APPENDIX VI

April 1993

Physical Model Study of Saugus River and Tributaries, MA

Flood Damage Reduction Project

by N. J. Brogdon, J. S. Ashley, and J. E. Hite

Hydraulic Laboratory
Department of the Army
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Prepared for U.S. Army Engineer Division, New England
Waltham, Massachusetts

Contents

Preface v

Conversion Factors, US Customary to Metric (SI) Units of Measurements vii

1—Introduction 1

Background 1
Objective 2
The Model 2
Accuracy of measurements 3
Repeatability of the model 4

2—Model Test Conditions 6

Model Verification Procedure 6
Existing Conditions 7
Base Conditions 7

3—Description and Results of Conditions Tested 9

Existing and Base Conditions 9
Plan 1 10
Plan 2 10
Plan 2A 11
Plan 2B 11
Plan 2C+7 11
Plan 2C+3 and 2C-0 12
1.0 ft Sea Level Rise Tests 13
Emergency Gate Closure Tests 13
Tainter Floodgate Closure Tests (Normal Operation) 15

4—Riprap Design 16

5—Discussion of Test Results 22

Plan 2C+7 22
1.0 ft Sea Level Rise Test 24
Series I Emergency Gate Closure Tests 24
Series II Emergency Gate Closure Tests 25
Tainter Floodgate Closure Tests (Normal Operation) 25

Figures 1-78

Preface

The model study reported herein was requested by the U.S. Army Engineer Division, New England (NED). WES was requested to conduct physical, numerical and navigation model studies of the Saugus River system to determine the effects of a proposed tidal floodgate structure and alternate plans on tidal flushing, tide levels, sedimentation/erosion, tidal currents, and navigation.

This study was conducted in the Hydraulics Laboratory of WES during the period of October, 1990 to September, 1992, under the general supervision of Messrs. F. A. Herrmann, Director of the Hydraulics Laboratory; R. A. Sager, Assistant Director of the Hydraulics Laboratory; W. H. McAnally, Chief of the Estuaries Division (ED); D. R. Richards, Chief of the Estuarine Simulation Branch (HE-S); and N. J. Brogdon, Jr., project engineer for the physical model study. Mr. Richards was project manager for the study. Physical model technicians who assisted thoughout the investigation included Messrs. John S. Ashley; John T. Cartwright; and Charles Holmes.

Group visits were held in March and July 1992 at WES to provide the non-Federal interest and other members of the Corps an opportunity to review and comment on both the physical and numerical models and navigation studies. On 9 March, 1992 the non-Federal interest accompanied by representatives of NED and WES reviewed and discussed the physical and numerical models, river currents and operation/navigation of the model tanker and lobster boat through alternatives of the proposed floodgate structure (Plans 1 and 2C) during the flood and ebb tides. On 10 and 11 March,1992 St. Louis District (SLD), designers of the floodgate structure, met with NED, Headquarters representative (HQ) and WES to review the model results and discuss concerns including those raised by the sponsor.

Construction sequencing of the floodgate structure and navigation of the vessels through each construction phase for both the flood and ebb tides were reviewed in the physical model and discussed on 1 July ,1992 at a joint meeting with the non-federal sponsors, US Coast Guard, HQ, NED, SLD and WES. The meetings were very productive in identifying and resolving issues and informing representatives of the sponsor and Corps on the modeling efforts by WES.

This report was prepared by Messrs. Brogdon, Ashley, and Hite. Mrs. Marsha C. Gay, Information Technology Laboratory, WES edited this report.

Col. L. G. Hassell is the Commander of WES. Dr. Robert W. Whalin is the Director .

Conversion Factors, U.S. Customary to Metric (SI) Units of Measurements

Non-SI units of measurements used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acre-feet	1233.482	cubic metres
cubic feet	0.02831685	cubic metres
cubic yards	0.07645549	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
gallons	3.785412	cubic decimetres
inches	25.4	millimetres
miles (us nautical)	1.852	kilometres
miles (us statute)	1.609344	kilometres
square feet	0.09290304	square metres
square miles	2.589988	square kilometres

1 Introduction

Background

The Saugus-Pines River System (Figure 1) is located approximately 6 miles north of Boston Harbor. The Saugus River, and its Pines River tributary, has a watershed area of approximately 47 square miles. The Saugus River originates about 13 miles inland, and flows in a south-easterly direction before discharging into Broad Sound, between the cities of Lynn and Revere. The downstream 4.7 miles of the Saugus River is tidal. This portion of the system, including the Pines River system, is comprised of multiple storm-drained coastal urban areas and extensive tidal marshland. The entire 3 miles of the Pines River system is tidal. The rivers have a relatively small watershed, therefore freshwater discharge is low in comparison to tidal flows. The river systems are classified as well-mixed. Several cities and towns, including Revere, Lynn, Saugus, and surrounding area are subjected to periodic flooding resulting from strong north-easterly storms and hurricanes.

The U.S. Army Engineer Division, New England (NED) was authorized to develop a plan consisting of seawall and revetment construction, sand dune reconstruction, and a tidal floodgate to protect these communities from periodic flooding. The central feature of the protection scheme is a floodgate structure in the entrance of the Saugus-Pines River system.

The U.S. Army Engineer Waterways Experiment Station (WES) was requested to assist the NED in the design of the structure. Two types of models, physical and numerical were used thoughout the course of the investigation. A three-part model study was developed. Part one employed a global numerical model using the USACE TABS-MD modeling system. This model was used to evaluate overall effects on circulation, tide levels, and sedimentation. It was also utilized to generate boundary conditions for the physical model, the second part of the overall study. The physical model was used to obtain detailed information on current velocities and current patterns in the immediate vicinity of the proposed structure for use in the structure design, and for use in the numerical model study. The third part, a navigation study using the physical model, evaluated navigation safety for the several proposed plans and also for the various construction stages of the structure. Current velocity data generated from the physical model were used in the riprap design analysis.

Objective

The objective of the model study was to provide the most cost effective and environmental sound design configuration for the Saugus River flood gates, (with cost effective defined by the minimum number of gates necessary to provide safe and effective operation of the structure,) and to evaluate the impact of the structure on: (a) basin water surface elevations, (b) water circulation, (c) current velocities, (d) sedimentation and erosion, (e) navigation, and (f) water surface elevations and current velocities in conjunction with a 1.0 ft. rise in sea level.

The primary objective of the physical model was to provide: (a) detailed hydrodynamic information for the structure design, (b) detailed water surface elevation and current velocity data on the three-dimensional flow field in the immediate vicinity of the structure for use in the numerical model part of the study, and in the design of riprap protection. The physical model was also used to conduct the navigation study which is reported separately.

The Model

See Model Photos on Project Website under "The Project" (Floodgate).

The limits of the physical model are shown in Figure 2. Figures 3 and 4 are photographs of the model. The model was approximately 110 ft long and 80 ft wide at the widest point (about 4500 square feet). The model reproduced an area beginning about 600 ft. south of Point of Pines, and encompassed a portion of Lynn Harbor to a point on the north shore about 2000 ft north-east of the General Edwards Bridge. The upstream limits of the model extended just upstream from the confluence of the Saugus and Pines Rivers, approximately 1200 ft upstream of the General Edwards Bridge. Installed in the model to model scale were the General Edwards Bridge piers, the Point of Pines Yacht Club Piers, the MDC public fishing pier (removed for plan testing), the General Electric pipeline pier, and the pier on the south shore immediately upstream from the General Edwards Bridge.

The model was constructed to an undistorted scale of 1:50. Using Froudian relations the following ratios can be computed:

Characteristics	Ratio	Model Prototype
Length	1	50
Area	1	2500
Volume	1	125000
Time	1	7.0711
Velocity	1	7.0711
Discharge	1	17677.75

Horizontal grid coordinates of the model were based on the State coordinate system, and the vertical control was based on the National Geographic Vertical Datum (NGVD). The model was housed inside an existing shelter to protect it from the weather and to permit uninterrupted operation. Table 1 shows the State X-Y coordinates for the water surface elevation stations, and Table 2 shows the coordinates for the current velocity stations monitored thoughout the course of this model investigation.

The model was equipped with all the necessary appurtenances to measure water surface elevations, current velocities, current directions, and discharge. The model was not tidal; however, flow in either the ebb or flood direction was reproduced using a centrifugal pump linked to a system of pipes and valves that allowed flow reversal. Discharges into and withdrawn from 5 headbays were monitored by inline flow monitors. Current velocity measurements were obtained with a miniature Price meter. Each meter had 5 cups, constructed of a light weight plastic material, 0.04 ft (2 ft prototype) in diameter mounted on a thin horizontal wheel 0.08 ft (4 ft prototype) in diameter. The center of the cups were 0.05 ft (2.5 ft prototype) from the bottom of the frame. The meters were calibrated frequently and were capable of measuring current speeds as low as 0.028 fps (0.2 fps prototype). Current directions were obtained using a very lightweight vane that pivoted with the current. Current directions were read to the nearest 10 degrees.

Current velocity measurements for the gate closure and emergency gate closure tests were obtained with a Model 403 Low-Speed velocity probe and digital indicator having a capacity to record model currents speeds in excess of 25 fps (prototype). Current speeds occurring during these tests exceeded the capability of the miniature price meters.

Accuracy of Measurements

Water surface measurements were made with point gages graduated to 0.001 ft (0.05 ft prototype) and were generally read to the nearest 0.0005 ft. Current velocity measurements were generally read to the nearest 0.20 fps (prototype). Discharge enterning/exiting the model were reproduced within an accuracy of $\pm$ 1.0 percent.

Repeatability of the Model

During the early stages of model testing it was observed that current velocity measurements were not as repetitive as usually observed in similar models. These discrepancies were more pronounced during ebb flow conditions. A check was first made to ensure that the pump was not surging, resulting in varying discharge and water surface elevations at the control boundaries. The pump and resulting water surface elevations at the control boundaries were found to be performing in a satisfactory manner. The next step was to check the calibration of the current velocity meters used in determining the current magnitude. The calibration check showed that the problem was not in the meters. Following these equipment checks, visual observations were made, and these observations revealed that currents were oscilating as a result of turbulence at the confluence of the Pines and Saugus Rivers. This was concluded to be a natural occurrence (vortex shedding) and could not be eliminated from the model. This condition probably occurs in the prototype. Thereafter, in an effort to minimize the impact of this phenomenal, each sampling location was monitored 5 times, instead of the usual 3, in order to achieve a more uniform average current velocity.

