a 4 person crew, a 18 foot boat, a D 50 total station instrument, Yamaha Quad, X 15 sonar chart recorder, rope and approved surface supplied diving gear complete with a pneumofathometer guage for the inspection.

The first stage of the inspection was to ascertain the elevation of the bottom at the 64 intersecting points and determine the size of material on the bottom at the same points. To accomplish this a rope was tied to the rail on the deck of the structure at stations 1 through 8. The rope was paid out to the boat with the divers and equipment on board. The rope was used to keep the dive station relatively in line with the intended grid, while the instrument was used in conjunction with the motor for fine adjustments in position. The position parallel to the structure (sections A - H) was determined with the use of one man lining the bow of the boat thru the survey stakes. When both perpendicular and parallel lines crossed an anchor was lowered as a down line for the diver to follow to the exact point on the bottom where the survey was to be conducted. Lines 5 were done on June 26 with a water elevation of 734.75 . Lines 4,3,2 and 1 were completed on June 27 with a water elevation of 734.63. The water level is used as a benchmark to compute the bottom elevation by subtracting the depth of water as determined by the pneumofathometer worn by the diver from this figure. The accuracy of the divers pneumofathometer was ascertained through the use of a chain at the beginning and end of each day. The bottom elevations are recorded on the drawing # 1. The elevations recorded from the divers pneumo guage are those that are located at the intersecting points of the stations. The figures in the intermediate areas were attained through the use of the X 15 Sonar graph recorder on the afternoon of June 27. 1995. The chart recordings were aligned using the instrument to keep the boat on course and a man lining the transducer through the stakes on the shore as the stations were passed from H to A. While the diver was on the bottom completing the initial survey he observed the size and consistency of the material on the bottom at all locations. These observations are recorded on drawing # 2. The rock size was determined at each point with the average size being noted along with the largest size in the location. The second part of the survey required the placing of a 6 foot x 6 foot box on the bottom in pre determined areas and the number and size of the rocks recorded. These results are recorded on drawing # 3 Special Concerns.

The problem the divers had in many places was that the anchor or down line would enter an area deeper than many of the surrounding rocks and this gives a distorted view of the number of rocks in a particular area. It also tends to fool the sonar recorder by showing a bottom higher than that found with the pneumofathometer the diver used. This appears to be a problem in one or two areas on the Drawing #1.

East Wall | 10 meters | Structure Center | 15.84 meters | West Wall

80 meters

	8	7	6	5	4	3	2	1
A	715.28	715.28	-716.22	721.22	723.13	722.13	724.13	721.13
	720.78	718.28	----	726.28	726.28	726.78	725.28	725.28
B	724.28	716.28	717.22	726.22	725.13	724.63	721.13	724.13
	724.78	717.78	----	725.78	725.78	723.78	720.78	724.78
C	725.28	717.28	713.22	725.22	724.13	720.13	717.13	723.13
	724.78	717.78	720.78	723.78	724.78	720.78	716.78	724.78
D	723.28	717.28	717.22	721.22	723.13	719.13	716.63	724.13
	722.78	718.28	719.28	722.28	721.78	720.78	718.28	724.63
E	723.28	717.28	715.22	720.22	721.13	719.13	718.13	724.13
	724.78	719.78	717.78	722.78	723.78	719.78	720.78	725.63
F	726.78	722.28	719.22	724.22	724.13	723.13	722.13	727.13
	726.03	725.78	722.28	724.78	723.28	724.78	723.76	726.13
G	726.28	726.28	723.22	724.22	723.13	725.13	725.13	725.13
	725.78	726.78	725.78	724.22	723.78	725.78	725.28	725.63
H	725.28	725.28	722.22	724.72	723.13	723.13	725.63	726.13

FLOODWAY GATES
EROSION SURVEY
RED RIVER FLOODWAY

DOMINION DIVERS LTD
19 ARCHIBALD ST.
WINNIPEG, MAN. R2J0V

DRAWING #1
BOTTOM ELEVATIONS

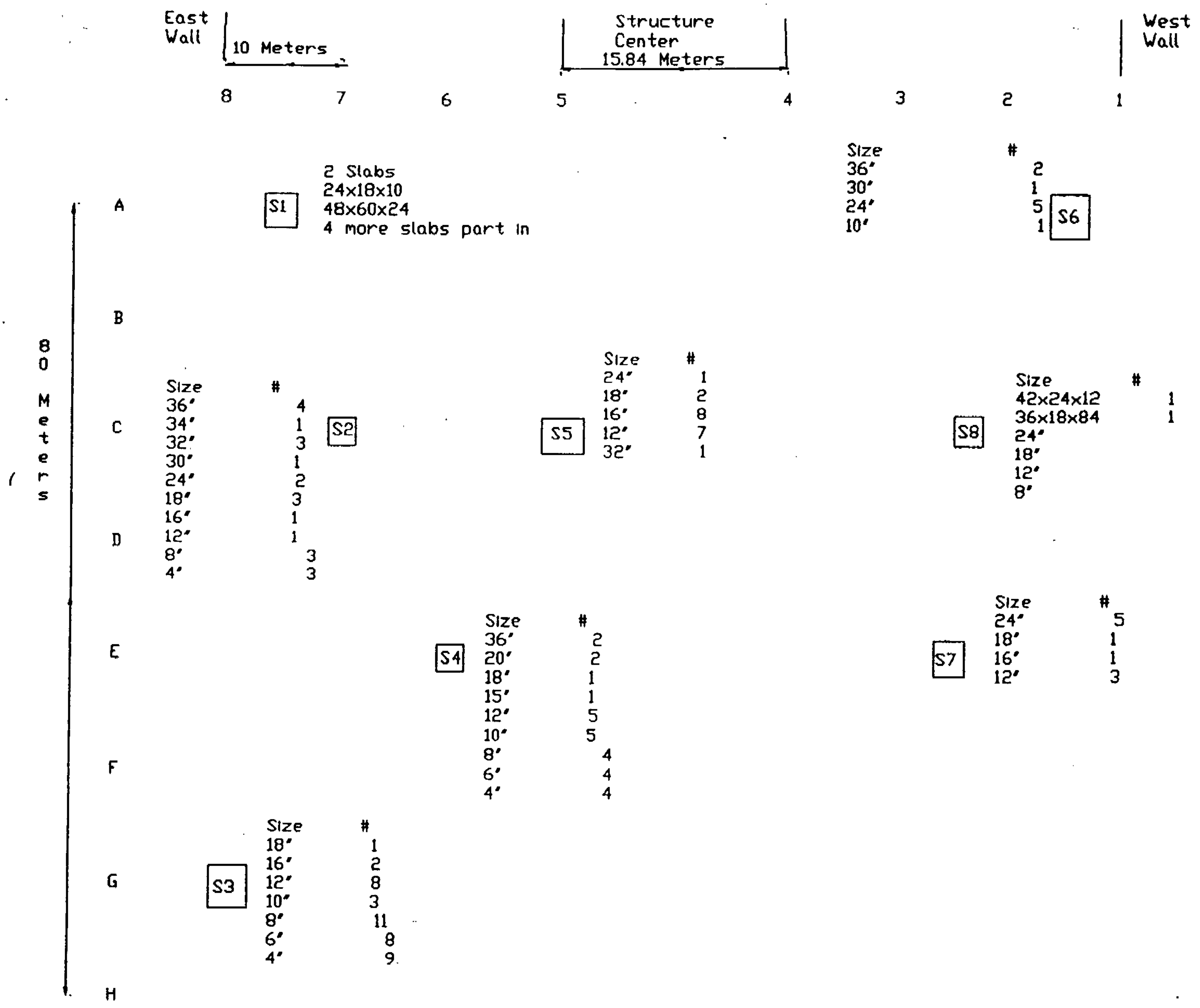
	East Wall	10 Meters		Structure Center 15.84 Meters				We Wa
	8	7	6	5	4	3	2	1
A	715.28 15-18"rock 36" largest	715.28 30"-36" rock up to 60"	716.22 48" rocks largest 96' scour hole	721.22 24"-36"rocks 60" largest	723.13 36"-42" rocks	722.13 30"-48" rocks 60" largest	724.13 slabs of rock 72"x36"x24" 72"x72"x24" 60x72x24" 60"x48"x24"	721.13 slabs of roc 84"x84"x24" 48"x36"x24" 12"x12"x18"
B	724.28 12"-15" rock One 40" block	716.28 slabs of rock 72"x60"x36" 48"x60"x24" 48"x48"x36"	717.22 slabs of rock 36"x24"x12" 24"x12"x18"	725.22 12" rocks avg. 32" largest	725.13 24"-36" rocks	724.63 32"-48" roxks 60" largest	721.13 Slabs of rock 72"x60"x24" 36"x24"x28" 36"x48"x24" 72"x60"x24"	724.13 slabs of roc 48"x36"x24" 48"x8"x24" 36"x42"x24"
C	725.28 8"-12" rock 18" largest	717.28 24"-36" rocks	713.22 12"-24" rocks 36" largest	725.22 12"-16" rocks 24" largest	724.13 18"-24" rocks 36" largest	720.13 18"-24" rocks 36" largest	717.13 18"-24"rocks 42" Largest	723.13 12"-18" rock 30" largest
D	723.28 gravel bottom 8"-15" rock	717.28 24"-36" rocks	717.22 12"-16"rocks largest 24"	721.22 12" rocks avg./ 24" largest	723.13 18" rocks 36" largest	719.13 8"-12" rocks 1 32" rock	716.63 12"-18" rocks 36" largest	724.13 8"-12" rocks 24" largest
E	723.28 8"-12" rocks gravel bottom	717.28 24"-36" rocks	715.22 18" rocks avers 12-36" rocks	720.22 12" rocks 18" largest	721.13 8"-12" rocks 18" largest	719.13 12" rocks	718.13 16"-24" rocks	724.13 4"-12" rocks
F	726.78 12"-32" rock	722.28 12"-24" rock	719.22 4"-24" rocks 12" avg.	724.22 8"-10"rocks 18" largest	724.13 6"-12" rocks sand gravel	723.13 12"-18" rocks 24" largest	722.13 10"-16" rocks 36" largest	727.13 4"-12" rocks
G	726.28 sand botton 8"-12" rocks	726.28 10"-14" rock 1 36" rock	723.22 8"-16" rocks 18" largest	724.22 8" 16" rocks 24" largest	723.13 8"-12" rocks 16" largest	725.13 12" rocks	725.13 12"-15" rocks 32" largest	725.13 4"-12" rocks
H	725.28 10"-14" rocks	725.28 8"-12" rocks	722.22 4"-10" rocks 12" largest	724.72 12" rocks 18" largest	723.13 8"-12" rocks	723.13 6"-12" rocks 18" largest	725.63 8"-12" rocks 16" largest	726.13 12"-24" rocks

80 Meters

FLOODWAY GATES
EROSION SURVEY
RED RIVER FLOODWAY

DOMINION DIVERS LTD.
19 ARCHIBALD ST.
WINNIPEG, MAN. R2J0

DRAWING #2
BOTTOM TYPE



FLOODWAY GATES
EROSION SURVEY
RED RIVER FLOODWAY

DOMINIOIN DIVERS LTD.
19 ARCHIBALD ST.
WINNIPEG, MAN. R2J 0V7

DRAWING # 3
SPECIAL CONCERNS

RED RIVER FLOOD INLET CONTROL STRUCTURE EROSION STUDY

APPENDIX B

FABRIC FORM TECHNICAL DATA

FABRIC FORMS FOR CONCRETE

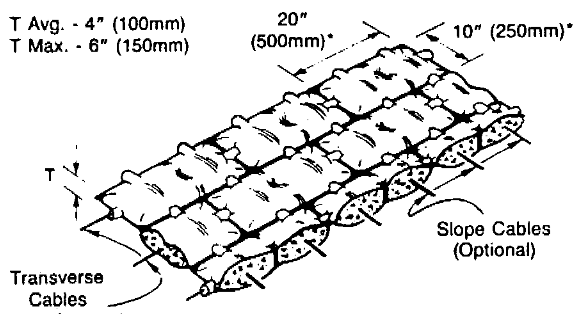
Erosion Control Revetments

fabriform®

The FABRIFORM® Process*** uses a double-layer fabric form especially woven for optimum strength, stability, adhesion and filtering characteristics. A highly fluid sand/cement slurry is pumped into this fabric envelope after it has been placed on the area to be protected. Revetments can be cast as easily under water as in-the-dry.

