UFL/COEL-92/013

Ponce De Leon Channel Shoaling and Erosion Studies
Punta Gorda, Florida

by

Hsiang Wang
Jung L. Lee
Lihwa Lin

September, 1992

Prepared for

City of Punta Gorda, Florida

REPORT DOCUMENTATION PAGE

1. Report No.	2.	3. Recipient's Accession No.
4. Title and Subtitle Ponce De Leon Channel Shoaling and Erosion Studies Punta Gorda, Florida		5. Report Date September 18, 1992
		6.
7. Author(s) Hsiang Wang, Jung L. Lee, Lihwa Lin		8. Performing Organization Report No. UFL/COEL-92/013
9. Performing Organization Name and Address Coastal and Oceanographic Engineering Department University of Florida 336 Weil Hall Gainesville, Fl.32611		10. Project/Task/Work Unit No.
		11. Contract or Grant No.
12. Sponsoring Organization Name and Address Public Works Department, City of Punta Gorda 900 West Henry Street Punta Gorda, Fl.33950		13. Type of Report Final
		14.

15. Supplementary Notes

16. Abstract

The shoaling and bank erosion at Ponce de Leon channel, Punta Gorda, Florida, have caused concern of harming the mangrove community along the channel. Three factors were identified that could contribute to the bank erosion of Ponce channel. They are tidal induced current, wind waves penetrating from the Charlotte Harbor, and wakes caused by boat traffic. According to the field experiments and numerical modeling studies, it was determined that the combined wind wave and tidal current force is the major cause to the bank erosion. Wind wave appears to play a more important role because of its dynamic nature. For the present cross-sectional channel condition the tidal current alone is only a moderate erosional force.

It was also found that reopening the barge canal would cut the tidal current strength in the Ponce channel by a half. This current reduction would reduce but not eliminate bank erosion in the lower reach as the wind-wave induced force will remain to be an important erosional factor.

A number of remedial alternatives were given in the report. The most direct method is to provide bank protection. The extent of the protection depends upon the extent of wind wave penetration which could be as deep as 150 m into the channel under the present channel entrance condition.

17. Originator's Key Words	18. Availability Statement
Boat wake Erosion Tidal current Wave force	

19. U. S. Security Classif. of the Report	20. U. S. Security Classif. of This Page	21. No. of Pages	22. Price
Unclassified	Unclassified	48	

PREFACE

This report presents results of field measurements and numerical modeling of the Ponce Channel and the associated canal network for channel shoaling and bank erosion evaluations. The study identifies major forces responsible for channel shoaling and bank erosion so as to determine proper remedial measures.

The study in this report is sponsored by the City of Punta Gorda. The study and the report were prepared by the Department of Coastal and Oceanographic Engineering, University of Florida.

Special appreciation is due to Mr. Thomas M. Wilcox, former deputy director of Public Works, and Mr. Mark Gronceski, former city environmental engineer, for their preparation and technical assistance of this research project in the initial stage. Other personnel at City Public Works Department including Mr. James Stillwell, Mr. Richard Bensen and Ms. Diana Lester also provided their support at various stages of the study. Appreciation is also due to Dr. G. Miao and Mr. T. Kim in the Coastal Engineering Department, University of Florida, for their participation in the field experiments.