Table 3 and Figures 5 and 6 show data resulting from tests (spring tide-ebb direction) conducted to determine effects of current meters. Figures 5 and 6 were constructed from data presented in Table 3. Two stations were monitored (mid-depth) for a period of 15 minutes (1 hr 46 minutes prototype) with sampling made every 15 seconds or every 1 minute 46 seconds prototype. Station 2.58-E was located on the navigation channel centerline upstream of the General Edwards Bridge. Station 2.58-G was located in the 2nd bay of the bridge north of the navigation opening. Each station was monitored with meters C-1 and C-3. Data in Table 3 show that with meter C-1 the average velocity at station 2.58-E was 1.935 fps, and with meter C-3, the average velocity was 1.932. The maximum current velocity measured with meters C-1 and C-3 were 2.0 fps and 2.1 fps, respectively. The minimum current velocity measured by each meter was 1.8 fps. Current velocities at station 2.58-E were fairly uniform thoughout the 15 minute sampling period, as the maximum deviation (maximum-minimum) was only 0.20 fps for meter C-1 and 0.3 fps for meter C-3.

Similar observations made at station 2.58-G were more erratic than currents observed at station 2.58-E. Average current velocity over the 15 minute sampling period at station 2.58-G with each of the two meters, C-1 and C-3, were fairly consistent, as the average obtained with meter C-1 was 1.600 fps and the average with meter C-3 was 1.635. However, the difference between maximum and minimum velocity varied from 1.8 fps to 1.2 fps with meter C-1 and from 1.9 fps to 1.3 fps with meter C-3. The deviation (maximum-minimum) was 0.6 fps with each meter. Station 2.58-G was located in the zone where the greatest effects resulting from the merging of the Pines and Saugus Rivers was realized.

Current velocities were also observed in the Pines and Saugus Rivers (water surface elevation Stations 1 and 2). These observations are shown in Table 4 and in Figures 7 and 8. These observations were made over a period of 10 minutes (1 hr 10 minutes 49 seconds prototype) with the same meter. Data were collected every 15 seconds (1 minute 46 seconds prototype) to measure pump consistency. These observations were consistent with observations made on similar models. The observations at Station 1 (Pines River) were slightly more erratic (maximum 2.8 fps-minimum 2.5 fps) than observations made at Station 2 (Saugus River). Maximum current velocity at Station 2 was 1.1 fps and the minimum was 0.9 fps. The Pines River discharged about 77 percent of the total flow entering the system and the effects of vortex shedding was slightly greater at Station 1.

Another series of check tests was conducted during which time station 2.44 B (located on the navigation channel centerline upstream of the proposed structure) was monitored for 20 minutes (2 hours 21 minutes 25 seconds prototype) simultaneously with observations made at station 2.58 E, 2.58 F and 2.58 G. These data presented in Table 5 show Station 2.44 B current velocities were consistent thoughout the 20 minute sampling period as the maximum deviation was only 0.2 fps; Whereas, the maximum deviation at Stations 2.58 E, 2.58 F and 2.58 G was 0.4 fps, 0.7 fps, and 0.5 fps, respectively. Stations 2.58 E, 2.58 F, and 2.58 G were closer to the confluence of the Pines and Saugus Rivers and therefore showed greater effects of the vortex shedding phenomenal caused by the merging of the Pine and Saugus Rivers. Currents in the vicinity of the proposed structure were fairly consistent and repetitive thoughout the sampling period.

There were no problems of repeatability when the model was set to reproduce flood direction flow conditions.

2 Model Test Conditions

Model Verification Procedure

The global numerical model was used to generate boundary condition data which was used in the physical model verification. Flow entering/exiting (flood-ebb) from the ocean end of the model was separated by three headbays, labeled A, B and C in Figure 2. Two additional headbays, located on the Pines and Saugus rivers, provided for flow entering/exiting from the landward end of the model. The data consisted of water surface elevations and discharges at the 5 model headbays for existing spring tide and neap tide conditions. These data were results of dynamic numerical model tests, and represent a specific time during the tidal cycle when discharges entering/exiting the system were at their peak. These peak discharge periods represent periods when maximum current velocity occurs. Physical model boundary data for Existing spring tide conditions were generated from numerical model tests reproducing a tidal range of 13.10 ft. Maximum flood and ebb current velocity occurred at hours 22.5 and 26.5, respectively, during the numerical model test tidal cycle. Neap tide boundary data resulted from numerical model tests reproducing a tide of range 6.00 ft. Maximum flood and ebb current velocity occurred at hours 22.5 and 15.5, respectively, during the numerical model tidal cycle. Water surface elevations at the five (5) headbays occurring at the times of peak discharge were also calculated by the numerical model tests.[1]

The flows entering and exiting through Headbays A, B, and C were adjusted to conform to the numerical model flows calculated at the Pines and Saugus River boundaries. This method of calculating discharge for the ocean boundaries was done in this manner for all tests conducted in the physical model.

The discharge at each of the five headbays was set and monitored though-out each test. The head or control water surface elevation was set at the Pines River boundary for all ebb flow conditions and at Headbay A boundary for all

[1] Lin, J. C. and D. R. Richards, 1993. *Saugus River and Tributaries*, MA; "Flood Damage Reduction Project, Numerical Model Study" USAE Waterways Experiment Station, Vicksburg, MS.

flood flow conditions. No adjustments were made to control the water surface at the other boundaries.

Existing Conditions

Table 6 shows the physical model boundary conditions for Existing spring tide conditions (tide range of 13.10 ft). The total discharge entering and exiting the system was 21,240 cfs, and 21,980 cfs, flood and ebb directions, respectively. The water surface elevation at Headbay A was maintained at elevation 3.47 ft NGVD for flood condition. The Pines River headbay water surface elevation was maintained at elevation 4.95 ft NGVD for the ebb flow condition.

Table 7 shows the physical model boundary conditions for Existing neap tide conditions (tide range of 6.00 ft). The total discharge entering and exiting the system was 8,971 cfs, and 8,005 cfs, flood and ebb directions, respectively. The water surface elevation at Headbay A was maintained at elevation 1.462 ft NGVD for flood condition. The Pines River headbay water surface elevation was maintained at elevation -0.252 ft NGVD for the ebb flow condition.

Base Conditions

The physical model Base bathymetric conditions were identical to Existing conditions, except that abandoned I-95 highway embankment had been cut or enlarged to allow additional flow into the marsh. This cut, beyond the limits of the physical model, was located on the Pines River approximately 1.75 miles inland of the General Edwards Bridge. Therefore, the global numerical model was used to generate the physical model boundary data for this condition. Details of the I-95 cut, dimension and flow passage etc. are discussed in detail in the numerical model report.[1] These boundary data, for conditions of both spring and neap tides reproduced in the physical model, are shown in Tables 8 and 9, respectively. The total discharge entering and exiting the system for spring tide conditions (Table 8) was 22,100 cfs and 22,500 cfs, flood and ebb directions, respectively. Water surface elevations for Base spring tide conditions were maintained at 6.23 ft NGVD (Headbay A) and 2.85 ft NGVD (Pines River headbay) for flood and ebb flow conditions, respectively. The total discharge entering and exiting the system for Base Neap tide conditions (Table 9) was 8,330 cfs and 7,630 cfs, flood and ebb directions, respectively. Water surface elevations at boundary headbay A was maintained at 1.461 ft NGVD for flood flows. During ebb flow testing, the Pines River headbay water surface elevation was maintained at elevation -0.253 ft NGVD. The tide ranges, 13.10 ft for spring tide and 6.0 ft for neap

---

1 Lin, J. C. and D. R. Richards, 1993. *Saugus River and Tributaries*, MA; "Flood Damage Reduction Project, Numerical Model Study" USAE Waterways Experiment Station, Vicksburg, MS.

tide, were identical to Existing conditions. However, the time of occurrence of peak current velocity was slightly different due to the additional storage area provided by the cut through I-95. Time of occurrence during the tidal cycle of maximum flood and ebb current velocities for Base spring tide conditions were at hours 22.5 and 26.0, respectively. The time of occurrence of maximum flood and ebb current velocities for the Base neap tide were at hours 22.95 and 15.75, respectively. Base condition water surface elevation and discharge boundary data were used for all plan testing. Plan 2C (final plan) was also tested for Existing spring tide boundary conditions.

3 Description and Results of Conditions Tested

Existing and Base Conditions

Existing and Base conditions (Figure 9) each consisted of existing bathymetric conditions. The only difference between Existing and Base conditions was the I-95 cut. Installation of the I-95 cut altered existing water surface elevations, discharges, and time of occurrence at the model boundaries, as discussed in the previous section.

Water surface elevations and current velocity data resulting from Existing spring tide conditions are presented in Tables 10 and 11, respectively. Water surface elevations and current velocity data resulting from Existing neap tide conditions are presented in Tables 12 and 13, respectively.

Water surface elevations and current velocity data resulting from Base Spring tide conditions are presented in Tables 14 and 15, respectively. Water surface elevations and current velocity data resulting from Base neap tide conditions are presented in Tables 16 and 17, respectively.

A. For the purpose of this report, only data collected on Ranges 2.43 and 2.44 (immediately seaward and landward of the proposed structure, respectively) will be compared in profile form. Data collected at other locations are presented in the tables. Figures 10 and 11 show water surface elevation profiles, for spring tide Existing verus spring tide Base, flood and ebb flows, respectively. Average current velocity (depth averaged) profiles comparing spring tide Existing and spring tide Base for flood and ebb conditions are shown in Figures 12 and 13, respectively. Profiles for flood and ebb flow directions comparing neap tide Existing and neap tide Base conditions water surface elevations and depth averaged current velocities are shown in Figures 14-15, and 16-17, respectively.

Plan 1

Description of plan. Plan 1 (Figure 18) consisted of the original structure design with nine 50 ft wide floodgates north of the 100 ft wide navigation opening and one 50 ft wide floodgate south of the navigation opening. The bottom elevation of the navigation opening was existing elevation (all deeper than project depth,-18 ft NGVD). The bottom or seat elevation of the tainter floodgates was -14 ft. NGVD. Approaches to the floodgates were dredged to elevation -14 ft. NGVD, both upstream and downstream of the structure. The roadway or support structure connecting the floodgates had an inverted "U" shape with the bottom elevation at 0.0 ft NGVD.

Test results. Tests with Plan 1 installed were conducted for Base spring tide conditions only. Water surface elevation data are presented in Table 18. Current velocity data (flood direction only) were collected at ranges 2.43 and 2.44 (downstream and upstream of proposed structure, respectively), and are presented in Table 19.

Spring tide water surface elevation profiles comparing Plan 1 and Base data at ranges 2.43 and 2.44 for flood and ebb flows are shown in Figures 19 and 20, respectively. Depth averaged spring tide current velocity data (Plan 1 versus Base) for flood and ebb flows are shown in Figures 21 and 22, respectively.