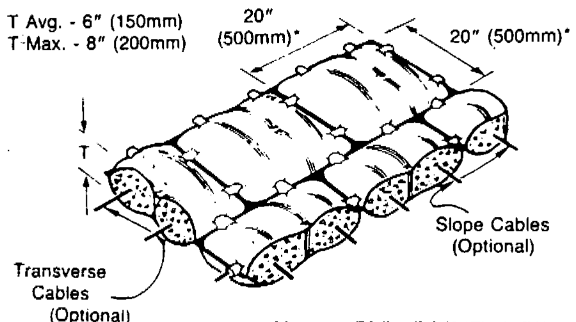
Fabriform® Articulating Block Fabric Styles

- designs based on experience -



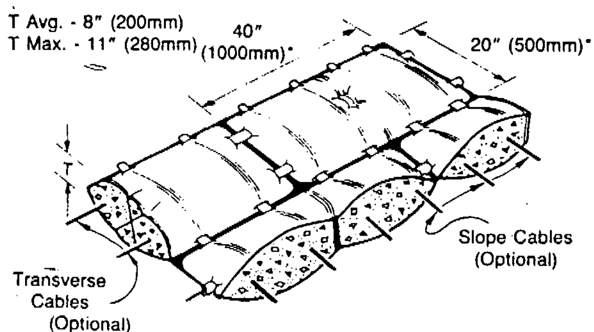
Mat. wt. - 47 lbs./ft.² (230kg/m²)**
Wt. of 10" x 20" block - 65 lbs. (30kg)
Mortar req. - 1 yd.³/80 ft.² (0.1 m³/m²)

4" AB



Mat. wt. - 70 lbs./ft.² (347kg/m²)**
Wt. of 20" x 20" block - 194 lbs. (88kg)
Mortar req. - 1 yd.³/60 ft.² (0.14 m³/m²)

6" AB



Mat. wt. - 93 lbs./ft.² (455kg/m²)**
Wt. of 20" x 40" block - 519 lbs. (236kg)
Mortar req. - 1 yd.³/40 ft.² (0.2 m³/m²)

8" AB

AB revetment fabric is a form for casting in place heavy-duty, rectangular concrete blocks in a staggered joint pattern. Revetments are reinforced by cables inserted between the two layers of fabric prior to slurry injection. Completely embedded in mortar, cables allow the cast-in place concrete blocks to flex (articulate) with changing soil and water conditions. Revetment fabrics are woven of 100% nylon of which at least 50% by weight consists of textured fibers for optimum filtering characteristics and adhesion to mortar.

Block thickness is controlled by spacer cords in the middle of each block. Lateral flow of mortar is controlled by shop-installed diaphragms (grout stops) located at spaced intervals.

Panels of fabric are shop-assembled in predetermined sizes convenient to handle and are joined together side-by-side at the job site by means of zipper closures attached to the upper and lower layers of fabric. Nylon rope has been used instead of galvanized cables which, as an option, are installed perpendicular to block width only and are referred to as "slope cables." Transverse cables, parallel to block width, may be also inserted if required.

Cable sizes commensurate with block weight are suggested in the Guide Specifications on the back of this sheet. Final selection of cable for each job is at the option of the Engineer.

For budgetary estimates, contraction allowance should be made for approximately 23% additional fabric area.

Note: Information contained in this publication is offered in good faith as a guide to placement of Fabriform® erosion control revetments. It is based on experience obtained under a variety of conditions. However, information contained herein will not apply to every job and dimensions, as a result of site conditions and installation procedures. The user is cautioned to obtain from others such professional and technical services as may, in his own judgement, be necessary or desirable to insure effective and economical installations.

* Dimensions shown are net cast-in-place block sizes, without articulating hinges. Articulating hinges are approximately 1 inch (25mm) wide and are designed to provide relief of hydrostatic pressures.

** Dry weight based on a specific weight of 2.2 or 140 lb./cf. Unit weight may vary with material proportions and source.

*** U.S. Patent Nos. 3,425,227 and 3,396,545.
Other U.S. and foreign patents issued.

CONTECH
an innovative process of
CONSTRUCTION TECHNIQUES, INC.
P.O. Box 360007
Cleveland, Ohio 44136 U.S.A.
Telephone: (216) 572-8300 · Fax: (216) 572-5533

Guide Specifications
For Fabriform® Articulating Block Revetment and Fabric Installation

I. GENERAL

The surfaces to be protected shall be prepared and graded to such an extent that they are normally stable in the absence of erosive forces. A fabric envelope in a mat configuration shall be positioned over these surfaces and filled with a pumpable sand/cement slurry in such a way as to form a stable mat of suitable weight and configuration.

The Contractor shall furnish records of past successful experience in performing this type of work. The Contractor shall save the Owner harmless from liability of any kind arising from the use of any patented or unpatented invention in the performance of this work.

II. MATERIALS

A. Fabric Design

Fabric-forming material shall consist of double-layer, open-selvage fabric joined in a mat configuration. Fabric shall be woven of 100% continuous multifilament nylon fiber of which at least 50% by weight shall be bulk textured fiber. Staple yarn shall not be allowed.

The tensile strength of spacer cords used to control block thickness shall total not less than 600 lbs. (2.7kN) at each section of control.

Fabric, designated as _______________ on the drawings, shall be woven in such a manner as to provide articulation joints, surrounding fine aggregate concrete-filled blocks measuring approximately __________" x __________" x __________". (See Note 1 below.) Block thickness shall be measured as described in Section III. C. of this specification.

Note 1: Designer shall indicate here the fabric designation required from choice of fabric styles listed below. Fabric style designates approximate nominal block thickness & size:

4" AB - 4" x 10" x 20" (100mm x 250mm x 500mm)
6" AB - 6" x 20" x 20" (150mm x 500mm x 500mm)
8" AB - 8" x 20" x 40" (200mm x 500mm x 1,000mm)

The two layers of fabric shall be connected at the center of each block with spacer cords of such a length as to positively control thickness of the finished block and to produce a pronounced corrugation in the surface of the form, when filled, to serve as evidence of complete and uniform filling of the fabric block form.

Articulation joints between adjacent blocks shall be staggered in such a manner as to avoid formation of a continuous channel from top to bottom of the slope..

Forms for individual blocks shall be interconnected with conduits to allow for passage of fluid grout between all adjacent blocks and to provide a sheath for protection of cables or ropes, if used, between adjacent blocks. At least one conduit shall be provided along each edge of adjacent blocks. Cast-in-place distance between conduits are approximately 10" (250mm) in the slope direction and 12" (300mm) in the transverse direction. The flat width of each conduit as woven shall be not less than 3" (75mm) nor more than 5" (125mm).

B. Fiber and Fabric Specifications

Fiber and fabric materials shall meet the minimum requirements, as listed and reported by an independent testing agency, shown on the attached Specification sheet.

C. Fabric Porosity

Fabric porosity is essential for the successful execution of this work. At the direction of the Engineer, the Contractor shall demonstrate the suitability of fabric design by injecting the proposed grout into 5½" (140mm) diameter. The sleeves shall be constructed of a single layer of the same basic fabric material. Test cylinders, 12" (300mm) long, shall be cut from each specimen and tested in accordance with ASTM C-39.

D. Relief of Hydrostatic Uplift

Fabric designated as _______________ on the drawings shall be woven in such a manner as to provide interwoven bands of attachments between blocks. These bands shall control the length-and-width block dimensions and also act as filter strips to provide relief of hydrostatic uplift beneath the completed revetment.

E. Tensile Reinforcing Members (If Required)

Tensile reinforcing members (cables), where required, shall be threaded through cable conduits between adjacent blocks. (See Note 2 below.)

Note 2: Tensile members, when used, are normally threaded through every conduit parallel to the slope. Transverse tensile members may also be threaded through conduits perpendicular to the slope, at the option of the designer.

Slope cables shall consist of _______________ on approximately __________ in. (_______mm) centers cast-in-place.
Transverse cables shall consist of _______________ on approximately __________ in. (_______mm) centers cast-in-place. (See Note 3 below.)

Note 3: Designer normally specifies either 3/16" (5mm) galvanized steel aircraft cable, 7 x 19 construction, or 1/4" (6mm) nylon rope. 1/8" (3mm) steel aircraft strand, 1 x 19 construction, has also been specified. Other types of tensile reinforcing may be specified. Cable spacing must be a multiple of conduit spacing as called for in Section II. A.

Where necessary, tensile members shall be joined by means of copper connectors. Aluminum connectors in direct contact with cement grout will not be permitted. All tensile members and connectors shall be completely embedded in the hardened grout. Exposed tensile members between adjacent blocks will not be permitted.

F. Fabric Assembly

Adjacent fabric panels shall be connected by sewing or by means of zippers. The two top layers of fabric and the two bottom layers of fabric shall be separately joined so as to ensure full block thickness between the two parallel seams. A single seam in which all four layers of fabric are joined at one point will not be permitted. If required, grout stops may be installed parallel to and in between individual mill widths at predetermined intervals to regulate the flow of fluid fine aggregate concrete. Grout stops shall be so designed as to produce full block thickness along the full length of the grout stop.

G. Fine Aggregate Concrete

Fine aggregate concrete shall consist of a mixture of portland cement, fine aggregate and water so proportioned and mixed as to provide a readily pumpable slurry. Admixtures and/or a pozzolan may be used with the approval of the Engineer. Use of super plasticizers and/or silica fume require special precautions. The hardened fine aggregate concrete shall exhibit a compressive strength of 2,000 psi (14 MPa) at 28 days when specimens are made and tested according to the provisions of ASTM C-31 and C-39.