Contents

1 **Introduction** 1

2 **Field Investigation** 1

3 **Results from Field Measurements** 3

3.1 Tide Record 3

3.2 Current Measurement 7

3.3 Channel Bathymetric Survey 7

3.4 Flow Pattern Measurement 14

3.5 Sand Tracer Study 14

3.6 Boat Wake Study 14

4 **Wind Wave Estimation** 20

5 **Bank Erosion Assessment** 20

6 **Effects of Barge Canal on Tidal Hydraulics** 25

6.1 Existing Condition 25

6.2 Effect of Reopening the Barge Channel 30

7 **Conclusions and Recommendations** 30

7.1 Conclusions 30

7.2 Recommendations 35

References 38

A Ship Wave Prediction Model **39**

A.1 Prediction of Maximum Wake Height 39

A.2 Prediction of Maximum Wake-induced Velocity 41

List of Figures

1 Ponce de Leon Inlet and the associated canal network. 2

2 Locations of tide gages and current measurement stations. 4

3 Tidal histories measured from August 2nd to 23rd. 5

4 Tidal histories measured during August 3rd. 6

5 Current measured in 8/20/91 (flood). 8

6 Current measured in 8/20/91 (ebb). 9

7 Current measured in 8/21/91 (8:00-10:30). 10

8 Current measured in 8/21/91 (10:30-12:00). 11

9 Depth-averaged currents measured at all six stations. 12

10 Measured bottom bathymetries. 13

11 Flood flow pattern measured from drogue study. 15

12 Ebb flow pattern measured from drogue study. 16

13 Results of sand tracer study for N1, N2 and N3 locations. 17

14 Results of sand tracer study for S1, S2 and S3 locations. 18

15 Measured boat wake heights and wake-induced velocities. 19

16 Flow-induced shear stress vs. mean velocity. 22

17 Excess stress due to combined tide-wave effect for various wind speed. 24

18 Time-averaged flood flow pattern under existing condition. 26

19 Time-averaged ebb flow pattern under existing condition. 27

20 Current vectors of average flood flow under existing condition. . . . 28

21 Current vectors of average ebb flow under existing condition. 29

22 Time-averaged flood flow pattern under reopening of barge canal. . 31

23 Current vectors of average flood flow under reopening of barge canal. 32

24 Time-averaged ebb flow pattern under reopening of barge canal. . . 33

25 Current vectors of average ebb flow under reopening of barge canal. 34

26 A schematic of possible layout of jetty configurations. 37

A.1 Comparison of predicted and measured maximum wake heights. . . 40

A.2 Comparison of predicted and measured maximum wake velocities. . 41

List of Tables

1 Measured wake height and bank-normal velocity vs. boat speed. . . 20

2 Prediction of wave-induced along-bank velocity. 21

3 Boat traffic statistics at Ponce de Leon channel of February 91. . . 23

PONCE DE LEON CHANNEL SHOALING AND EROSION STUDIES
PUNTA GORDA, FLORIDA

1 Introduction

Ponce de Leon channel is one of the major boat accesses for the Punta Gorda community. The channel has been experiencing shoaling and bank erosion. This causes concern of harming the mangrove community along the channel. The purpose of this study is to determine the current and flow conditions along the channel so as to identify the forces of bank erosion as well as the effect of bank erosion on channel shoaling. Hopefully, the findings will lead to proper remedial measures.

For a tidal channel such as the Ponce channel, bank erosion could be caused by any of the following factors, or the combination of them:

a. Tidal induced current.

b. Forces induced by boat wakes.

c. Forces induced by wind waves propagating into the channel.

This study, therefore, was concentrated at estimating the effects due to the above cited factors and, also, the effect of reopening an old barge canal to reduce the tidal forces in the Ponce channel. The study was accomplished through field measurements augmented with a numerical model.

2 Field Investigation

Ponce de Leon Inlet serves as the major outlet for a large canal network shown in Figure 1. The only other boat access is on the northwest corner near Colony Pt. through an opening across the West Marion St. The entire Ponce de Leon channel cuts through a tidal marsh of mangrove growth and emptys into the Charlotte Harbor due west. On the southwest corner of the region there exists an abandoned barge canal which was blocked off at the two ends.