Plan 2

Description of plan. Plan 2 (Figure 23) consisted of five and three floodgates north and south of the navigation opening, respectively. With the installation of Plan 2, four floodgates on the north end of the proposed structure were eliminated, and two additional floodgates were added on the south side of the navigation opening. Four new current velocity stations (2.43 A1, 2.43 A2, 2.44 A1, and 2.44 A2) were added to show effects of the plan in these new floodgate openings. These new stations were monitored during all subsequent testing. Approaches to the floodgates were dredged to elevation -14.0 ft NGVD as indicated by the shaded areas shown in Figure 23. No change in depth was made in the navigation opening. The roadway support structure was the same as Plan 1.

Test results. Tests were conducted for only Base spring tide conditions. The resulting water surface elevations are presented in Table 20, and the current velocity data are presented in Table 21.

Spring tide water surface elevation profiles comparing Plan 2 and Base at ranges 2.43 and 2.44 for flood and ebb flows are shown in Figures 24 and 25, respectively. Depth averaged spring tide current velocity data (Plan 2 versus Base) for flood and ebb flows are shown in Figure 26 and 27, respectively.

Plan 2A

Description of plan. Plan 2A (Figure 28) had the same number and location of floodgates as Plan 2. The dredged approach shape and elevation to the southern three floodgates remained the same as Plan 2. However, the dredged approach to the northern floodgates was enlarged considerably as indicated by the shaded area in Figure 28. The roadway support structure remained the same as Plans 1 and 2.

Test results. Tests with Plan 2A installed were conducted for only conditions of a Base Spring tide. Water surface elevations and current velocity data resulting from this plan are presented in Tables 22 and 23, respectively.

Spring tide water surface elevation profiles comparing Plan 2A and Base data at ranges 2.43 and 2.44 for flood and ebb flows are shown in Figures 29 and 30, respectively. Depth averaged spring tide current velocity data (Plan 2A versus Base) for flood and ebb flows are shown in Figure 31 and 32, respectively.

Plan 2B

Description of plan. Plan 2B (Figure 33) structure was the same as Plan 2, however, the dredged approaches to the floodgates, both north and south of the navigation opening were enlarged as indicated by the shaded areas in Figure 33.

Test results. The results of Plan 2B on water surface elevations and current velocities are presented in Tables 24 and 25, respectively. Tests were conducted with the model operating for Base spring tide flood direction only. No data was collected with the model operating in the ebb direction.

Spring tide flood direction profiles comparing Plan 2B and Base water surface elevations and depth averaged current velocities are shown in Figures 34 and 35, respectively.

Plan 2C+7

Description of plan. The number of floodgates for Plan 2C+7 (Figure 36) was not changed from that of Plan 2. However, the roadway structure support system bottom elevation was raised to elevation +7.0 ft NGVD, and its shape altered from an inverted "U" shape to an inverted "L" shape. The approach elevations to both the north and south floodgates remained at elevation -14 ft NGVD, however, the shape of the dredged areas changed as indicated by the shaded areas in Figure 36. Plan 2C+7 floodgate position, number, and dredged approach configuration were recommended as the final plan following the navigation study.

Test results. Plan 2C+7 was tested for both Base spring tide and Existing spring tide conditions. The results of tests conducted for Base spring tide conditions are presented in Table 26 (water surface elevations) and in Table 27 (current velocities). Existing spring tide condition results are presented in Tables 28 and 29, water surface elevations and current velocities, respectively.

Spring tide water surface elevation profiles comparing Plan 2C+7 and Base data at ranges 2.43 and 2.44 for flood and ebb flows are shown in Figures 37 and 38, respectively. Depth averaged spring tide current velocity data (Plan 2C+7 versus Base) for flood and ebb flows are shown in Figure 39 and 40, respectively.

Spring tide water surface elevation profiles comparing Plan 2C+7 (existing boundary water surface elevations and discharge) and Existing (no structure) at ranges 2.43 and 2.44 for flood and ebb flows are shown in Figures 41 and 42, respectively. Depth averaged spring tide current velocity data (Plan 2C+7 versus Existing) for flood and ebb flows are shown in Figure 43 and 44, respectively.

Plans 2C+3 and 2C-0

Description of plans. Plans 2C+3 and 2C-0 number of floodgates, position, and dredged approaches were identical to Plan 2C (Figure 36). The only difference was the bottom elevation of the tainter gates in the raised position. The bottom elevation of the floodgates in the raised position for Plan 2C +3 was set at +3.0 ft NGVD, and 0.0 ft NGVD for Plan 2C-0.

Test results. The water surface elevation and current velocity data resulting from tests conducted with Plan 2C+3 installed and reproducing Base spring tide conditions are presented in Tables 30 and 31, respectively. Tables 32 and 33 present water surface and current velocity data, respectively, resulting from tests conducted with Plan 2C-0 for conditions of a Base spring tide. Current velocity data was collected only on ranges 2.43 and 2.44 with Plan 2C-0 installed.

Spring tide water surface elevation profiles comparing Plan 2C+3 and Base data at ranges 2.43 and 2.44 for flood and ebb flows are shown in Figures 45 and 46, respectively. Depth averaged spring tide current velocity data (Plan 2C+3 vers Base) for flood and ebb flows are shown in Figures 47 and 48, respectively.

Spring tide water surface elevation profiles comparing Plan 2C-0 and Base data at ranges 2.43 and 2.44 for flood and ebb flows are shown in Figures 49 and 50, respectively. Depth averaged spring tide current velocity data

(Plan 2C-0 versus Base) for flood and ebb flows are shown in Figures 51 and 52, respectively.

1.0 ft. Sea Level Rise Tests

Description of tests: The global numerical model was used to generate boundary conditions for a 1.0 ft. increase in sea level. Base (I-95) bathymetric conditions with Plan 2 C+7 installed in the numerical model used to generate the physical model boundary data. A tide range of 13.10 ft, same as Base, was used in the numerical model to calculate the peak discharge/current velocities for use in the physical model tests. The time of occurrence of the peak discharge during the flood flow portion of the tidal cycle was the same as calculated for Base (hr. 22.5). However, the peak discharge/current velocities for the ebb flow portion of the tidal cycle occurred slightly later than base, hour 26.75 for the 1.0 ft sea level rise versus hour 26.5 for base conditions. Table 34 shows the boundary data used for the physical model tests. Total discharge entering the system for a flood condition (Headbays A, B and C) was 19,800 cfs. The control water surface elevation at Headbay A was 7.23 ft NGVD. Total discharge entering the system for an ebb condition (Pines and Saugus Rivers) was 26,200 cfs, and the control water surface elevation at the Pines River was 2.80 ft NGVD.

Tests results: Tests were conducted for only Base bathymetric conditions (I-95 cut with Plan 2C+7 in place). Resulting water surface elevation and current velocities for both flood and ebb flow conditions are presented in Tables 35 and 36, respectively.

Spring tide water surface elevation profiles comparing 1.0 ft sea level rise and Plan 2C+7 Spring Tide data at ranges 2.43 and 2.44 for flood and ebb flows are shown in Figures 53 and 54, respectively. Depth averaged spring tide current velocity data (1.0 ft sea level rise versus Plan 2C+7 spring Tide) for flood and ebb flows are shown in Figures 55 and 56, respectively.

Emergency Gate Closure Tests

Description of Series I tests. Several tests were conducted in the physical model to simulate conditions that would occur in an emergency situation when a gate could not be closed in anticipation of an approaching storm tide. Tests were conducted with: (a) the miter gate (navigation opening) open and all the tainter gates closed; (b) the miter gate and all tainter gates closed except a single tainter gate south of the navigation opening (Gate A); and (c) miter gate and all tainter gates closed except a single tainter gate north of the navigation opening (Gate C). Each of the above tests was conducted for several ocean water surface elevations and discharges. All tests were conducted with Plan 2C+7 installed in the model and operating with flow in the flood direction. For the Series I tests, the model was started with conditions of a spring tide flood condition (water level and discharge set for this condition) and allowed to stabilize, then the appropriate gate condition was set up in the model. In the single tainter gate open tests, the discharge had to be reduced, since a single gate could not pass the original 100 percent of spring tide discharge. Changes in discharge resulted in changes in the original water surface elevation at the control headbay. The model was allowed to stabilize for each condition prior to collection of data.

Tests results. Figures 2 and 57 show the location of water surface and current velocity stations monitored during this test series. Table 37 shows the discharge at the various headbays for the emergency gate closure tests conditions. Table 38 shows resulting water surface elevations and current velocity data collected for three discharge conditions conducted with the miter gate open (gate B) and all tainter gates closed. Table 39 shows data resulting from three discharge conditions conducted with the miter gate closed and all tainter gates closed, except gate A. Table 40 shows data collected for three discharge conditions with the miter gate and all tainter gates closed, except gate C. Current velocities were measured only at the bottom.

Station locations for Series I tests are shown in Figure 57. Current velocity profiles start at range 2.43, Station A, B, or C. Ranges, located upstream of range 2.44, were numbered in feet upstream from range 2.44. (One foot in the model is equivalent to 50 feet prototype.) Figure 58 shows bottom depth velocity profiles taken with gate B (miter) open for discharges of 22,100 cfs, 11,050 cfs and 5,525 cfs. Figures 59 and 60 show similar velocity profiles with tainter gate A or C open, respectively, for discharges of 11,050 cfs, 5,525 cfs and 3,315 cfs.

Description of Series II gate closure tests. Following Series I tests, a second series of emergency gate closure tests were conducted. These tests conditions represent the most adverse conditions that could be expected to occur during a close down operation. Figure 61 shows the location of stations monitored during this series of tests, and are different from the Series I tests. Stations A and C were located 25 ft (prototype) south and north of the gate centerline (Station B), respectively. Tests were conducted with either gate A or C open. All other gates were closed. Tests were also conducted with gate C closed 50 percent of total closure. Table 41 shows the relationship between total discharge entering the system; ocean water surface elevation; bay water surface elevation; and head differentials in feet for the conditions investigated. Tests were conducted for 6 discharges. As in the Series I tests, Series II tests were conducted with Plan 2C+7 installed in the model and operating in the flood direction only.

Tests results. The results of Series II Emergency Gate Closure tests are presented in Tables 42-47 and Figures 62-67.