The average compressive strength of FABRIFORM cast test cylinders, as described in Paragraph C above, shall be at least 20% higher at 7 days than that of companion test cylinders made in accordance with ASTM C-31 and not less than 2,500 psi (17 MPa) at 28 days.

III. INSTALLATION

A. Fabric Storage

Immediately following receipt of fabric on the job site, fabric shall be inspected and stored in a clean dry area where it will not be subject to mechanical damage or exposure to moisture or direct sunlight.

B. Fabric Placement

Prior to fine aggregate concrete injection, the dual-walled fabric shall be positioned over a geotextile filter fabric, as specified by the Engineer at its approximate design location, making appropriate allowance for approximately 11% contraction of the fabric in each direction which will occur as a result of fine aggregate concrete injection. Cables shall be securely attached to the ground anchor system or beam at the crown of the slope to prevent slippage of the fabric as it is being filled with fine aggregate concrete. Cable length shall be approximately 10% less than fabric length and the ends of cables which protrude through the fabric shall be provided with clips and external washers so that the cable will be placed in tension when the fabric envelope is filled with mortar. Cables shall each be fastened to separate points of attachment so that the point of anchorage is in a direct line with the cable itself.

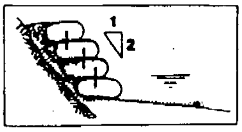
Panels of fabric shall be factory assembled in predetermined sizes and joined together side-by-side at the job site by means of zipper closures attached to the upper and lower layers of fabric. If joining of panels as described above, is impractical, adjacent panels may be overlapped a minimum of 760mm (30"), subject to Engineer's approval. In no case will simple butt joints between panels be allowed, unless underlayment of similar fabric is sewn and overlapped a minimum of 760 mm (30") of finished pumped mat.

C. Following placement of the dual-walled fabric over the geotextile filter, fine aggregate concrete shall be injected between the top and bottom layers of fabric through small slits cut in the upper layer of fabric. The injection pipe shall be wrapped tightly at the point of injection with a strip of burlap during pumping. After pumping, the burlap shall be pushed into the slit as the injection pipe is withdrawn in order to minimize spillage of fine aggregate concrete on the surface of the revetment. The sequence of fine aggregate concrete injection shall be such as to insure complete filling of the revetment-forming fabric to the thickness specified by the fabric manufacturer.

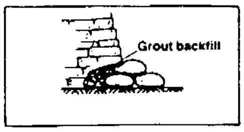
Foot traffic will not be permitted on the freshly pumped mat when such traffic will cause permanent indentations in the mat surface. Walk boards shall be used where necessary. Excessive fine aggregate concrete which has been inadvertently spilled on the mat surface shall be cleaned up with a broom and shovel. Use of a water hose to remove spilled fine aggregate concrete from the surface of a freshly pumped mat will not be permitted.

During fine aggregate concrete injection, the block thickness may be measured by inserting a short piece of stiff wire through the crowns of the blocks at several locations from the crest to the toe of the slope. Any block measuring less than 90% of the average of all thickness measurements shall be reinjected until acceptable thickness has been attained.

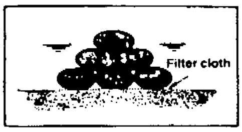
FABRIFORM® CONCRETE BAGS TECHNICAL DATA



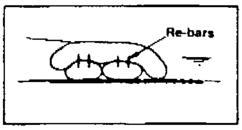
SHORELINE PROTECTION



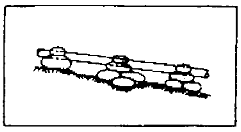
STRUCTURAL REPAIRS



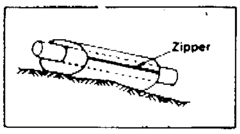
GROINS



BREAKWATERS



PIPELINE SADDLES



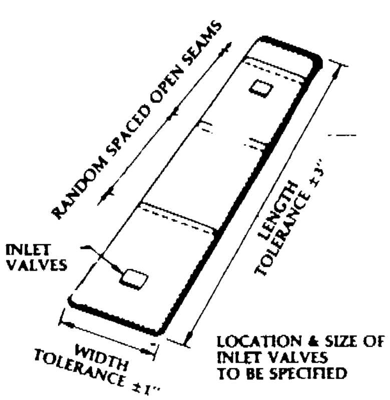
PIPELINE JACKETS

DESIGN

1. Molded concrete blocks cast in fabric bags are intended for use in construction of groins, breakwaters, retaining walls and the protection of lake and ocean shorelines subject to severe wave action in which they are an effective alternate for quarry stone. They may also be used as underwater coffer dams or for the repair of concrete, masonry or timber crib structures.
2. Used as waterfront protection over erodable soil, particular attention is directed to the importance of providing security against toe scour and undermining or consolidation of underlying soil. A layer of filter cloth beneath and behind the blocks may be required. An outboard skirt with a chain weighted hem may be useful. Fabriform Revetments have been used as a supporting platform under the blocks on soft soil.
3. These blocks can be doweled together by inserting steel reinforcing bars as suggested below.
4. In construction of breakwater and groins, the top layer of blocks should be oriented in as far as possible perpendicular to the predominant direction of wave action or "nose-on" to the waves.

INSTALLATION

1. Filter cloth underlayment or a supporting platform of Fabriform Revetment is first installed if required.
2. Concrete bags are positioned with steel stakes which hold the bags in place during filling and maintain alignment of the completed structure.
3. Filling of the bags is most easily accomplished with a sand/cement mortar usually containing about nine sacks of cement and 2300 lbs. of concrete sand per cu. yd. (12 sacks of cement and 1360 kg sand per m^3). Leaner mixes are satisfactory providing pumping characteristics are acceptable. If pea gravel is added to the mix, material cost is reduced, minimum hose size is increased, and labor cost may be increased.
4. To maintain alignment of stacked bags, the joints of which should be staggered, light gage 4" (100mm) angles may be temporarily secured to the lower layer of bags so that it acts as a sill to prevent the subsequent layer from rolling forward.
5. If reinforcing steel connectors between blocks are required, the bars are first inserted through the fabric and into the bag containing fresh concrete. The succeeding bag is threaded over these bags and filled with concrete.



ORDERING INFORMATION

1. All Fabriform concrete bags are assembled to order. Shipment can usually start within five days from receipt of order.
2. Seams are folded and double needle stitched.
3. Specify bag size by length and width, allowing for contraction of the bag as it fills with mortar. Manufacturing tolerance is plus or minus 3" (75mm) in length and width.
4. Specify location of inlet valves and outside diameter of injection hose to be used.
5. Bags may be shop assembled to irregular non-rectangular shapes on request. Bulk fabric is available for job site assembly of special bags.
6. In estimating freight costs, it is conservative to use:
   Weight - 14 lbs. per hundred square feet of bag area ($7kg/10m^2$).
   Cube - 40 cu. ft. per 6,000 square feet of bag area ($0.2m^3/100m^2$).

CONTECH
CONSTRUCTION TECHNIQUES, INC.
P.O. Box 360007
CLEVELAND, OHIO 44136

FABRIFORM® CONCRETE BAG GUIDE SPECIFICATIONS

SCOPE OF WORK

Work covered by these specifications consists of furnishing all labor, materials, and equipment for placing Fabriform Concrete Bags at locations described in plans. The contractor shall furnish records of past successful experience in performing this type of work and shall submit to the engineer for approval a description of the materials to be used and schedule of installation sequence.

MATERIALS

A. Concrete shall consist of a mixture of cement, aggregates, admixtures, and water so proportioned as to provide a pumpable mixture. Concrete shall attain a minimum strength as determined by the average of three consecutive tests of not less than 2,500 p.s.i. (17MPa) at 28 days.

B. Fabric bags shall be made of high-strength water-permeable fabric. Seams shall be folded and double stitched. Each bag shall be provided with a self-closing inlet valve to accommodate insertion of concrete pumping hose. Two valves shall be provided for bags more than 20′ (6m) long. Fabric filter cloth, if required, shall be water permeable fabric woven of 1,900 denier producer bulk textured nylon fill in a 1,260 denier multifilament warp with a thread count no less than 20 x 18 nor more than 25 x 25.

INSTALLATION

Fabriform Concrete Bags shall be cast on a foundation of rock, natural soil, or well-compacted fill. Bags shall be positioned and filled in such a way that they abut tightly. Joints between bags in successive tiers shall be staggered. Reinforcing steel dowels, if required, shall be inserted through the fabric into the fresh concrete within the bag. Succeeding bags shall be stretched tightly and slipped over protruding dowels. Bars shall be inserted through fabric by parting fibers rather than cutting or tearing fibers.
Backfill behind filled concrete bags shall be lean concrete or well-compacted granular soil. If soil backfill is used, heavy nylon filter cloth shall be placed between blocks and backfill to prevent loss of soil.

FABRIC DESCRIPTION

Warp	20 ends/in. (780 ends/m) 1260 Denier Continuous Multifilament Nylon	
Fill	18 picks/in. (702 picks/m) 1900 Textured Continuous Multifilament Nylon	
Approx. Wt.	8 oz./yd.^{2} (271 g/m^2)	(ASTM D-3776)

TYPICAL FABRIC TEST DATA (As reported by Independent Testing Agency)

- Thickness 36 mils (0.9mm) (ASTM D-1777)
- Grab Strength @ 12″ (300mm) per minute Strain Rate (ASTM D-1682)
  Warp: 475 lbs. (2.1kN) @ 19% Elongation
  Fill: 520 lbs. (2.3kN) @ 37% Elongation
- Mullen Burst Strength - 990 psi (6.8 MPa) (ASTM D-3786)
- Trapezoidal Tear Strength @ 12″ (300mm) per minute Strain Rate (ASTM D-4533)
  Warp: 240 lbs. (1.07kN)
  Fill: 240 lbs. (1.07kN)
- Puncture Strength: 169 lbs. (0.8kN) (ASTM D-3787 Modified)
- Constant Head Permeability:0.09cm/sec (ASTM D-4491)
  Permittivity: 0.98 cm/cm x sec
  Flow Rate: 72 gal/min/ft^2 (2.9m^3/min/m^2)
- Porosity: 105 CF/min (3m^3/min) (ASTM D-737)
- Equivalent Opening Size - U.S. Corps of Engineers CW-02215
  #40 U.S. Standard Sieve = 0.0167 in. (0.42mm)

RED RIVER FLOOD INLET CONTROL STRUCTURE EROSION STUDY

APPENDIX C

ENVIRONMENT LICENSE PROPOSAL APPLICATION FORM

Manitoba

Environment

Environmental Management

Bldg. 2, 139 Tuxedo Avenue
Winnipeg, Manitoba
R3N 0H6

Fax (204) 945-5229

FAXED
CLIENT FILE NO.: 4049.00

July 6, 1995

Mr. Rick Hay
Department of Natural Resources
Rm. 200, 1495 St. James St
PO Box 44
Winnipeg MB R3H 0W9

Dear Mr. Hay:

RE: RED RIVER FLOODWAY INLET STRUCTURE EROSION CONTROL

Thank you for your Proposal submitted pursuant to the Environment Act.