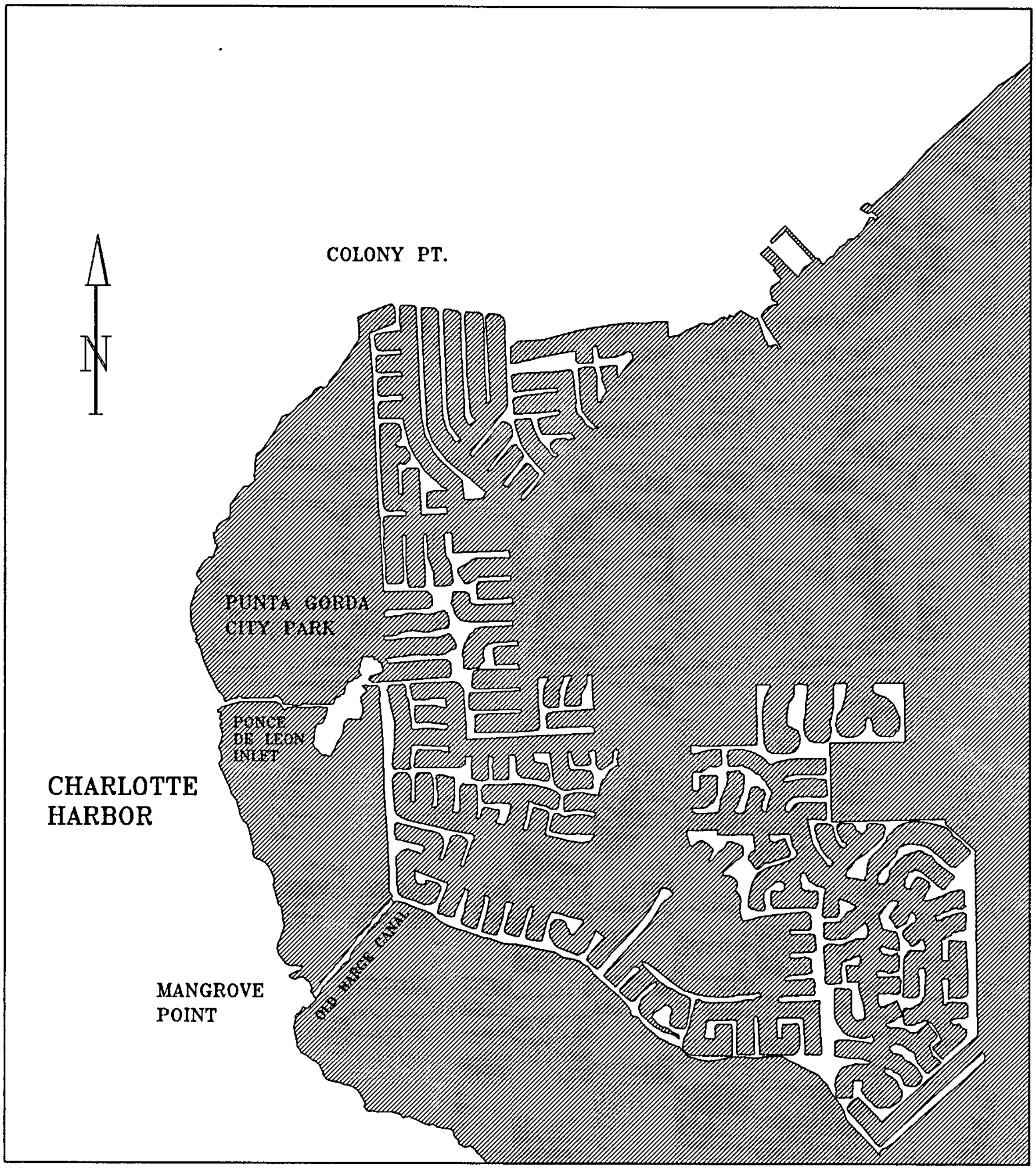

Figure 1: Ponce de Leon Inlet and the associated canal network.

Field measurements were carried out first in the summer of 1991. Three tide gages were installed for a period of three weeks from August 2nd to 23rd. One gage was located near the inlet entrance at Channel Marker 8 in the Charlotte Harbor, one near the junction of inlet and canal network at Channel Marker 18, and the third near the old barge canal at the southwest corner of the canal network. The purpose of the third gage is to evaluate the effect of reopening the barge canal. Current measurements were taken during August 19th and 20th at six cross sections for the hydrodynamic study. The locations of these tide gages and current measurement cross sections are marked in Figure 2. Sand samples were collected along the channel and near the mouth of the channel. Hydrographic serveys were conducted to determine the bathymetry in the channel and near inlet entrance area. Drogue studies were performed in the entrace area for both flood and ebb. Sand tracer studies were also conducted outside the entrance to determine the pattern of sediment movement.

In the summer of 1992, another field experiment was carried out within the channel near the inlet entrance to evaluate the boat wake effects. During the experiment, an 18-ft Mckee craft with a 90-Hp outboard motor was used to traverse the channel at various speeds and the so induced waves near the bank were recorded. During the same test period, a number of passages of boats of various sizes were through the channel at near idle speed. The waves caused by these boats were also recorded.

3 Results from Field Measurements

The results of performed field measurements from the summer of 1991 and the summer of 1992 are summarized in this section. These results will be used later for a rational evaluation of shoaling and bank erosion in the Ponce channel.

3.1 Tide Record

The measured water surface level fluctuations at the three tide gage locations were plotted in Figure 3. The surface variations were mainly tidal induced as there was no major storm or runoff event occurred during the measurement period. The differences of the measured surface fluctuations at the three gage locations were overall small. It is noticed that while the magnitudes of surface fluctuations are almost identical from the three gage locations, the tide at Channel Marker 18 (near channel-canal junction) actually lagged behind that at Channel 8 (inlet entrance) by about 6 minutes and a time lag of similar order also existed between the old barge canal gage location and Channel Marker 18. This can be seen in Figure 4