Figure 62 (Table 42 data) shows bottom current velocity profiles at Station A, B, and C for a discharge of 13,800 cfs with tainter gate A open only. Figure 63 (Table 43 data) shows a similar bottom profile with only tainter gate A open for a discharge of 12,526 cfs. Bottom current velocity profiles with only tainter gate C open and with a discharge of 13,800 cfs are shown in Figure 64 (Table 44 data). Figure 65 (Table 45 data) shows the tainter gate C velocity profiles for a discharge of 12,773 cfs. Tainter gate C velocity profiles for a discharge of 16,560 cfs are shown in Figure 66 (Table 46 data). Figure 67 (Table 47 data) shows velocity profiles for a discharge of 9,200 cfs with tainter gate C closed 50 percent of total closure.

Tainter Floodgate Closure Tests (Normal Operation)

Description of tests. Tests were conducted with conditions consisting of: (a) all gates open 100 percent (miter and tainter); (b) all tainter gates open and miter gate closed; (c, d, and e) miter gate closed; tainter gates closed 25, 50 and 75 percent of total closure, respectively. Plan 2C+7 was installed in the model thoughout the tests. Current velocities data collected during these tests were used in the design of the riprap blanket that will be positioned on either side of the proposed structure. The location of stations monitored during the tests are shown in Figure 68. The analysis of these data in respect to riprap design are discussed later in this report.

The original boundary condition setup for this series of tests was identical to the Series I tests described earlier. The model was set up (head and discharge) for spring tide flood conditions. The discharges varied from 100 percent to 15 percent of total discharge (see Table 37). The water surface elevation at the control headbay (headbay A) varied as discharge varied.

Test results. The data resulting from this series of tests are shown in Tables 48-55 and in Figures 69-76. Data included in the tables show: (a) total discharge entering and exiting the system; (b) head differential between Headbay A and the Pines River; (c) water surface elevation at ranges 2.43 and 2.44; (d) bottom current velocities seaward and landward of the tainter gates (A2-G).

Data resulting from tests conducted with all gates open and a discharge of 22,100 cfs is shown in Table 48 and Figure 69. Data resulting from tests with all tainter gates open and the miter gate closed for a discharge of 22,100 cfs are shown in Table 49 and Figure 70. Tables 50 (Figure 71) and 51 (Figure 72) show data resulting from conditions of all tainter gates 25 percent closed with discharges of 22,100 cfs and 11,050 cfs, respectively. Tables 52 and 53 (Figures 73 and 74) show results with tainter gates 50 percent closed and with discharges of 22,100 cfs and 11,050 cfs, respectively. Tables 54 and 55 (Figures 75 and 76) show data resulting from tests conducted with the tainter gates 75 percent closed and discharges of 11,050 cfs and 5,525 cfs, respectively.

4 Riprap Design

An analysis of current velocities measured in the Saugus River physical model was performed to determine riprap requirements for the structure. The riprap plan was designed based on normal operating schedule (no gate malfunctions). A normal operation schedule consisted of closing the 100-ft wide miter gate first and then closing the eight 50-ft wide tainter gates as quickly as possible prior to the effects of approaching storm surge.

The physical model was not designed to operate with unsteady flows. To simulate the normal gate operations, the gates and the discharges in the three ocean headbays and the Pines and Saugus Rivers were set to provide the conditions shown in Table 37. Bottom current velocities (approximately 2.0 ft above the bottom) were measured at one station on the ocean side of the structure, in the center of the structure, and at five stations on the bay side of the structure. Current velocity station locations for this series of tests are shown in Figure 68. Plan view of the resulting current velocity measurements are shown in Figures 69-76.

Bay side riprap design for normal gate closure operations. The highest current velocities were observed with the miter gate closed and the tainter gates closed 75 percent of total closure, for a flood flow direction discharge of 11,050 cfs. These current velocities measurements are shown in Figure 75. A current velocity of 12.1 fps was measured in the center of gates A and A2 and a current velocity of 10.4 fps was measured 15 ft downstream from gate A in an area where riprap protection is proposed. Since these were the highest current velocities measured for normal gate operation conditions, the riprap protection on the bay side of the structure was designed for these conditions.

The riprap protection on the bay side for a 30-ft-wide section adjacent to the structure was designed based on a current velocity over the riprap of 12.1 fps. The current velocity exiting the structure was maximum, 12.1 fps, at the beginning of the protection and then decreased in the flood direction. The riprap should be designed for the maximum current velocity since movement immediately adjacent to the structure could jeopardize the entire bay side blanket. The D_{50} minimum size stone according to the Ishbash Equation for high turbulence areas, assuming a specific stone weight of 165 pounds per cubic foot and specific water weight of 64.3 pounds per cubic foot, is 2.0 ft for a velocity of 12.1 fps. The Ishbash Equation used to determine the D_{50}

minimum size stone in this analysis is shown in Hydraulic Design Chart 712-1 of the Corps of Engineers' Hydraulic Design Criteria. The Ishbash coefficient, C, in the equation was 0.86. A 2.0-ft-spherical stone weighs 691 lbs. The thickness for placement in the dry and subject to turbulent flows is 66 in. The gradation limits for a 66-in. thick blanket with a minimum W_{50} size stone of 691 lbs from EM 1110-2-1605 for riprap placed in the dry and subject to turbulent flow conditions are shown below.

Thichness=66 inches **Specific weight=165 pounds per cubic foot**		
Percent Lighter by Weight	**Limits of Stone Weight in Pounds**	
	Upper Limit	**Lower Limit**
100	4,259	1,740
50	1,262	852
15	631	266

Thickness for placement underwater should be increased 50 percent. This protection, designated Type 1 riprap, will be adequate for the first 30-ft-wide section on the bay side of the structure for the flow conditions shown in Figure 75.

Riprap protection is also needed for an additional 50 ft from the 30-ft-wide blanket adjacent to the structure on the bay side. This riprap protection was designed based on a bottom velocity of 8.5 fps. This velocity was determined from the measurements made on the bay side of gate A2 shown in Figure 75. The 9.8 fps current velocity was measured 15 ft away from the structure and the 5.4 fps current velocity was measured 65 ft further inland from the previous measurement. A linear interpolation between these two point measurements gives a current velocity of 8.5 fps 30 ft inland of the structure. The D_{50} minimum size stone according to the criteria above, and again assuming a specific stone weight of 165 pounds per cubic ft and specific water weight of 64.3 pounds per cubic ft, is 1.0 ft for a velocity of 8.5 fps. A spherical stone this size with a specific weight as stated above weighs 87 lbs. The thickness for placement in the dry and subject to turbulent flows is 33 inches. The gradation limits for a 33 in thick blanket with a minimum W_{50} size stone of 87 lbs from EM 1110-2-1605 for riprap placed in the dry and subject to turbulent flow conditions are shown below.

Thichness=33 inches Specific weight=165 pounds per cubic foot		
Percent Lighter by Weight	Limits of Stone Weight in Pounds	
	Upper Limit	Lower Limit
100	532	313
50	225	106
15	112	33

This protection, designated Type 2 riprap, will be adequate for protecting the area between 30 and 80 ft away from the structure on the bay side in the flood flow direction for the conditions shown in Figure 75.

The 80 feet of riprap protection on the bay side of the structure is sufficient due to the decrease in velocity in the flood direction as shown by the data in Figure 75. However, local scour should be expected at the termination of the riprap blanket. The riprap protection should be "keyed in" at the termination of the 80 ft blanket to prevent excessive movement. Examples of methods used to transition from the riprap to the unprotected bottom are shown in Figure 77. EM 1110-2-1605 indicates granular filters are recommended for riprap placement adjacent to structures. EM 1110-2-1901 presents guidance for this type filter. The top of the riprap should be placed one to two feet below the elevation of the gate sill. Side-slope riprap should be the same size as the invert riprap.

Ocean side riprap design for normal tainter gate operation. Bottom current velocities measured on the ocean side of the structure for normal tainter gate operations indicate the highest current velocity 50 ft oceanward of the structure occur with the miter gate closed, the tainter gates open 100 percent, with discharge conditions of 22,100 cfs. Current velocities up to 2.9 fps were measured 50 ft oceanward of the structure as shown in Figure 70. Flow approaching the structure accelerates, causing an increase in the current velocities. Since current velocity information is not available immediately adjacent to the structure on the ocean side, the current velocity measured in the center of the structure would give a conservative design. The highest current velocity measured in the center station occurred with the conditions shown in Figure 75 and was 12.1 fps. The riprap design for this current velocity is discussed above and consist of a D_{50} minimum size stone of 2.0 ft and a dry placement blanket thickness of 66 inches. This protection should extend from the structure oceanward a distance of 30 ft. This is a conservative design and it may be more practical to use smaller riprap such as Type 2 and grout the entire blanket to increase the stability.

Recommended riprap design for normal gate operations. The riprap plan recommended for the Saugus River structure based on the normal operating conditions shown in Figure 75 consist of a 66 inch thick blanket of riprap with a D_{50} minimum size stone of 2.0 ft extending 30 feet inland of the structure followed by a 33 inch thick blanket of riprap with a D_{50} minimum size stone of 1.0 ft. The riprap should be "keyed in" at the termination in a method similar to those shown in Figure 77. On the ocean side of the structure, protection is needed a distance of 30 ft from the oceanward lip of the sill. This protection should either be the Type 1 riprap or the Type 2 riprap with the entire blanket grouted. The recommended riprap design for normal gate operations for current velocities as shown in Figure 75 is shown in Figure 78.

Riprap design for gate malfunction. A riprap design to protect the structure from the severe flow conditions that might result if a gate malfunctioned during the closing procedure was also investigated. The conditions tested for this scenario are shown in Figure 41. The conditions tested with ocean water surface elevations equal to and less than 7.60 ft NGVD, bay water surface elevations equal to and less than 1.70 ft NGVD, and one tainter gate open and the remaining gates closed are representative of those conditions which would exist with near maximum astronomic tides, low freshwater inflows into the bay, and a malfunction in gate operation. The condition with the ocean water surface elevation at 11.55 ft NGVD and the bay water surface elevation at 6.7 ft NGVD represents near maximum ocean and bay water surface elevations difference. Gates A and C are located adjacent to and on the south and north side of the navigation channel, respectively.

Resulting water surface elevations and current velocities measured 2 ft off the bottom for the test conditions shown in Table 41 are shown in Tables 42-47. Current velocities greater than 20 fps exiting the structure were measured for 4 of the 6 conditions tested. Current velocities equal to or greater than 20 fps were measured 30 ft from the structure on the bay side for 3 of the 6 conditions tested.