The contact person assigned to co-ordinate review and assessment of the Proposal is:

Bruce Webb
Telephone: 945-7021

Your Proposal will be advertised in the media (copy attached), placed in the Public Registries and circulated to government departments for review.

The above contact person may be in contact with you for further information as may be required.

Yours truly,

Pat Doa
for: L. Strachan, P. Eng.
Director
Environment Act

Attachment

c: D. Wotton, Regional Director
Centennial Public Library
Riel Community Office
David B. MacMillan, P. Eng., KGS Group

Manitoba

Notice of Environment Act Proposal

The Department of Environment has received a proposal pursuant to the Manitoba Environment Act from the following operation and invites public participation in the review process:

RED RIVER FLOODWAY INLET STRUCTURE EROSION CONTROL (FILE 4049.00)

A Proposal has been filed by the KGS Group on behalf of Manitoba Natural Resources for remedial erosion protection work on the downstream side of the Red River Floodway Inlet Control Structure. The structure is located on the Red River near Turnbull Drive adjacent to river lots 73, 74, 176 and 177 in the Parish of St. Norbert. The proposed work would consist of replacing riprap which has been removed by erosion since the construction of the structure. Replacement material would be placed on the bed of the river across its entire width to a distance approximately 250 metres downstream of the structure.

Anyone likely to be affected by the above operation and who wishes to make a representation either for or against the proposal should contact the Department, in writing, not later than AUGUST 14, 1995. Upon receipt of any notification, the Department may request a public hearing to be held and the concerned persons will be notified. Further information is available from the Public Registries in Bldg. 2, 139 Tuxedo Ave., Winnipeg; the Manitoba Eco-Network, 867 Westminster Ave., Winnipeg; and the Centennial Public Library, 251 Donald St., Winnipeg; or by contacting Bruce Webb at 945-7021.

Manitoba Environment

Environmental Approvals

Building 2
139 Tuxedo Avenue
Winnipeg, Manitoba R3N 0H6

Toll Free 1-800-282-8069
FAX (204) 945-5229

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Ad No. 0167EA

For Publication in the Winnipeg Free Press - Saturday, July 15, 1995.

KGS GROUP

KONTZAMANIS • GRAUMANN • SMITH • MACMILLAN INC.
CONSULTING ENGINEERS & PROJECT MANAGERS

June 26, 1995

File: 95-311-01

Manitoba Environment
Building 2
139 Tuxedo Avenue
Winnipeg, Manitoba
R3N 0H6

ATTENTION: Mr. L. Strachan
Director, Environmental Approvals

Re: Proposal for Environmental Approval for Red River Floodway Inlet Structure Erosion Control

Dear Mr. Strachan:

Please find attached an application for environmental approval for the project, Red River Floodway Inlet Control Structure - Scour of Downstream Rip Rap. The proposed work would consist of replacing the rip rap which has been scoured out from the river bed due to high flow velocities resulting from the operation of the inlet structure during spring floods. The extent of the proposed work will be the entire river width at the inlet control structure and for a distance from the downstream face of the control structure to a point approximately 250 metres downstream.

KGS Group is presently undertaking a study to determine the final form of the remedial work. Alternatives for restoration of the river bed include the dumping of rip rap from barges, the placement of the rip rap from finger groins constructed from each river bank, and the use of concrete erosion mats to line the existing scour hole. A further description of the work is included in the attached description of the proposed development.

The potential environmental impacts from either alternative are not considered to be significant. The project has been discussed with Mr. A. Derksen from Manitoba Fisheries and no specific concerns were identified. Further details on the project will be forwarded to your office when the design studies have been completed and construction details have been finalized.

If you have any comments or require clarifications on our application for an environmental license, please contact us.

Yours very truly,

David B. MacMillan, P.Eng.
Principal

Attachment

BMB/+

Manitoba Environment

Environment Act Proposal Form

This form prescribes the nature and sequence of the information required to file a proposal for a development pursuant to subsections 10(3), 11(7), and 12(3) of the Manitoba Environment Act.

Name of the development:	RED RIVER FLOODWAY INLET CONTROL STRUCTURE - SCOUR OF DOWNSTREAM RIPRAP
Legal name of the proponent of the development:	MANITOBA DEPT. OF NATURAL RESOURCES
Location of the development — Street address:	
Municipality:	RIVER LOTS 73,74,176& 177
City or Town:	WINNIPEG
Legal description:	PARISH OF ST.NORBERT ~~PLAN NO.10104 WLTO~~
Name of proponent contact person for purposes of the environmental assessment:	ROSS MADDER
Mailing address:	ROOM 200, BOX 44 / 495 ST. JAMES ST. WPG, MB
Postal code:	R3H 0W9
Telephone:	945-7462
FAX:	945-7782
Date:	JUNE 26, 1995
Signature of the proponent, or corporate principal of the corporate proponent:	
Printed name:	

OFFICE USE ONLY

Date Received ____________

Client File Number ____________

Development Review Class ____________

Region ____________

Departmental Contact Person ____________

Phone Number ____________

NOTE: The proponent should reproduce the underlined portions of each section as noted below, adding the required information following each section as it applies to the development. A response to all the sections is required.

DESCRIPTION OF THE DEVELOPMENT:

i) Certificate of Title showing the owner(s) and legal description of the land upon which the development will be constructed; or (in the case of highways, rail lines, electrical transmission lines, or pipelines) a map or maps at a scale no less than 1:50,000 showing the location of the proposed development:

ii) Name of the owner of mineral rights beneath the land, if not the same as as that of the surface owner:

iii) Description of the existing land use on the site and on land adjoining it, as well as changes that will be made thereto for the purposes of the development:

iv) Land use designation for the site and adjoining land as identified in a development plan adopted pursuant to the Planning Act or the City of Winnipeg Act, and the zoning designation as identified in a Zoning By-Law, if applicable:

v) A description of all previous studies and activities relating to feasibility, exploration, or project siting and prior authorization received from other government agencies:

vi) A description of the proposed development (including site plans), and the method of operation and hours of operation:

vii) An identification of any storage of gasoline or associated products:

viii) A description of the potential impacts of the development on the environment, including, but not necessarily limited to:
- type, quantity and concentration of pollutants to be released into the air, water or on land;
- impact on wildlife;
- impact on fisheries;
- impact on surface water and groundwater;
- forestry related impacts;
- impact on heritage resources;
- socio-economic implications resulting from the environmental impacts.

ix) A description of the proposed environmental management practices to be employed to prevent or mitigate adverse implications from the impacts identified in (viii) which will have regard to, where applicable: containment, handling, monitoring, storage, treatment, and final disposal of pollutants; conservation and protection of natural or heritage resources; environmental restoration and rehabilitation of the site upon decommissioning; and protection of environmental health.

SCHEDULE:

The proposed date of commencement of construction, commencement of operation, including staging of the development and termination of operation, if known.

FUNDING:

Name and address of any Government Agency (Federal, Provincial or otherwise) from which a grant or loan of capital funds have been requested, where applicable.

NOTE:

The Environment Act requires that subject to the Confidential Information clause, Section 47, a proposal shall be filed in the public registry.

Proprietary information provided in this form should be clearly noted. A separate summary of the proposal excluding the proprietary information should accompany the proposal for the public registry file.

20 copies of any bound report or blueprints supporting the Proposal are required.

The completed Proposal form should be sent together with a covering letter to:

Director, Environmental Approvals
Manitoba Environment
Building 2, 139 Tuxedo Avenue
Winnipeg, Manitoba
R3N 0H6

DESCRIPTION OF THE PROPOSED DEVELOPMENT

i. Location and Certificate of Title

The proposed development is located on the Red River at the Red River Floodway inlet control structure (Figure 1). All work related to the project will be confined to the Red River floodway right-of-way in River Lots 73, 74,176, and 177 in the Parish of St. Norbert, as shown on Plan No. 10104, Winnipeg Land Titles Office.

ii. Mineral Rights

The mineral rights beneath the work area are owned by the Crown. The proposed project will entail the replacement of a rip rap on the riverbed downstream of the Red River Floodway inlet control structure. Proposed work would be remedial in nature which would not alter the present ownership of subsurface mineral rights. Any potential impacts are considered negligible.

iii. Existing Land Use

The existing land use on the site is natural riverbed. The land use is therefore minimal. There will be no change to the existing land use as a result of the project.

iv. Land Use Designation

With the exception of the Red River floodway inlet right-of-way, the adjoining land is zoned agriculture.

v. Previous Studies

Three studies related to the erosion downstream from the inlet control structure were conducted previously, including:

- "Red River Floodway Inlet Control Structure Erosion Investigation", Inter-Departmental Memorandum, Department of Natural Resources, Water Resources Branch, September 17, 1975.
- "Red River Floodway Inlet Control Structure Erosion Investigation", Inter-Departmental Memorandum, Department of Natural Resources, Water Resources Branch, August 22, 1976.
- "Red River Floodway Inlet Control Structure Erosion Investigation", Inter-Departmental Memorandum, Department of Natural Resources, Water Resources Branch, August 22, 1979.

vi. Description of the Proposed Development

When the inlet control structure was constructed, the river bed was protected by a layer of rock rip rap, placed to a final design grade level of elevation 725 ft. The river bed, as surveyed in 1979, is shown on Figure 2. Surveys taken in 1994 show a similar pattern of erosion but with further depth of erosion of 1-2 feet. As indicated by the drawing, erosion of the riverbed has occurred to a depth of as much as 10 feet.

Several options for restoration and protection of the river bed to design levels are being considered, including the replacement of the rip rap with large rock (24 in. - 40 in.) which would be stable under future high flow velocities, and the lining of the riverbed with erosion control mats such as rock filled gabion structures. It may also be proven, from our investigations, that the scour hole downstream of the structure has reached a stable equilibrium shape, in which no further protection will be required.

Construction alternatives for the restoration of the erosion control would be by dumping of rock rip rap from barges or the construction of rock finger groins to provide access to the area of erosion for dumping of rock by dragline or backhoe. In either case, it may be advantageous to close one bay of the control structure and divert the entire flow thru the remaining open bay. This would provide an area of reduced flow velocities to facilitate placement of the rip rap. A staging area on each bank with access from the river bank to the existing dyke would also be provided. Access would be confined to floodway property.

From an environmental impact perspective, the methods of protection (rip rap or erosion control mats) and construction are considered to be similar. As such, the final selection of methods will not influence any conditions imposed upon the environmental license. Final drawings will be forwarded to the Department of Environment prior to construction to ensure conformance with the license requirements.

vii. Storage of Hazardous Materials

There will be no long term storage of gasoline or associated products on the site. A limited quantity of fuel may be stored for approximately 2 weeks during the construction period.

viii. Potential Impacts

- During construction there is potential for a limited increase in turbidity downstream due to disturbing of existing river bed sediments during the placement of the rip rap.

- The project will result in the placement of a rock fill on the existing river bed. The area of interest, however, consists of a similar rock fill. Since there will be no change in the form of the river bed, the impact on fisheries is minimal. The construction may require the closing of one bay in the inlet control structure and prevent movement of fish to the local area of construction. The remaining bay would remain open and movement of fish would not be restricted.