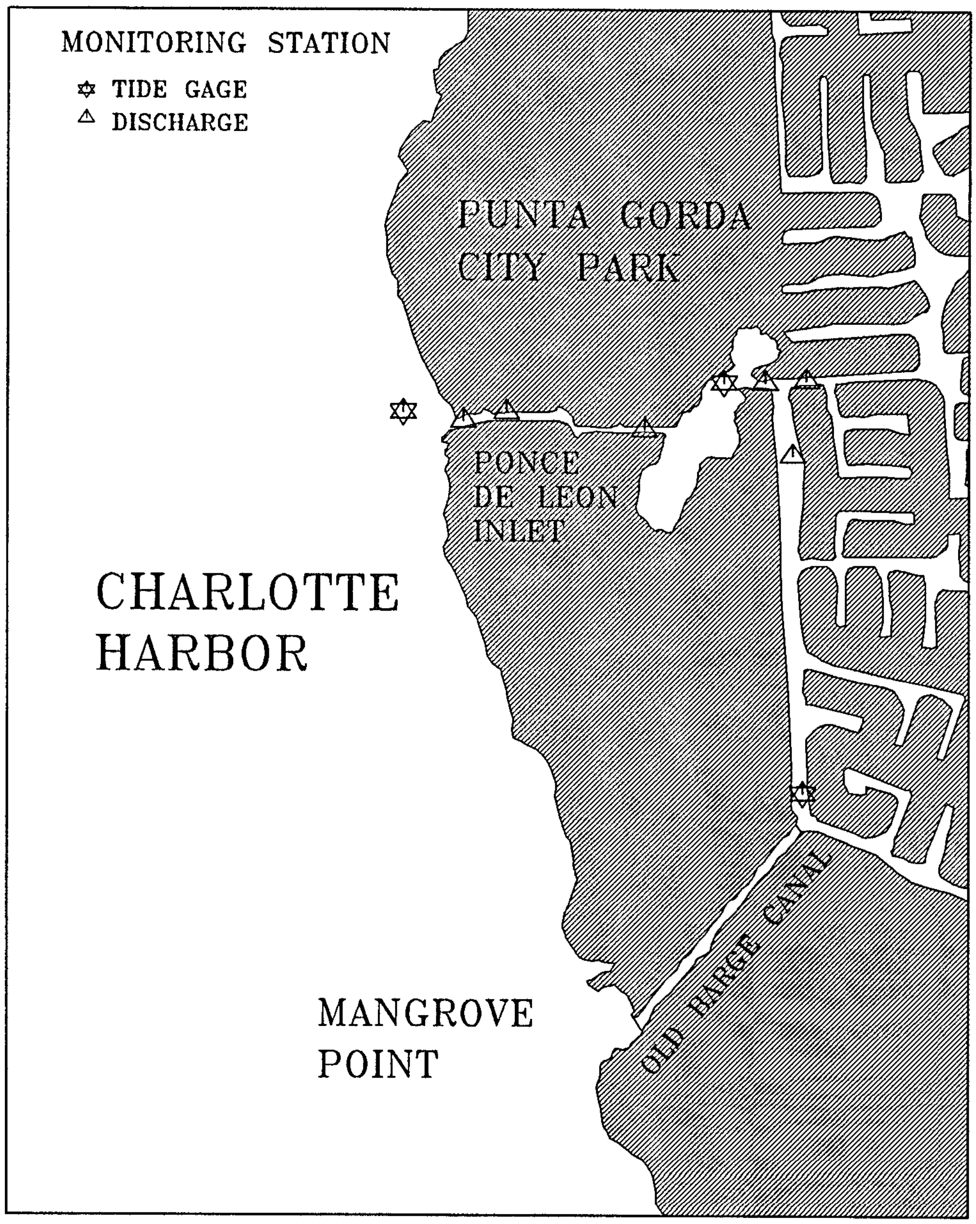

Figure 2: Locations of tide gages and current measurement stations.

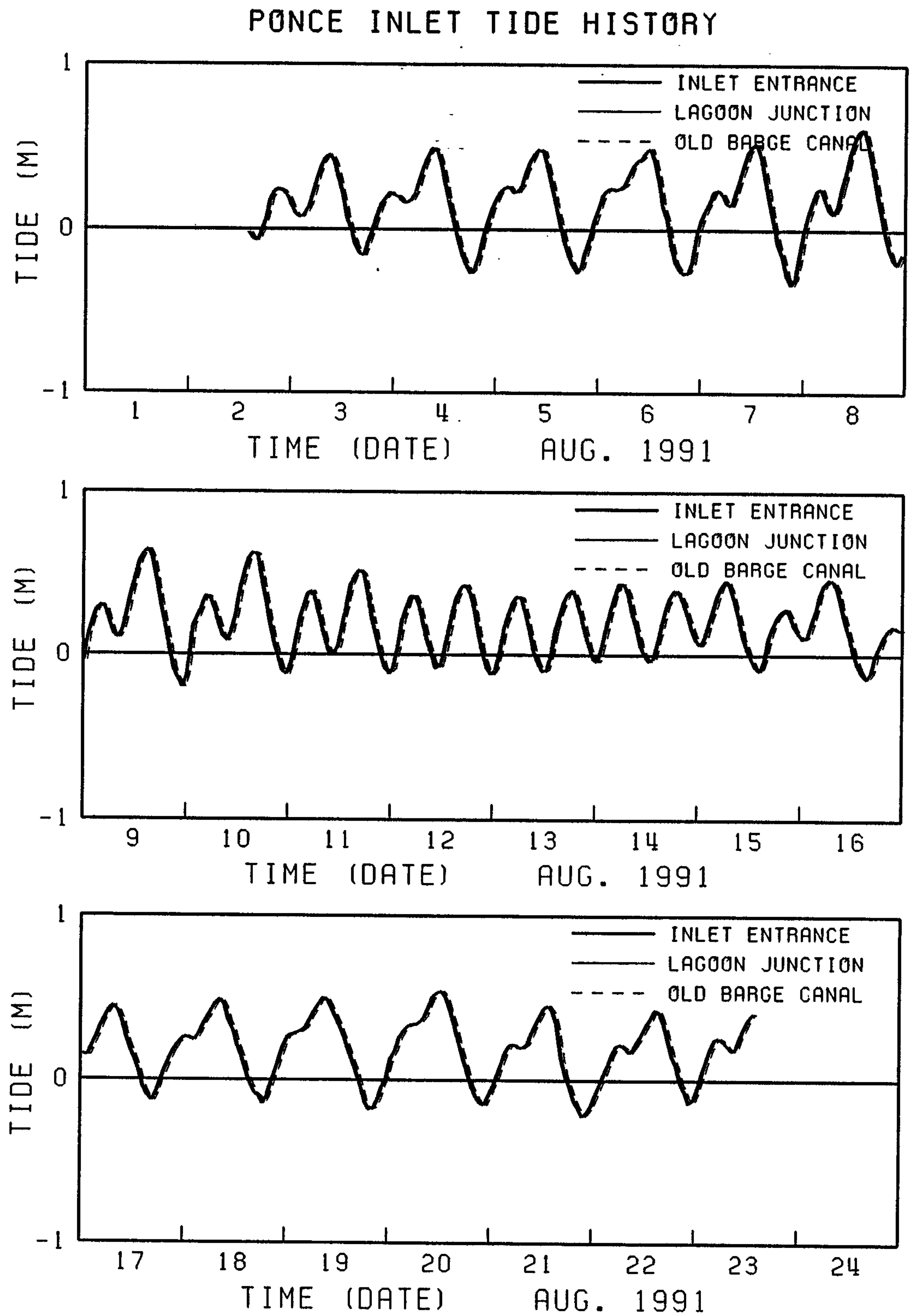

Figure 3: Tidal histories measured from August 2nd to 23rd.