Bay side riprap design for gate malfunction conditions. The highest current velocities measured on the bay side of the structure occurred with the ocean water surface elevation at 6.0 ft NGVD, bay side water surface elevation at -1.1 ft NGVD, with only gate C open. These measurements are shown in Table 45, and were obtained with a discharge of 12,773 cfs. Riprap requirements for the conditions shown in Table 45 are substantial. If the average current velocity over the 30-ft-wide area on the bay side of gate C is considered to be the average of the 6 bottom velocities measured in this area, then the average velocities over this area is 20.0 fps. Since a jet type flow exists for this condition, this method for determining average velocity is reasonable. The D_{50} minimum size stone according to the Ishbash Equation for high turbulence areas and assuming a specific stone weight of 165 pounds per cubic foot and specific water weight of 64.3 pounds per cubic foot, is 5.4 ft for a current velocity of 20.0 fps. A spherical stone this size weighs 13,600 lbs. Guidance for designing a graded riprap blanket with a D_{50} size this large is sparse. Usually, the blanket would consist of one or two layers of these large and gate operations shown in Table 41 are severe. More than 100 ft of large riprap would be necessary for protection on the bay side and additional current velocity information would be necessary to determine the amount and size for the conditions shown in Table 41.

5 Discussion of Test Results

The final design configuration (number and location of floodgates, and dredged approach areas) was determined during the course of the navigation study. Safe navigation in conjunction with cost effective and environmental sound design was the primary element in the selection of the final plan. The details of each plan investigated involving navigation safety will be adressed in the navigation study report. Plan 2C+7 was recommended as the best plan in respect to navigation safety. This plan meets the other objectives and was selected as the final plan. *

This report will not address individual data generated by the many plans investigated thoughout the physical model investigation. However, all hydraulic data collected during the course of the physical model study are presented in the form of tables and figures. The discussion of model tests results, for the purpose of this report, will concentrate on Plan 2C+7.

Plan 2C+7

Base spring tide conditions. Water surface elevations were influenced very little by the installation of Plan 2C+7. The difference in water surface elevations during flood flow from Station 3 (Headbay A) to Station 1 (Pines River) was 0.08 ft for Base conditions. With Plan 2C+7 installed, this difference increased to 0.15 ft, for an increase in head loss of 0.07 ft. The difference in water surface elevations for ebb flow conditions from Station 1 to Station 3 was 0.07 ft, and 0.10 ft with Plan 2C+7 in place, or an increase in head loss of 0.03 ft. The head loss across the structure (Range 2.43 to 2.44) during flood flow varied from a minimum of 0.00 ft at Stations A and B to a maximum of 0.08 ft at station C, D and F. Head loss during ebb flow varied from 0.00 ft at Station A to 0.05 ft at Station F. Head loss at other stations varied from 0.02 ft to 0.03 ft.

Current velocities were influenced to a greater degree than were water surface elevations. The largest increase in current velocity observed during plan flood flow conditions occurred at gate A (Stations 2.43A and 2.44A). At Station 2.43A depth averaged current velocity magnitude increased from 1.2 fps to 2.1 fps, and from 1.4 fps to 2.4 fps at Station 2.44A. In the

5 Discussion of Test Results

The final design configuration (number and location of floodgates, and dredged approach areas) was determined during the course of the navigation study. Safe navigation in conjunction with cost effective and environmental sound design was the primary element in the selection of the final plan. The details of each plan investigated involving navigation safety will be adressed in the navigation study report. Plan 2C+7 was recommended as the best plan in respect to navigation safety. This plan meets the other objectives and was selected as the final plan. *

This report will not address individual data generated by the many plans investigated thoughout the physical model investigation. However, all hydraulic data collected during the course of the physical model study are presented in the form of tables and figures. The discussion of model tests results, for the purpose of this report, will concentrate on Plan 2C+7.

Plan 2C+7

Base spring tide conditions. Water surface elevations were influenced very little by the installation of Plan 2C+7. The difference in water surface elevations during flood flow from Station 3 (Headbay A) to Station 1 (Pines River) was 0.08 ft for Base conditions. With Plan 2C+7 installed, this difference increased to 0.15 ft, for an increase in head loss of 0.07 ft. The difference in water surface elevations for ebb flow conditions from Station 1 to Station 3 was 0.07 ft, and 0.10 ft with Plan 2C+7 in place, or an increase in head loss of 0.03 ft. The head loss across the structure (Range 2.43 to 2.44) during flood flow varied from a minimum of 0.00 ft at Stations A and B to a maximum of 0.08 ft at station C, D and F. Head loss during ebb flow varied from 0.00 ft at Station A to 0.05 ft at Station F. Head loss at other stations varied from 0.02 ft to 0.03 ft.

Current velocities were influenced to a greater degree than were water surface elevations. The largest increase in current velocity observed during plan flood flow conditions occurred at gate A (Stations 2.43A and 2.44A). At Station 2.43A depth averaged current velocity magnitude increased from 1.2 fps to 2.1 fps, and from 1.4 fps to 2.4 fps at Station 2.44A. In the

1.0 ft Sea Level Rise Test

This test was conducted with Plan 2C+7 installed in the model. No Base test with conditions of a 1.0 ft sea level rise was conducted. Therefore, comparisons of water surface elevations and current velocities will be made against Plan 2C+7 spring tide conditions (Base test for this situation). Plan 2C+7, water surface elevation boundary conditions for the 1.0 ft rise in sea level, for both flood and ebb flow conditions were, as expected,.about 1.0 ft higher than Base Plan 2C+7. Differences between Plan 2C+7 and Base Plan 2C+7 water surface elevations for flood flow conditions were slightly greater than 1.0 ft, while the elevations occurring during ebb flow conditions were slightly less that 1.0 ft.

Current velocity observations on Ranges 2.43 and 2.44 made during Plan 2C+7 1.0 ft increase in sea level testing were not significently changed from those observed during the Base tests. Current velocities on Ranges 2.43 and 2.44 during flood flow condition were approximately 0.2 fps to 0.4 fps lower than observed for the base test. However, current velocity observations made during the ebb flow condition on these two ranges were generally 0.2 fps to 0.6 fps higher than observed for base conditions. This general trend was observed at other stations monitored during the testing.

Series I Emergency Gate Closure Tests

Miter gate open, all other gates closed. Bottom current velocities obtained for a flood discharge of 22,100 cfs and a head differential of 1.55 ft at Stations 2.43B, MSB, 2.44B, 2.44+2, 2.44+3, and 2.44+4 were 5.1 fps, 9.9 fps, 9.4 fps, 8.0 fps, 7.5 fps and 4.9 fps, respectively. Bottom velocity at Station 2.5A (located in the center of Points of Pines Yacht Club) was 1.1 fps. Current velocities obtained with the model operating with a discharge of 11,050 cfs and a head differential of 0.37 ft were; 2.0 fps at Station 2.43B,; 4.3 fps at Station MSB; 3.0 fps at Station 2.44B; 3.1 fps at Station 2.44B+1; 2.7 fps at Station 2.44B+2; 3.5 fps at Station 2.44B+3; and 2.9 fps at Station 2.44B+4. Current velocities measured with a discharge of 5,525 cfs and a head differential of 0.10 ft at all stations were below 2.5 fps.

All gates closed except gate A/or C. This series of tests was conducted with discharges of 11,050 cfs, 5,525 cfs and 3,315 cfs, with head differentials of 2.23 ft, 0.58 ft, and 1.17 ft, respectively. Bottom depth current velocity profiles for each gate (A or C) were very similar. Current velocities observed with gate C open were slightly higher than observations made with gate A open for each of the discharges tested. Maximum bottom current velocities were observed at Stations 2.44A/2.44C, respectively. Bottom current velocities obtained with a discharge of 11,050 cfs at Station 2.44A (Gate A open) was 10.9 fps, and 12.2 fps at Station 2.44C (gate C open). Bottom current velocities at Stations 2.44A+4 and 2.44C+4 (approximately 250 ft landward of the structure centerline) had decreased to 3.5 fps and 7.2 fps for the 11,050 cfs tests, with tests for gate A and C respectively. With a discharge of 5,525 cfs the maximum current velocity of 5.1 fps occurred at Station 2.44A+1 for Gate A tests and at Station 2.44C (5.7 fps) for gate C tests. The largest bottom current velocities for a discharge of 3,315 cfs were observed at Stations 2.44A and 2.44C, respectively for tests conducted with Gate A or Gate C open, and were 3.4 fps at each station.

Series II Emergency Gate Closure Tests

This series of tests simulated the most adverse conditions during a gate closure procedure. Tests were conducted with either gate A or C open, all other gates were closed. During this test series, discharge was varied (individual tests) from a maximum of 16,560 cfs to a minimum of 9,200 cfs (gate C closed 50 percent). Ocean water surface elevations varied from +11.55 ft NGVD to +5.98 ft NGVD. Head differentials (difference between Headbay A and Pines River) varied from a maximum of 7.80 ft to 4.85 ft. Generally, maximum bottom current velocities of approximately 20 fps were measured. Maximum current velocities for each test were generally observed at Stations 5 or 6, located at the centerline of the structure, or 50 feet landward of the structure centerline. The largest bottom current velocity observed (27.2 fps at Station C5B) was with gate C closed 50 percent. This condition simulated a gate closure failure with the discharge at 9,200 cfs, an ocean water surface elevation of +6.4 ft NGVD, and a bay elevation of -1.40 ft NGVD (head differential of 7.80 ft.

Tainter Floodgate Closure Tests (Normal Operation)

The first test was conducted with all gates open, including the miter gate, with a discharge of 22,100 cfs, and a head differential of 0.15 ft. This was the same as Base Spring Tide conditions. Maximum bottom current velocities were generally observed at the center of the structure (Range MS). A maximum current velocity of 2.8 fps was observed on Range MS at Station B. Current velocity magnitude generally decreased both north and south of gate C. The maximum bottom current velocity observed at Station A2 (southernmost station) was 2.2 fps, while at the northernmost station (Station G) a velocity of 1.9 fps was observed. The average current velocity on Range MS, 2.44, and 2.44+1 were 2.5 fps, 2.0 fps , and 2.0 fps, respectively.

The second test was conducted with the miter gate closed, a discharge of 22,100 cfs, and a head differential of 0.20 ft. As in the above test, maximum bottom current velocities occurred at stations located on the structure centerline (Range MS). Current velocities of 3.8 fps were observed at Station D and E. Bottom current velocities across Range MS averaged about 3.4 fps. The average bottom current velocity on Range 2.44 was about 3.2 fps, and on Range 2.44+1 the average was 2.3 fps. Flow behind the closed miter gate (Station B) was reversed due to a large eddy formed by the flow through the gates on either side. This flow reversal was common to all tests conducted with the miter gate closed. A maximum reversed current velocity with a magnitude of 1.5 fps was observed with this condition at Range 2.44+2, Station B.