- The construction activity will result in increased level of noise and disturbance to the general area. These conditions would persist for only a brief period of time (approximately 2 weeks). The potential for significant impact is therefore considered minimal.

- There will be no long term impact on surface water or groundwater. Short term impacts on surface water will occur if temporary groins are constructed to facilitate access to the area of construction. Closure of one bay in the inlet structure, if undertaken, would result in a doubling of local flow velocities through the structure. Table 1, below lists the velocities through the inlet structure for a range in discharge conditions during August and September.

TABLE 1
Flow Velocity Thru Inlet Structure

	August		September	
	Median	Upper Decile	Median	Upper Decile
Discharge (cms)	50	180	45	130
Velocity (m/s)				
1 Bay	0.8	2.5	0.7	1.9
2 Bays	0.4	1.2	0.4	0.9

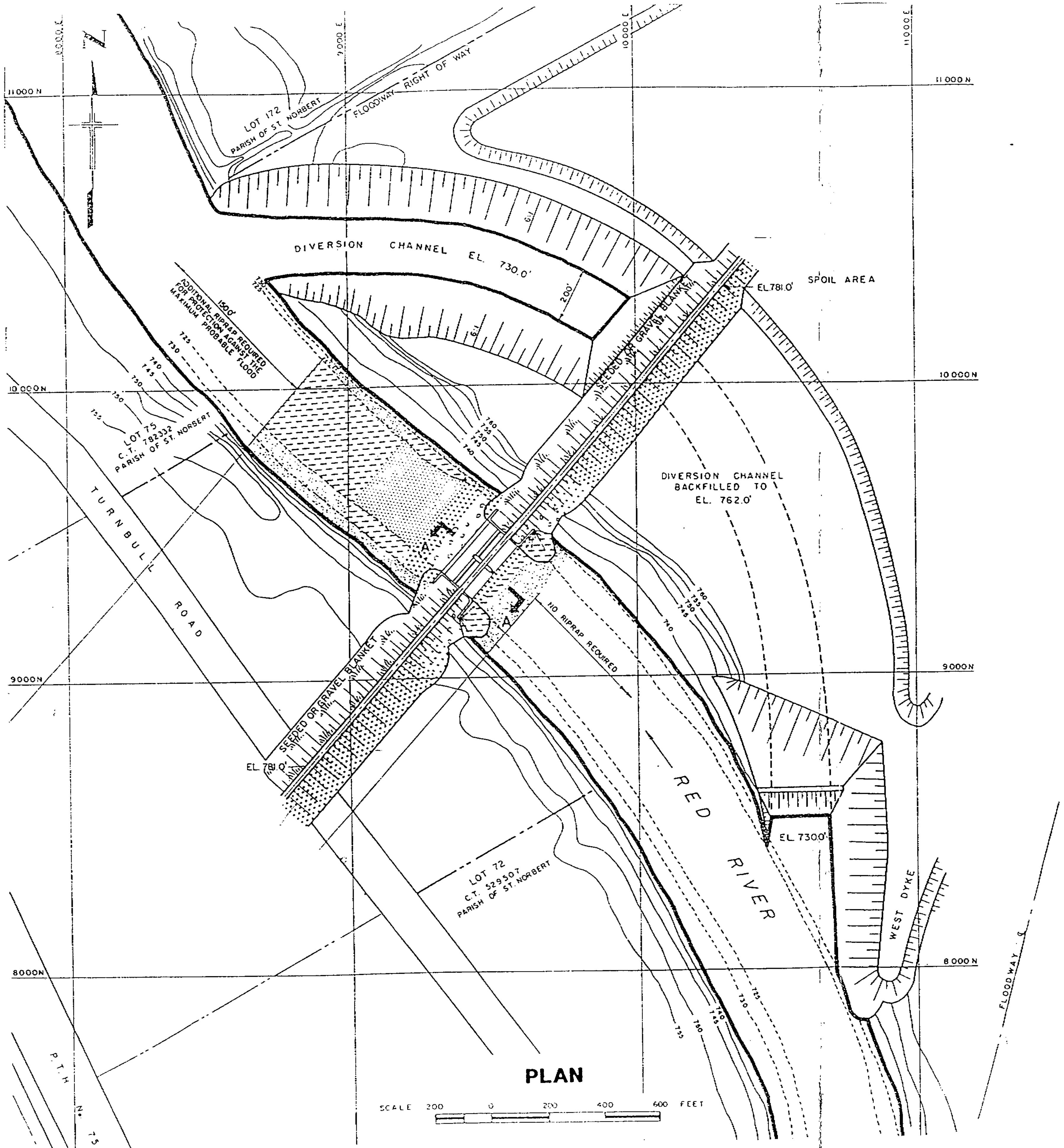
- The project was discussed with Mr. A. Derksen, Manitoba Department of Natural Resources, Fisheries Branch, and no specific concerns were identified. Fall spawning of fish in the Red River is limited and, as such, no concerns were identified. He suggested that these opinions will be confirmed at the Proposal review stage when more information will be available.

ix. Environmental Management Practices

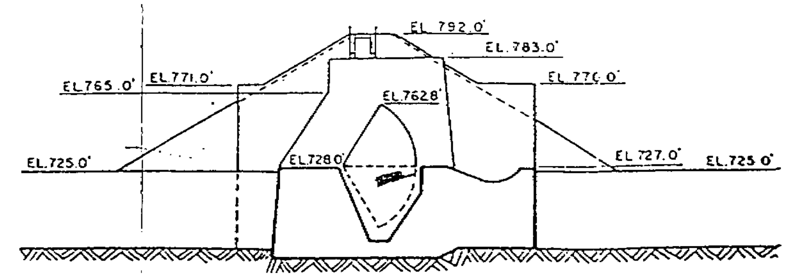
No significant environmental impacts are anticipated. Work will generally be conducted in accordance with practices outlined in the Manitoba Fisheries document, "Recommended Practices for Construction of Stream Crossings", thereby minimizing the potential for increased sediment during construction.

x. Schedule

The construction activity would require approximately two weeks to complete. Commencement is expected in August 1995 or September 1995.