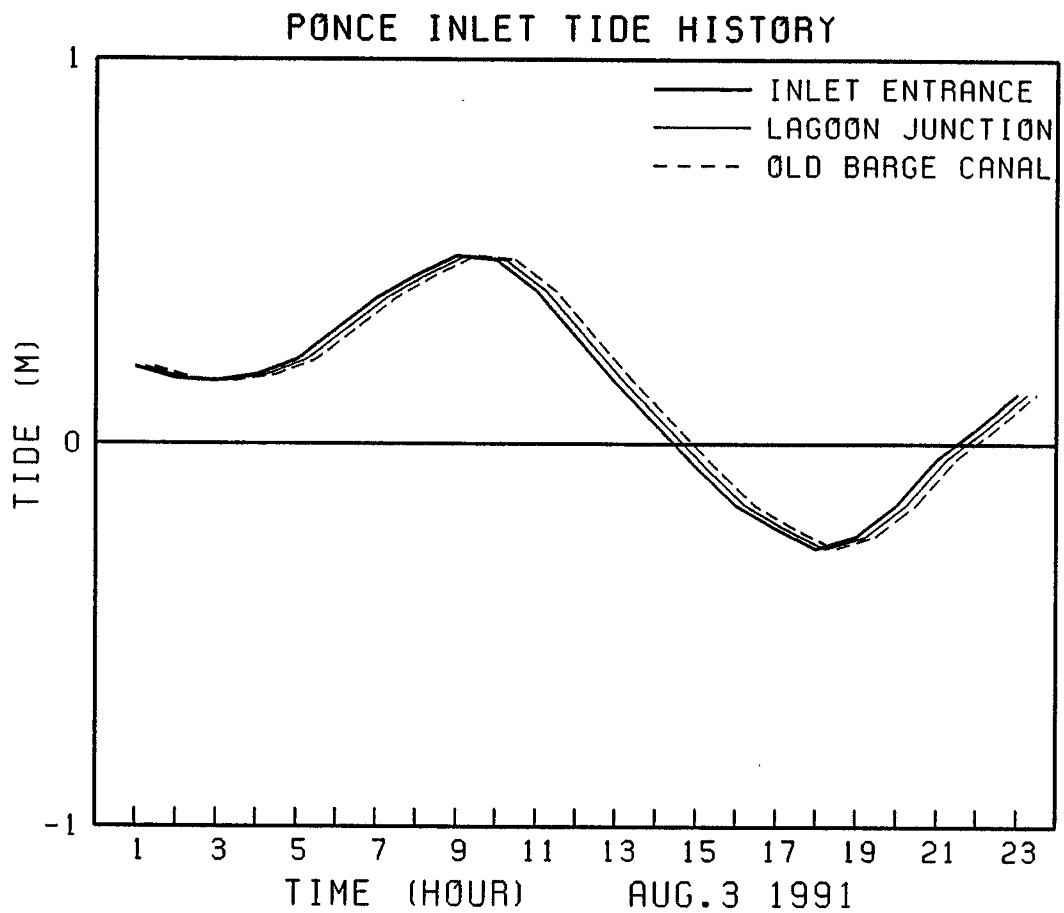

Figure 4: Tidal histories measured during August 3rd.

which shows the measured tides of August 3rd in an expanded time scale.

In the study region, the tidal action is a mixture of diurnal and semi-diurnal. It is seen in Figure 3 that the diurnal component dominated during the beginning and the end of the measurement period and the semi-diurnal component dominated in the middle. Like other inlets in this region, high water is usually associated with the diurnal tides. The spring tidal range is in the order of 1 m and the nip tidal range is around 0.6 m.

3.2 Current Measurement

Results of current measurement at the six cross stations are plotted in Figures 5 to 8. It should be noted here that the channel has no clearly defined bank as the channel extends into the surrounding wetland. The dotted lines in Figures 5 to 8 represent the visual limits of the vegetation along the channel. Stations 1 and 2 are in the lower reach of the channel near the inlet entrance; Station 3 is in the middle reach and Station 4 is at the end of the Ponce channel. Station 5 is in the canal network east of the channel and Station 6 is in the perimeter canal between the channel end and now-blocked barge canal. In the lower reach of the channel, particularly near the entrance, the tidal current is seen to be stronger along the south bank. The magnitudes of both flood and ebb are on the order of 0.65 m/s along the south bank and about 0.45 m/s along the north bank. In the middle reach (Station 3) where the cross section is the smallest, the current strength is about the same as the lower reach in the order of 0.6 m/s. The current strength reduces further at Stations 4, 5 and 6 because of the enlarged cross sections. Figure 9 shows the cross section averaged currents at all six stations. As far as the flow direction is concerned, flood water from Ponce Inlet and from Colony Pt. joint together at the end of the Ponce channel and flow southward into the perimeter canal between the end of Ponce channel and the barge canal, and then into the canal network. This flow direction reverses during ebb.

3.3 Channel Bathymetric Survey

Figure 10 plots the bottom bathymetries. The main channel is seen to be around 2.5 m deep in the lower reach and is skewed towards the north bank. In the middle and upper reach the channel is slightly deeper and the cross-section becomes almost symmetrical.