The third test was conducted with the miter gate closed, and all tainter gates closed 25 percent of total closure. A discharge of 22,100 cfs reproduced a head differential between Headbay A and Pines River of 0.40 ft. Again, fastest currents were observed on Range MS. Maximum bottom current velocities having a magnitude of 5.1 fps was observed at Station F on Range MS, and at Station D Range 2.44. However, average current velocity on Range MS was 4.8 fps, as compared to an average of 4.1 fps on Range 2.44. The average current speed on Range 2.44+1 was 2.9 fps. Behind the miter gate a reversed current with a magnitude of 1.7 fps was observed.

The fourth test was similar to the third (gates closed 25 percent). The difference was that the discharge was reduced to 11,050 cfs. This condition resulted in a head differential of 0.15 ft. Maximum current velocities of 2.6 fps were observed on Range MS, at Stations C and D. The average velocity on Range MS was 2.4 fps, 2.1 fps on Range 2.44 , and 1.5 fps on Range 2.44+1. A maximum reversed current behind the miter gate having a magnitude of 1.1 fps was observed.

The fifth test was conducted with a discharge of 22,100 cfs. The miter gate was closed and all tainter gates were closed 50 percent of total closure. This condition resulted in a head differential between Headbay A and Pines River of 1.28 ft. The maximum current velocity of 9.6 fps was observed on Range 2.44 Station E. Although the maximum single point velocity was observed at this location, the majority of current values on Range MS was greater than those on Range 2.44. The average current velocity on Range 2.44 was 8.2 fps as compared to an average of 9.2 fps on range MS. The maximum reversed current of 2.6 fps behind the miter gate was observed on Range 2.44+3.

The sixth test was similar to the fifth, except that discharge had been reduced to 11,050 cfs. Tainter gates remained at 50 percent closure. This condition resulted in a head differential of 0.31 ft. The maximum current velocity of 4.7 fps was observed on Range MS at Station A. However, flow was fairly well distributed over the entire length of the structure, as the minimum current value was 4.5 fps. Currents on both Ranges MS and 2.44 were almost equal, as the average on Range MS was 4.6 fps and 4.5 fps on Range 2.44. The average current velocity on Range 2.44+1 was 2.6 fps. The maximum reversed flow behind the miter gate had a velocity of 1.5 fps and was observed on Range 2.44+2.

The seventh test was conducted with the tainter gates closed 75 percent of total closure, and with a discharge of 11,050 cfs. The miter gate was closed. This condition reproduced a head differential of 2.15 ft. Maximum current velocities of 12.1 fps was observed on Range MS at Stations A2 and A. Current velocities on Ranges MS averaged 10.8 fps, 8.1 fps on Range 2.44 and

3.4 fps on Range 2.44+1. Flow was fairly well distributed across the entire structure. Behind the miter gate a reversed current velocity with a value of 2.7 fps was observed on Range 2.44+3.

The eighth test was conducted with a discharge of 5,525 cfs. The position of the gates remained the same as in the seventh test. This condition resulted in a head differential of 0.50 ft. The maximum current velocity was again observed on Range MS Station A, and had a value of 6.1 fps. The average current velocity on Range MS was 5.7 fps, indicating a uniform distribution of flow through the entire structure. The average velocity on Range 2.44 was 4.1 fps and 2.0 fps on Range 2.44+1. The reversed flow behind the miter gate had a maximum current velocity value of 1.5 fps and was observed on Range 2.44+3.

Table 11

Existing Spring Tide Conditions

Current Velocities

sample

		Flood Direction		Ebb Direction				Flood Direction		Ebb Direction	
Sta	Depth	Vel fps	Dir Deg	Vel fps	Dir Deg	Sta	Depth	Vel fps	Dir Deg	Vel fps	Dir Deg
2.2A	SUR	1.0	300	1.6	110	2.43J	SUR	1.8	250	1.5	100
	MID	1.0	300	1.8	120		MID	–	–	1.5	100
	BOT	1.7	300	1.4	120		BOT	1.3	250	1.4	100
2.2B	SUR	0.7	290	0.9	90	2.43K	SUR	1.9	240	1.2	100
	MID	0.7	290	1.0	90		MID	–	–	1.2	100
	BOT	0.4	290	0.9	90		BOT	1.7	240	1.1	100
2.3A	SUR	1.4	280	2.3	110	2.44A	SUR	1.9	270	1.9	100
	MID	1.3	280	1.9	110		MID	1.8	270	1.7	100
	BOT	1.1	270	1.5	110		BOT	1.8	270	1.5	100
2.3B	SUR	0.2	250	–0.2	220	2.44B	SUR	2.3	270	2.3	100
	MID	0.2	250	0.2	60		MID	2.2	270	2.2	100
	BOT	0.2	250	0.2	60		BOT	1.7	270	1.9	100
2.43A	SUR	2.0	270	1.8	90	2.44C	SUR	2.5	270	2.8	110
	MID	2.0	270	1.5	90		MID	2.5	270	2.6	110
	BOT	1.7	270	1.3	90		BOT	2.5	270	2.4	110
2.43B	SUR	2.0	260	2.1	110	2.44D	SUR	2.1	260	2.6	100
	MID	2.0	270	2.0	110		MID	2.0	260	2.2	100
	BOT	1.7	270	1.5	110		BOT	1.8	260	1.8	100
2.43C	SUR	2.3	270	2.1	100	2.44E	SUR	2.0	260	2.0	100
	MID	2.0	270	2.0	100		MID	1.7	260	1.7	100
	BOT	1.8	260	1.2	90		BOT	1.6	260	1.4	100
2.43D	SUR	2.4	270	2.5	110	2.44F	SUR	–	–	–	–
	MID	2.3	270	2.4	110		MID	–	–	–	–
	BOT	2.0	270	1.9	120		BOT	–	–	–	–
2.43E	SUR	2.5	270	2.5	110	2.44G	SUR	2.0	260	1.4	90
	MID	2.3	270	2.3	120		MID	1.9	260	1.4	90
	BOT	2.0	270	1.9	120		BOT	1.9	260	1.1	90
2.43F	SUR	2.0	260	1.3	100	2.44H	SUR	2.3	250	1.5	100
	MID	1.9	260	1.3	100		MID	2.0	250	1.3	100
	BOT	1.7	260	1.2	100		BOT	1.9	250	1.2	100
2.43G	SUR	2.1	260	1.8	100	2.44I	SUR	2.3	250	1.4	90
	MID	1.8	260	1.8	100		MID	1.8	250	1.5	90
	BOT	1.5	260	1.7	100		BOT	1.7	250	1.3	100
2.43H	SUR	1.8	260	1.8	110	2.44J	SUR	2.4	250	1.6	90
	MID	1.8	260	1.7	110		MID	2.0	250	1.5	90
	BOT	1.7	260	1.6	110		BOT	1.6	250	1.3	90

(Continued)

Table 36

Base 1.0 ft Sea Level Rise Conditions

Current Velocities, Plan 2C+7

Sta	Depth	Flood Direction Vel fps	Flood Direction Dir Deg	Ebb Direction Vel fps	Ebb Direction Dir Deg	Sta	Depth	Flood Direction Vel fps	Flood Direction Dir Deg	Ebb Direction Vel fps	Ebb Direction Dir Deg
2.2A	SUR	0.9	300	2.8	120	2.43J	SUR	–	–	–	–
	MID	0.9	300	2.5	120		MID	–	–	–	–
	BOT	0.9	300	2.0	120		BOT	–	–	–	–
2.2B	SUR	0.4	290	1.3	90	2.43K	SUR	0.9	200	0.2	200
	MID	0.3	290	1.2	90		MID	0.8	200	0.2	200
	BOT	0.3	290	1.1	90		BOT	0.7	200	0.2	200
2.3A	SUR	1.0	280	2.9	100	2.44A	SUR	2.3	280	2.6	110
	MID	1.0	280	2.6	100		MID	2.1	280	2.4	110
	BOT	0.8	280	2.2	100		BOT	1.7	280	2.1	110
2.3B	SUR	0.2	220	-0.2	200	2.44B	SUR	2.3	280	3.0	100
	MID	0.2	220	0.2	120		MID	2.0	280	2.8	100
	BOT	0.2	220	0.2	120		BOT	1.6	280	2.5	100
2.43A	SUR	1.9	280	2.9	90	2.44C	SUR	2.6	280	3.3	100
	MID	1.7	280	2.9	90		MID	2.3	280	3.1	100
	BOT	1.5	280	2.5	90		BOT	2.0	280	2.8	100
2.43B	SUR	1.8	270	3.3	100	2.44D	SUR	2.5	280	3.1	100
	MID	1.6	270	3.3	100		MID	2.3	280	3.0	100
	BOT	1.3	270	2.8	100		BOT	1.9	280	2.6	100
2.43C	SUR	1.6	270	3.3	100	2.44E	SUR	2.5	280	2.9	100
	MID	1.3	270	3.3	100		MID	2.3	280	2.6	100
	BOT	1.1	270	2.9	100		BOT	2.0	280	2.4	100
2.43D	SUR	1.7	260	3.3	100	2.44F	SUR	1.9	290	2.7	100
	MID	1.5	260	3.1	100		MID	2.2	290	2.4	100
	BOT	1.3	250	2.6	100		BOT	2.0	290	2.2	100
2.43E	SUR	1.6	250	3.0	100	2.44G	SUR	1.1	290	2.0	100
	MID	1.5	250	2.8	100		MID	0.3	290	1.7	100
	BOT	1.2	250	2.6	100		BOT	0.9	290	1.6	110
2.43F	SUR	1.6	240	2.9	100	2.44H	SUR	0.2	230	0.2	160
	MID	1.3	240	2.6	100		MID	0.2	230	0.2	160
	BOT	1.2	240	2.4	100		BOT	0.2	230	0.2	160
2.43G	SUR	1.3	230	2.0	100	2.44I	SUR	–	–	–	–
	MID	1.3	230	2.2	100		MID	–	–	–	–
	BOT	0.9	230	1.8	100		BOT	–	–	–	–
2.43H	SUR	1.2	210	0.3	100	2.44J	SUR	–	–	–	–
	MID	1.1	210	0.2	100		MID	–	–	–	–
	BOT	1.1	210	0.2	100		BOT	–	–	–	–

(Continued)