PLAN

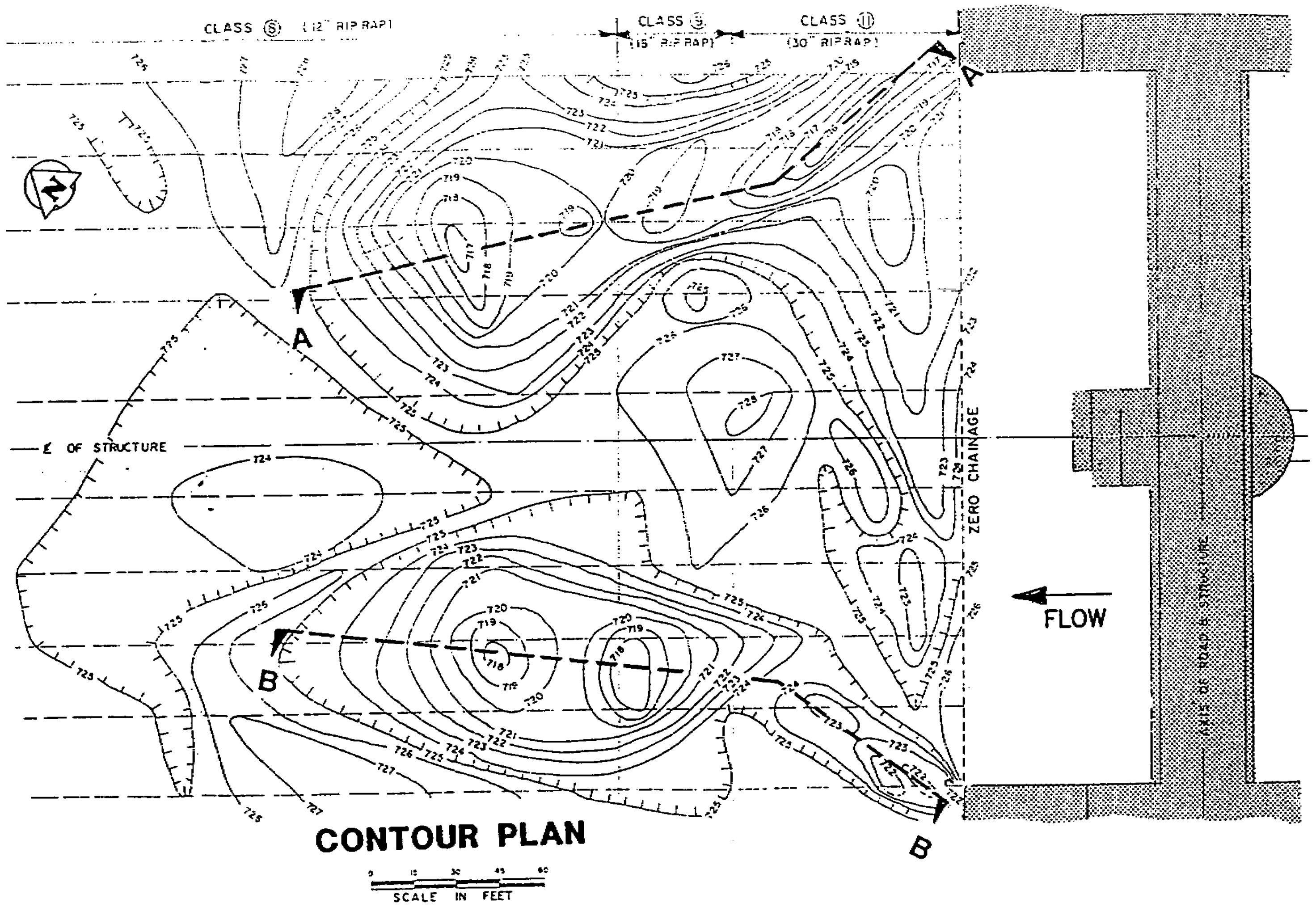


SECTION A-A

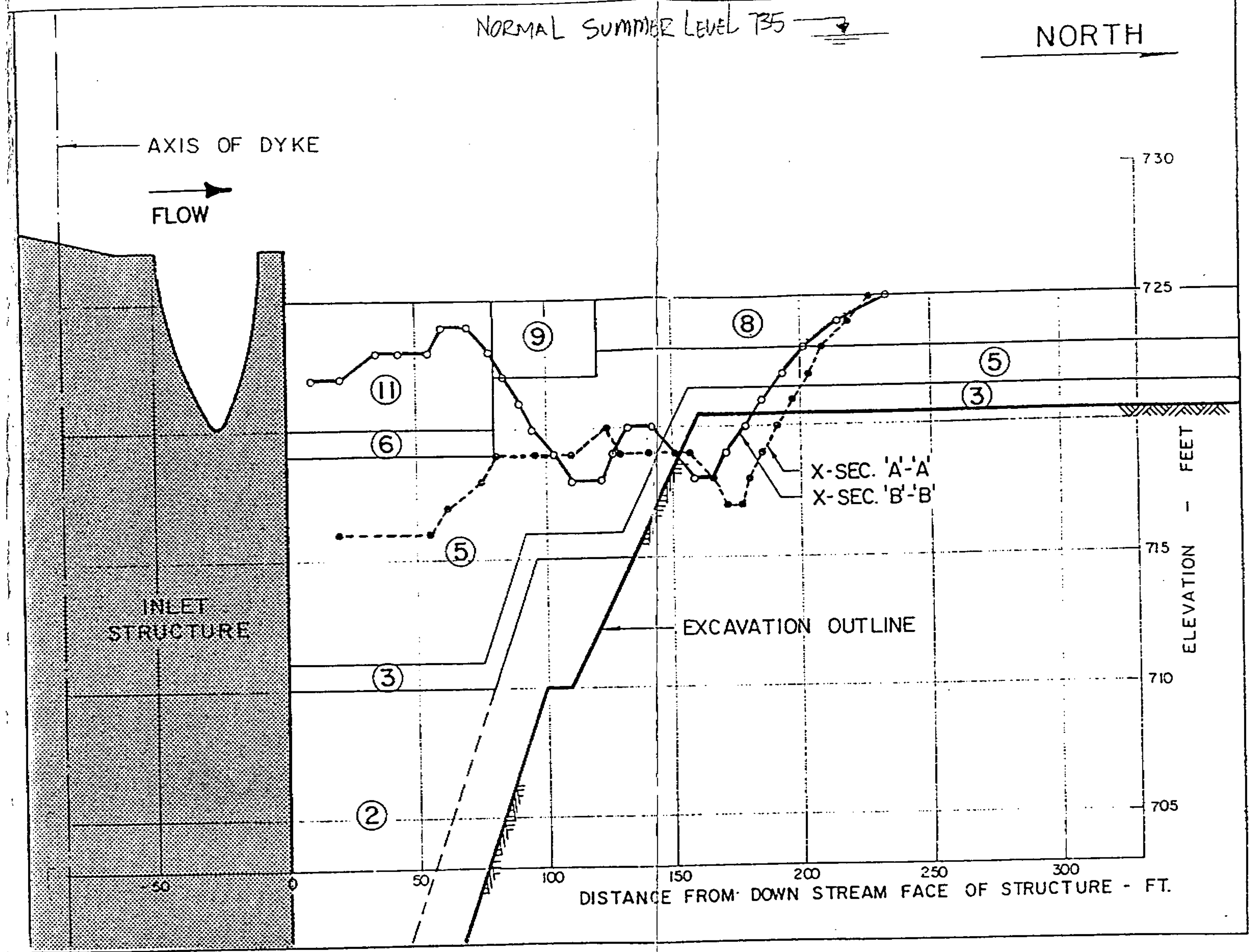
KGS GROUP	MANITOBA DEPARTMENT OF NATURAL RESOURCES
RED RIVER FLOODWAY EROSION STUDY	
GENERAL PLAN	
JUNE 1994	FIGURE 1

KGS FILE NO. 95-311-01

11" x 17"/ BASE SCALE: 1" = 1"



CONTOUR PLAN



SECTION AT CENTERLINE OF STRUCTURE

LEGEND :

CLASS OF MATERIAL	SIZE
⑪	30" RIPRAP
⑨	18" RIP RAP
⑧	12" RIP RAP
⑥	6" RIP RAP
⑤	6" FILTER STONE
③	11/2" FILTER STONE
②	TILL FILL

NOTES

1. Erosion depth as shown is based on soundings survey in June 25, 1979
2. Erosion protection to consist of rip rap or erosion mats placed downstream of structure to elevation 725 ft.