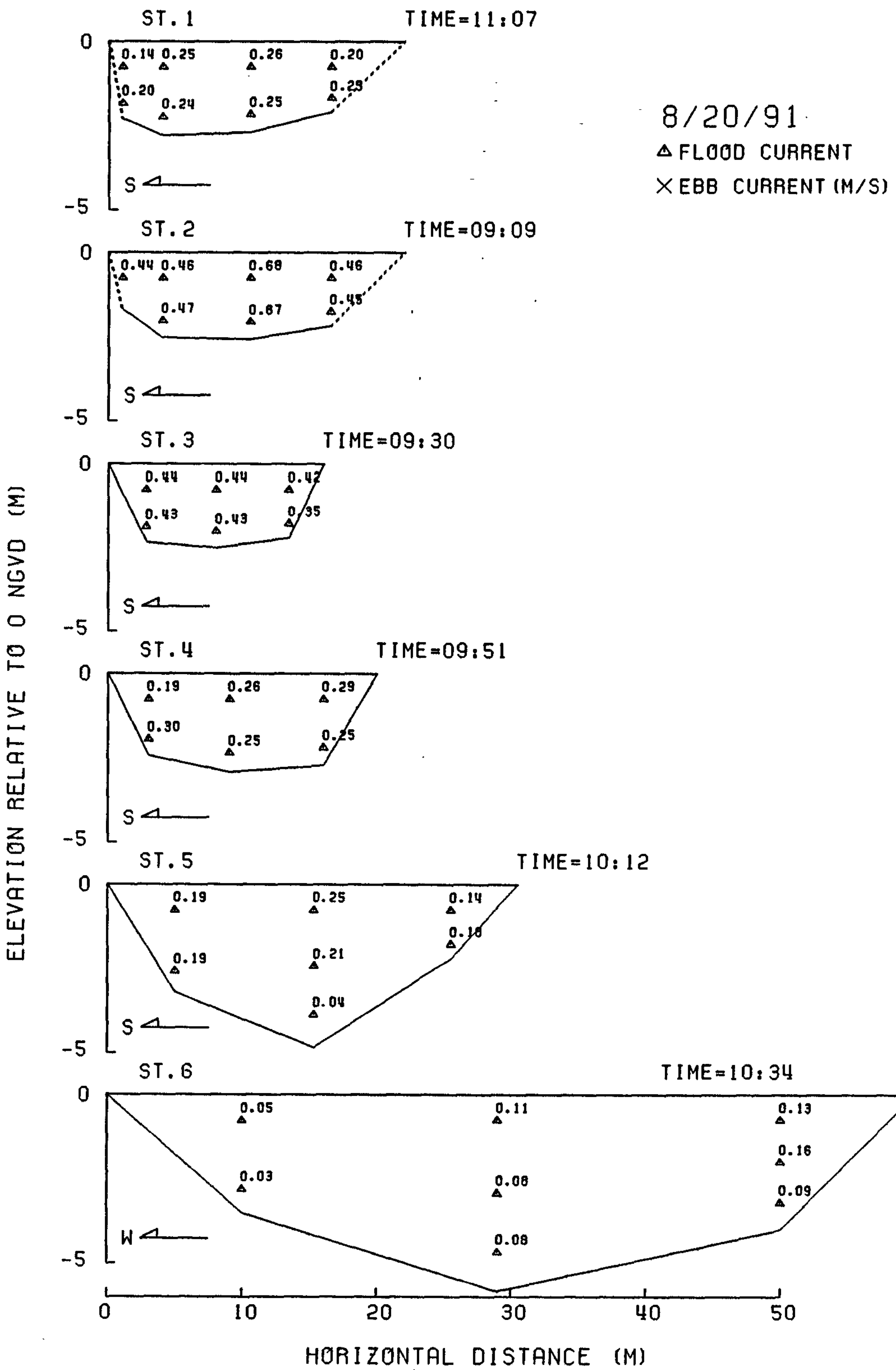

Figure 5: Current measured in 8/20/91 (flood).