Sample

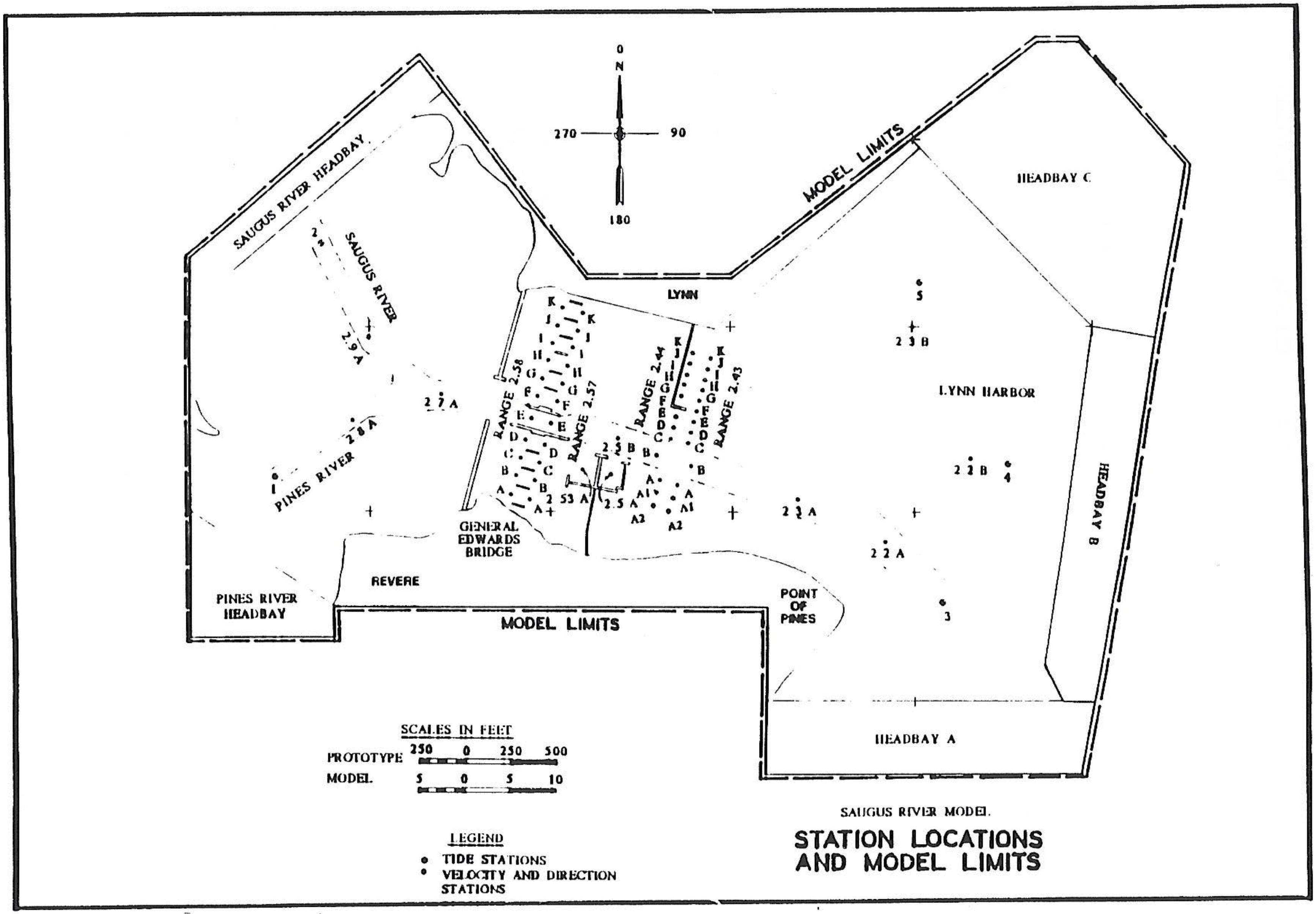

Figure 2. Model limits and station location map

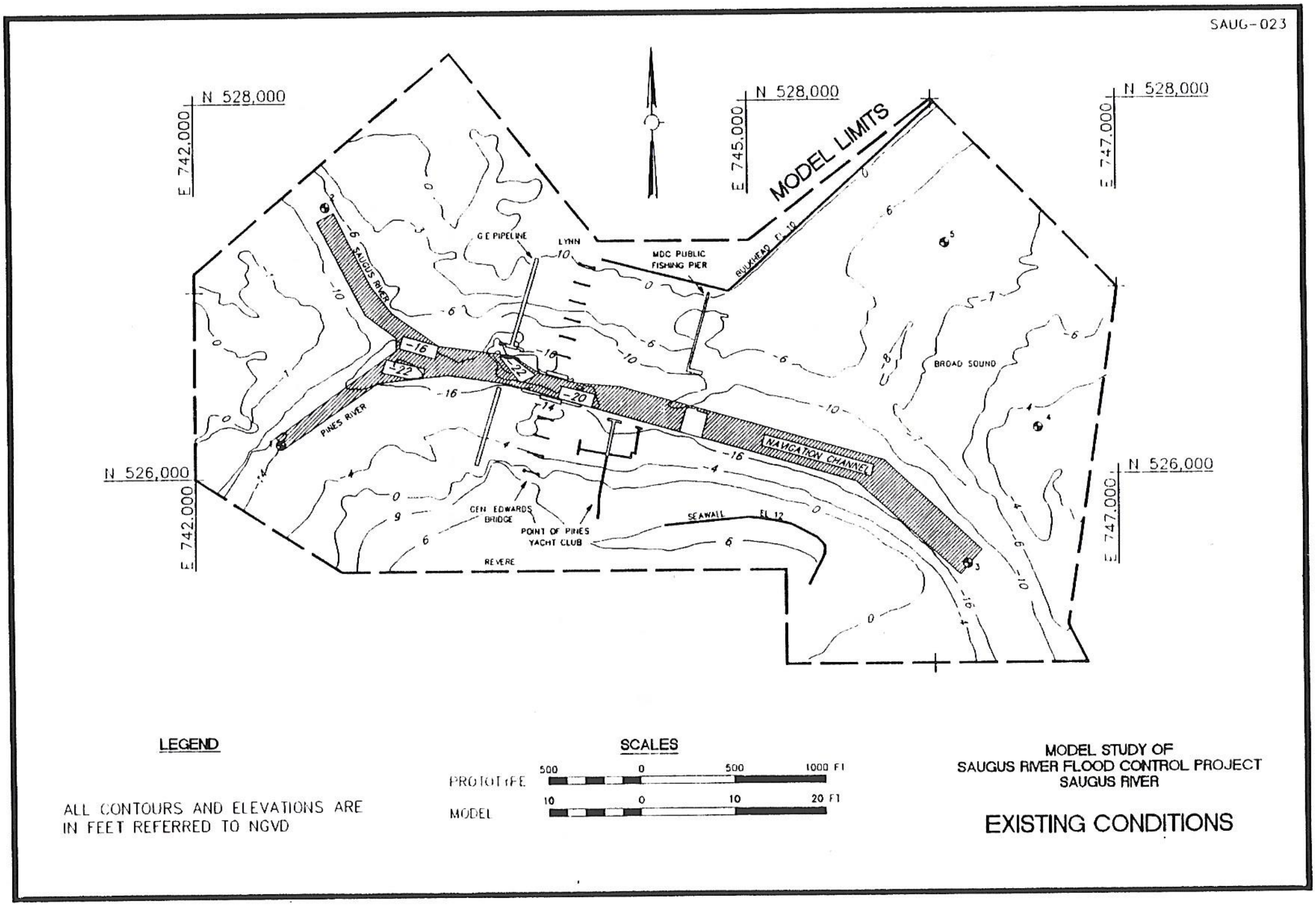

Figure 9. Existing conditions

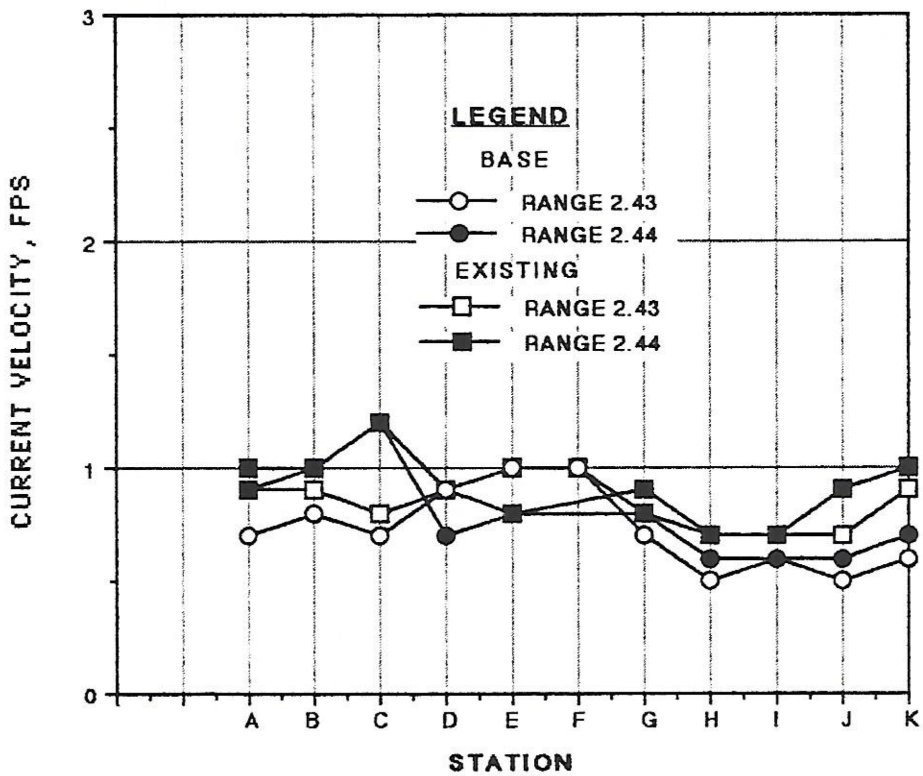

Figure 16. Current Velocities, Existing vers Base Neap Tide-Flood Direction

Sample

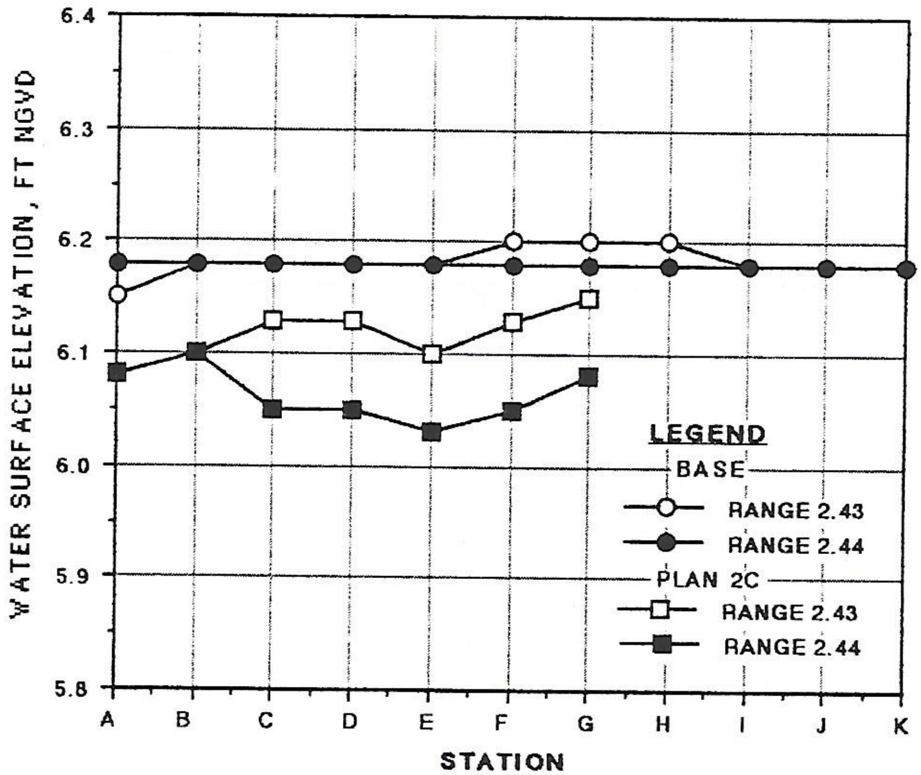

Figure 37. Water Surface Elevations, Plan 2C vers Base Spring Tide-Flood Direction