KGS GROUP

MANITOBA DEPARTMENT OF NATURAL RESOURCES

RED RIVER FLOODWAY EROSION STUDY

EROSION DOWNSTREAM OF INLET STRUCTURE

JUNE 1994

FIGURE 2

PHOTOGRAPHS

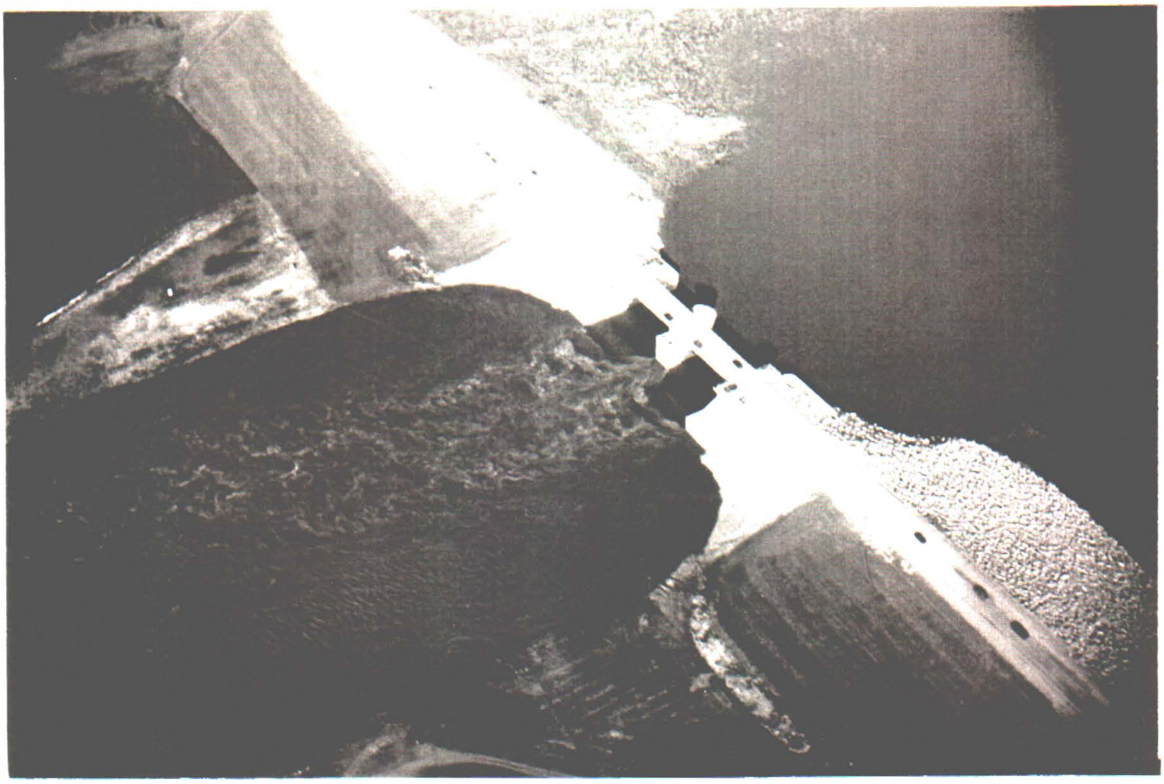


PHOTO NO. 1 - APRIL 1969 - NOTE POOR HYDRAULIC JUMP, WAVES

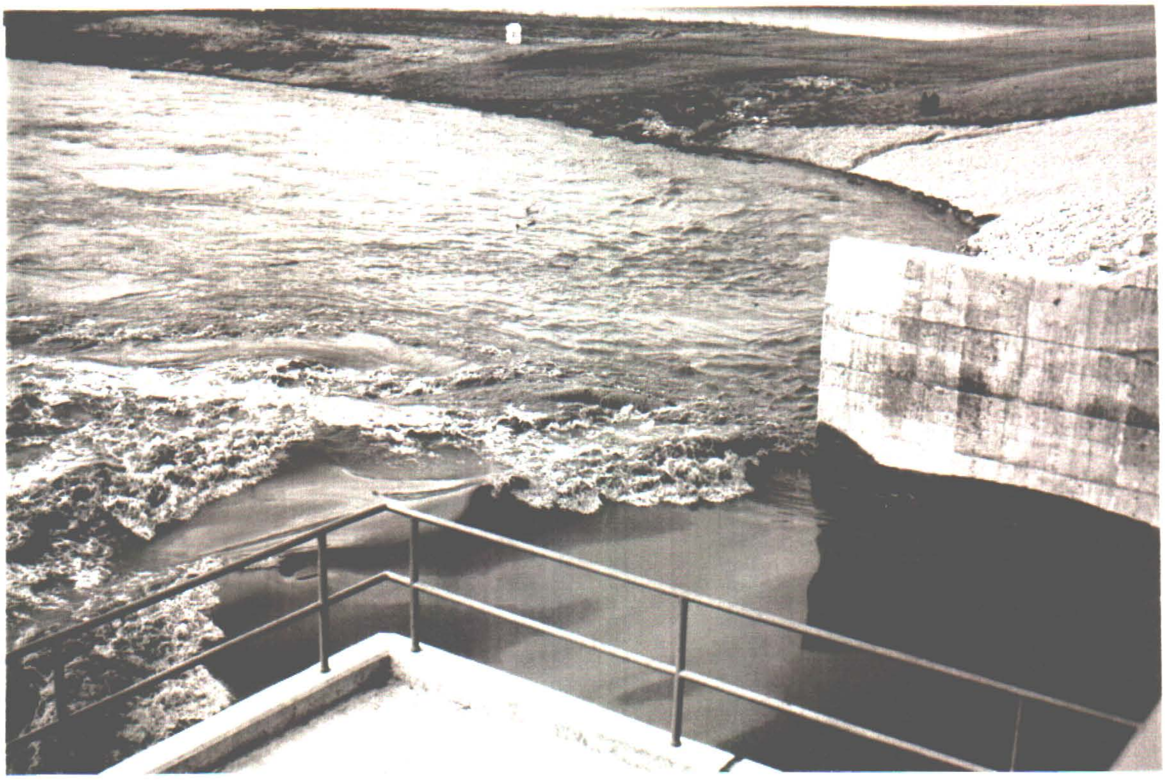


PHOTO NO. 2 - APRIL 15, 1969 - VORTICES AT BOUNDARY WITH HIGH VELOCITY FLOW

PHOTO NO. 3 - APRIL 15, 1969 - NOTE POOR JUMP

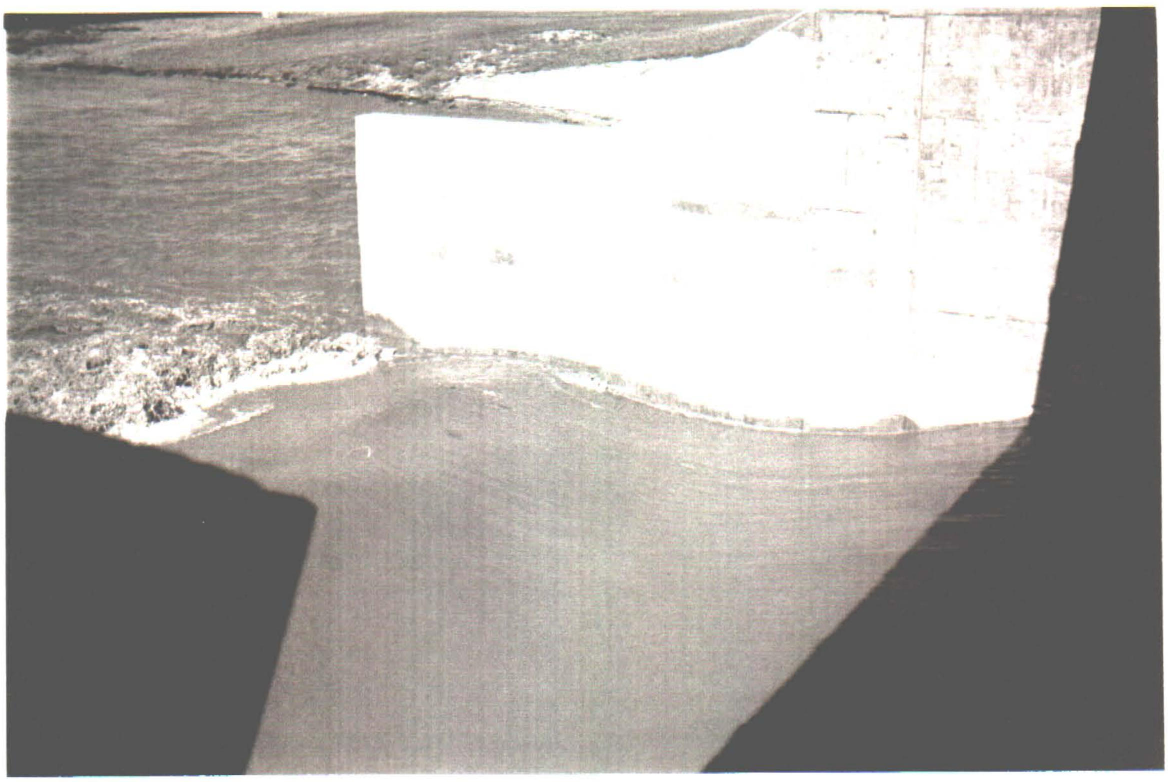


PHOTO NO. 4 - APRIL 15, 1969

PHOTO NO. 7 - APRIL 1974 - INLET STRUCTURE LOOKING UPSTREAM. NOTE GOOD HYDRAULIC JUMP

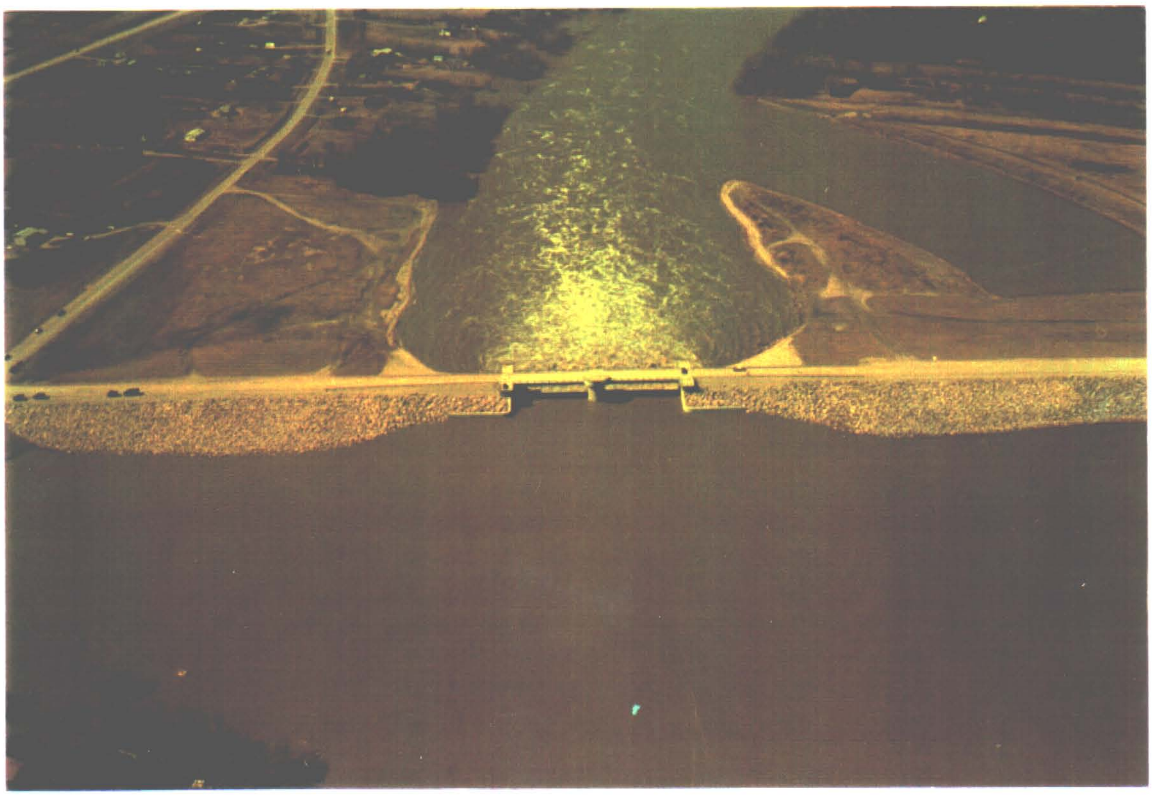


PHOTO NO. 8 - APRIL 1974 - INLET STRUCTURE LOOKING DOWNSTREAM

PHOTO NO. 5 - APRIL 1969 - NOTE GOOD HYDRAULIC JUMP, BUT JUMP EXTENDS PAST THE STRUCTURE

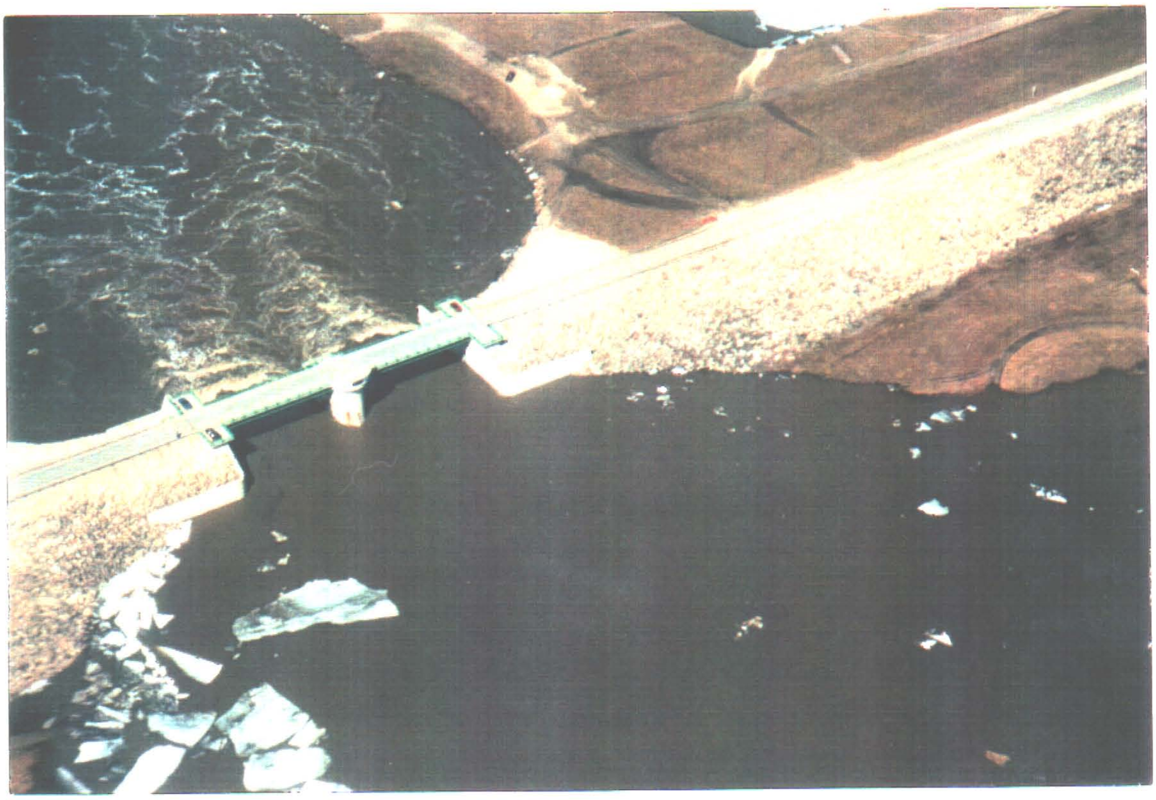


PHOTO NO. 6 - APRIL 1986 - HEAD DROP LESS THAN 4 FEET. NOTE POOR JUMP AND HIGH TURBULENCE

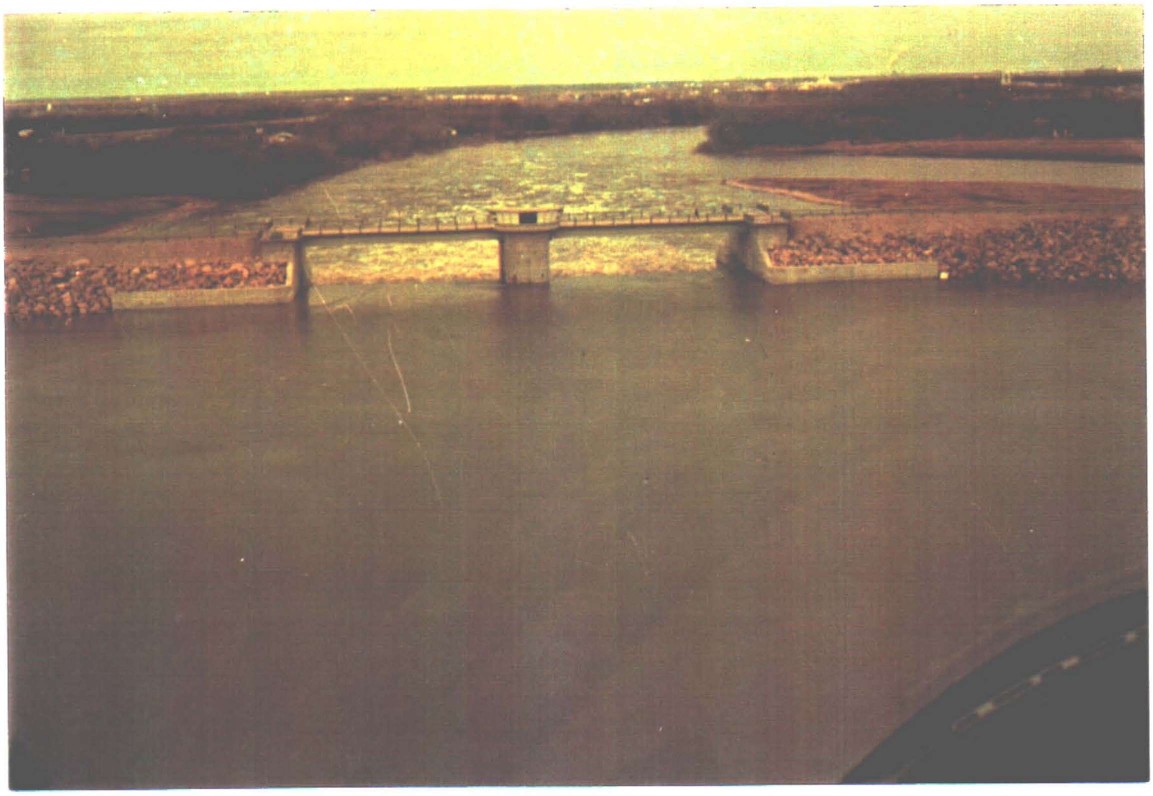


PHOTO NO. 9 - APRIL 1974 - INLET STRUCTURE LOOKING DOWNSTREAM

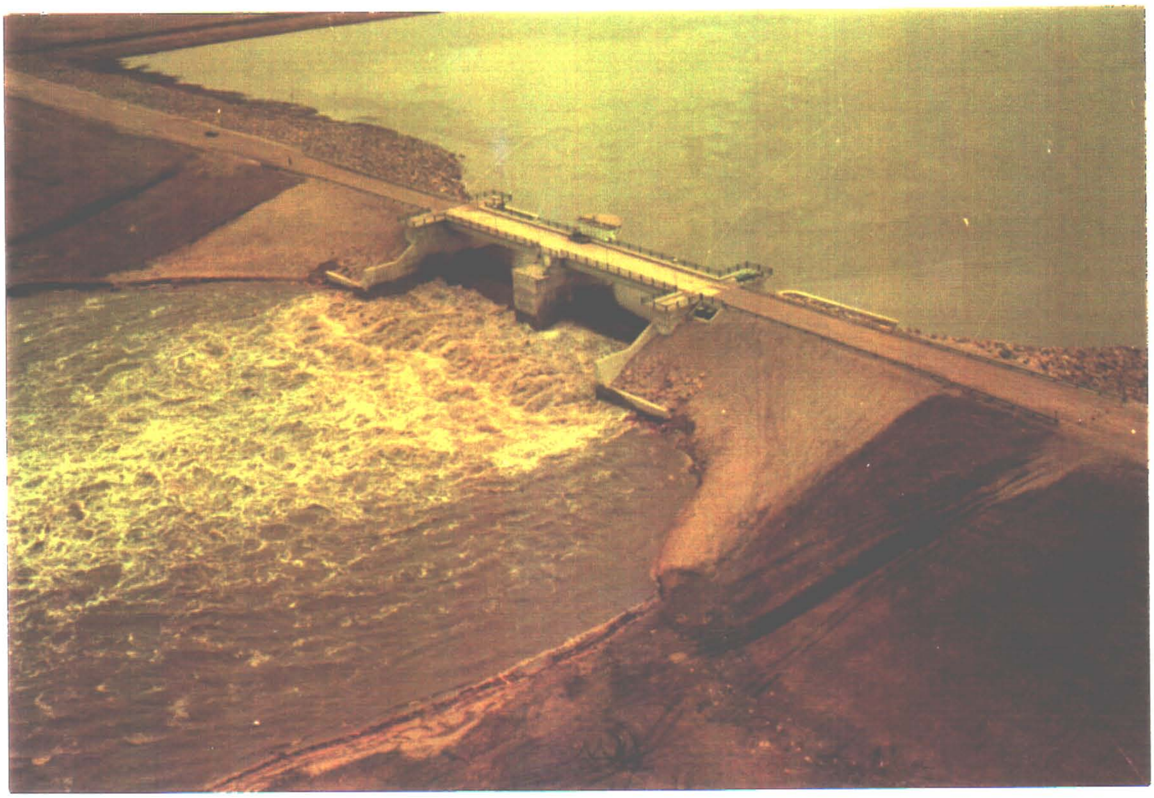