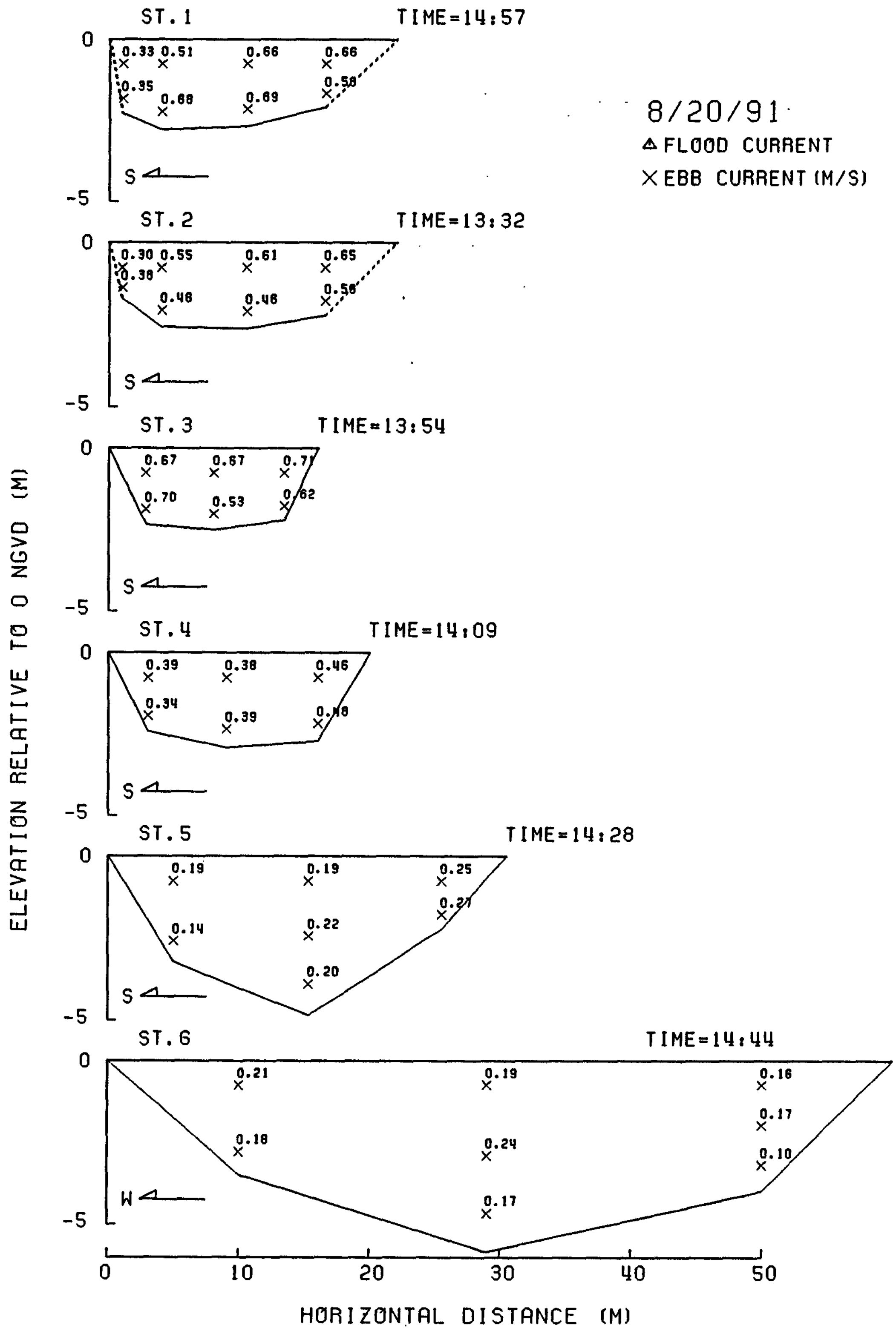

Figure 6: Current measured in 8/20/91 (ebb).