sample

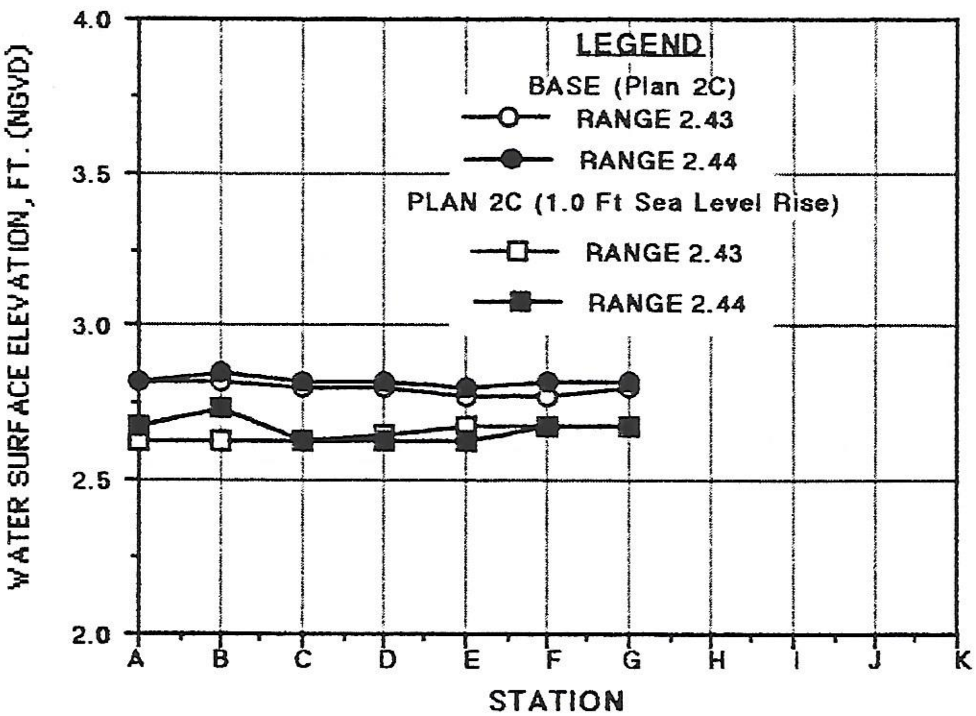

Figure 54. Water Surface Elevations, Plan 2C (1.0 Ft. Sea Level Rise) vers Base Plan 2C, Spring Tide-Ebb Direction

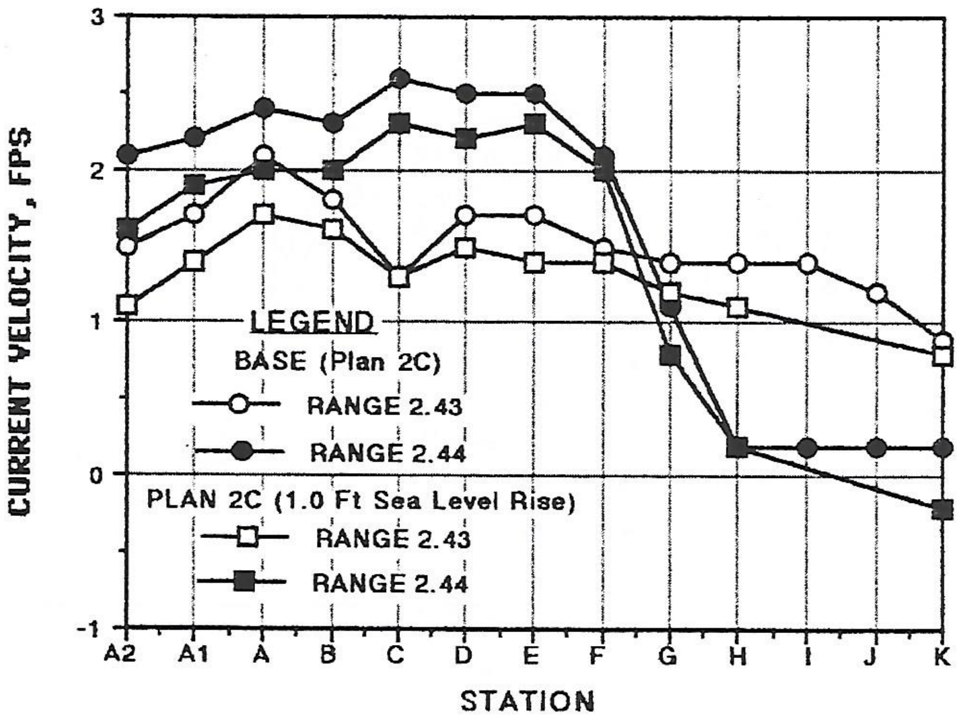

Figure 55. Current Velocities, Plan 2C (1.0 Ft Sea Level Rise) vers Base Plan 2C, Spring Tide-Flood Direction

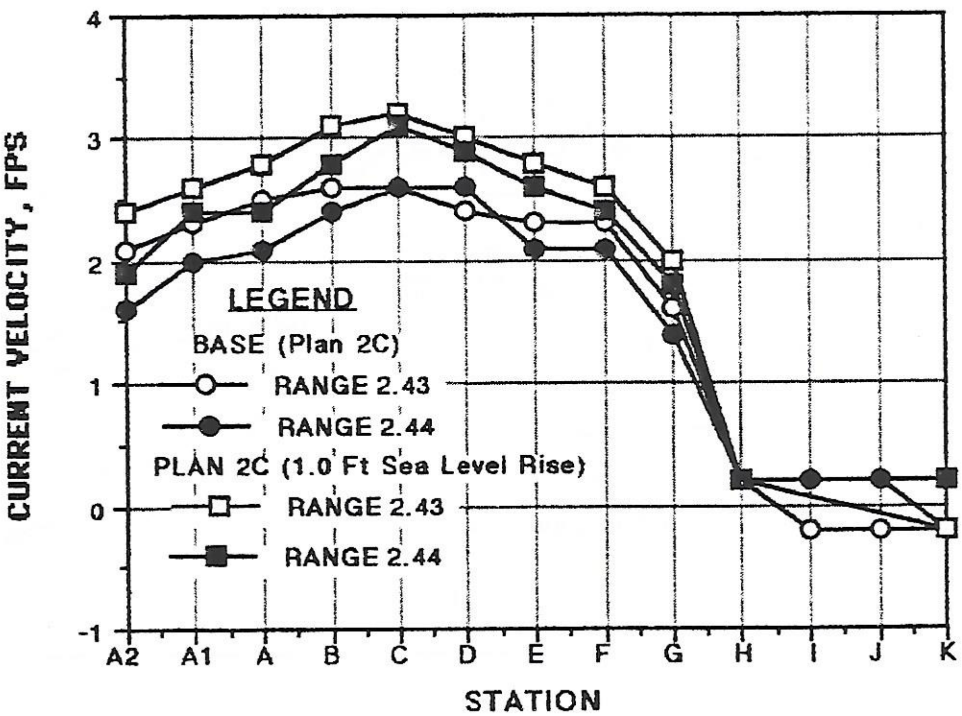

Figure 56. Current Velocities, Plan 2C (1.0 Ft Sea Level Rise) vers Base Plan 2C, Spring Tide-Ebb Direction

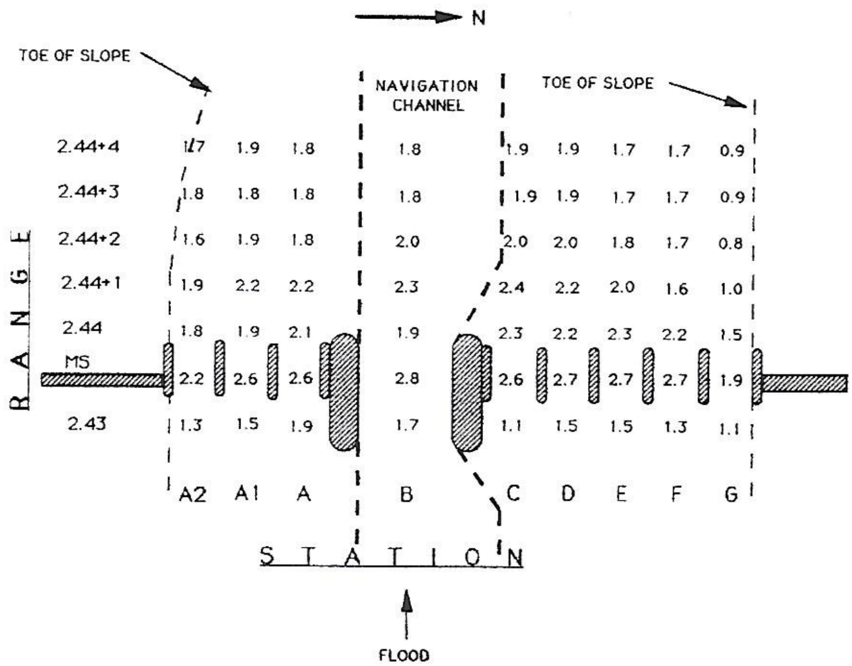

FIGURE 69. BOTTOM CURRENT VELOCITIES IN FPS
DISCHARGE - 22,100 CFS
MITER AND ALL TAINTER GATES OPEN 100 PERCENT

Sample