PHOTO NO. 10 - APRIL 1974 - HEAD DROP 17 FEET. NOTE GOOD HYDRAULIC JUMP, BUT JUMP OCCURS PAST END OF CONCRETE STRUCTURE

PHOTO NO. 11 - APRIL 1979 - NOTE POOR JUMP. HEAD DROP ±2 FEET

PHOTO NO. 12 - MAY 1979 - HEAD DROP ±14 FEET

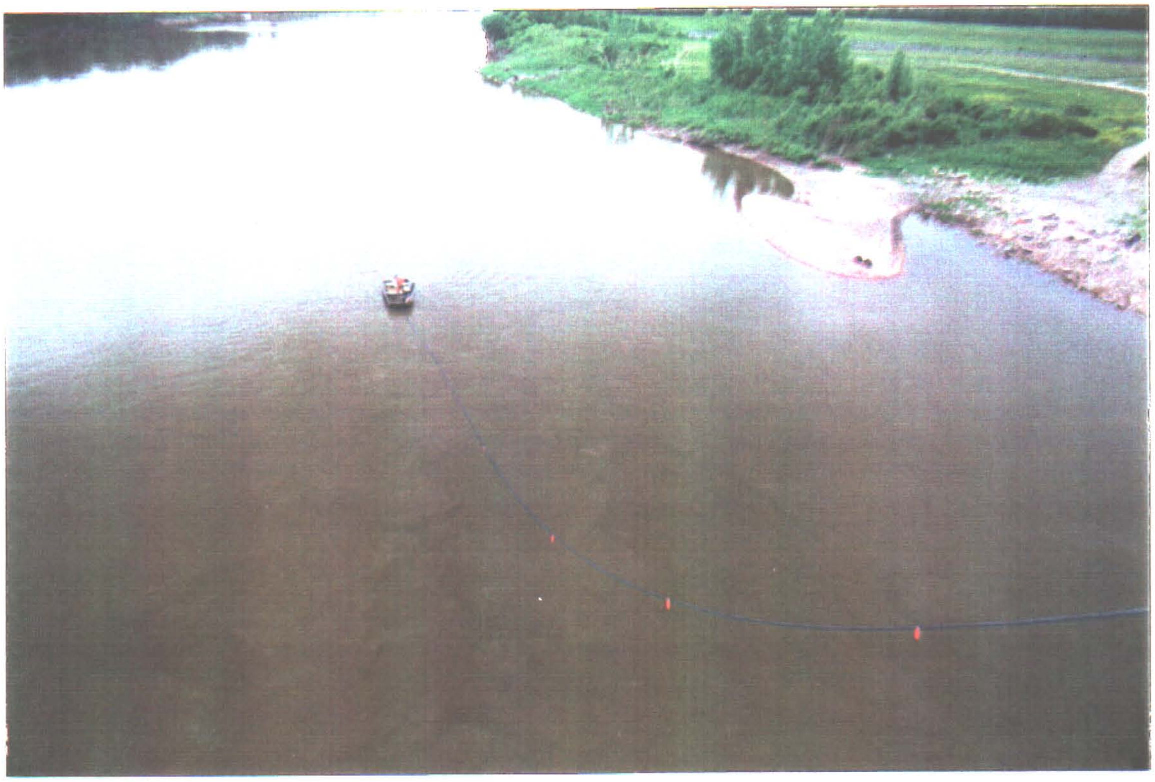


PHOTO NO. 13 - JUNE 1995 - DIVERS RECONNAISSANCE. NOTE DEPOSITION ON EAST BANK

PHOTO NO. 14 - JUNE 1995 - DOWNSTREAM ALIGNMENT OF BOAT USED IN DIVERS EXPLORATION. NOTE LINE MARKERS ON BANK

PHOTO NO. 15 - JUNE 1995 - NOTE DEPOSITION ON EAST BANK

PHOTO NO. 16 - JUNE 1995 - NOTE DEPOSITION ON WEST BANK

PHOTO NO. 17 - JUNE 1995 - EAST RIVERBANK LOOKING DOWNSTREAM

PHOTO NO. 18 - JUNE 1995 - TRANSIT USED TO ALIGN DIVER'S BOAT FOR UNDERWATER RECONNAISSANCE

PHOTO NO. 19 - JULY 1995 - EAST ABUTMENT FROM DEPOSITION ON EAST BANK, LOOKING UPSTREAM

PHOTO NO. 20 - JULY 1995 - WEST ABUTMENT FROM DEPOSITION ON WEST BANK, LOOKING UPSTREAM

PHOTO NO. 21 - JULY 1995 - WEST WING WALL. NOTE RIPRAP PROTECTION

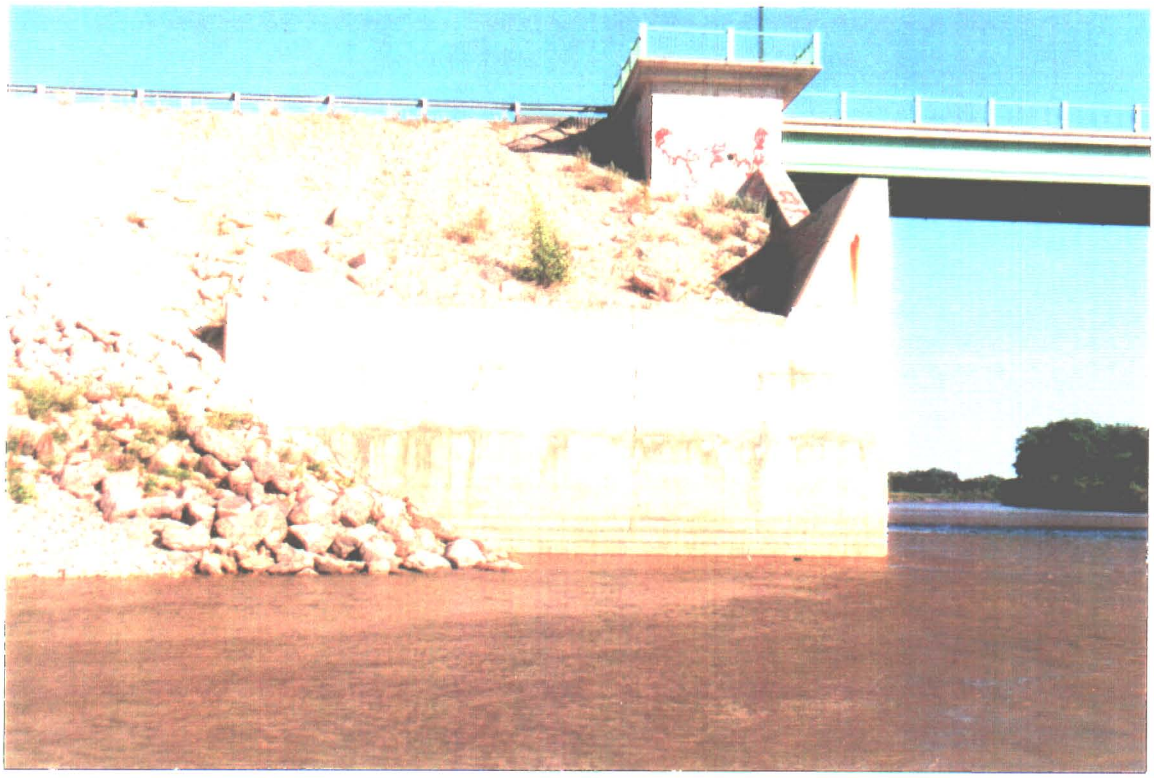


PHOTO NO. 22 - JULY 1995 - EAST WING WALL. NOTE RIPRAP PROTECTION