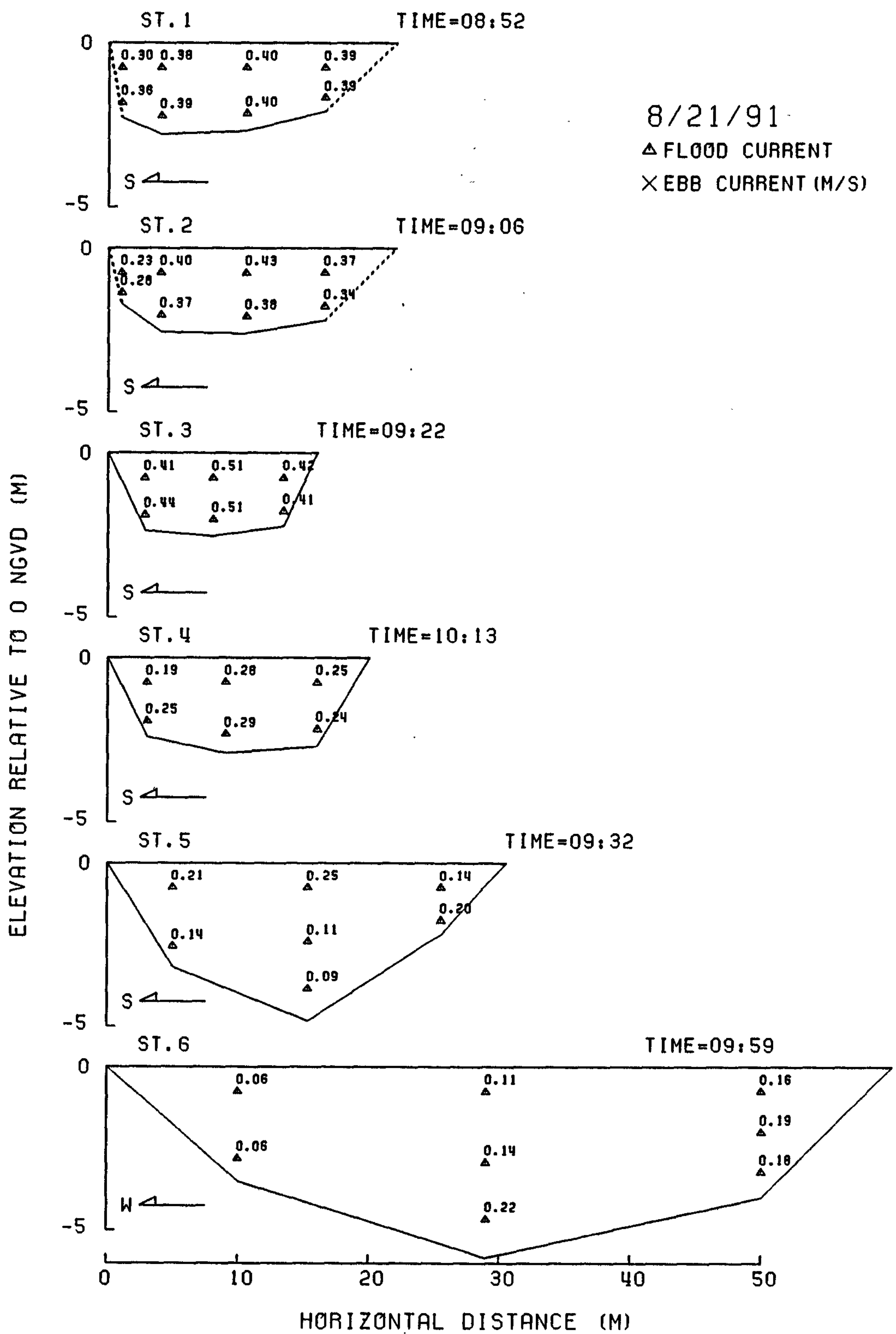

Figure 7: Current measured in 8/21/91 (8:00-10:30).

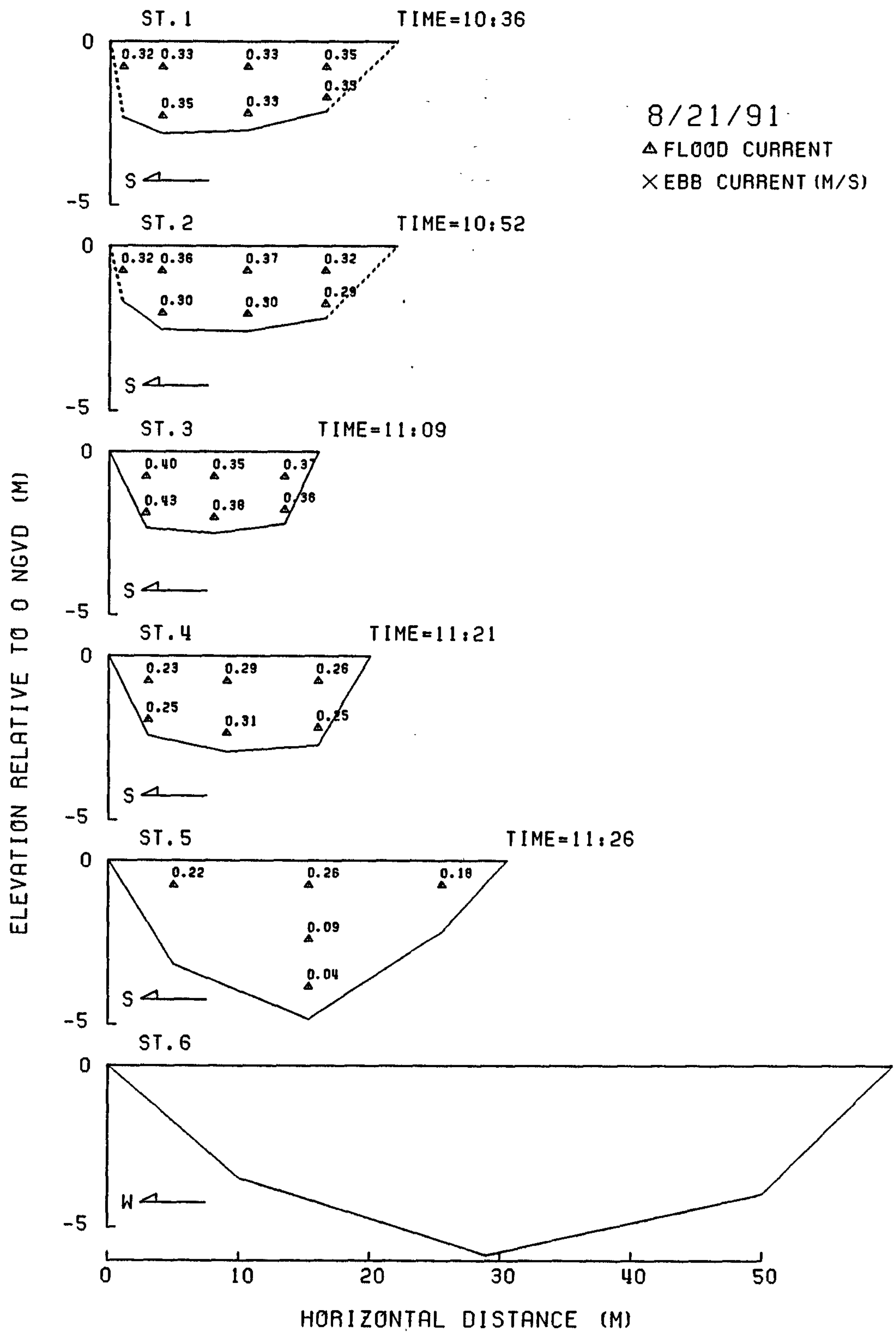

Figure 8: Current measured in 8/21/91 (10:30-12:00).

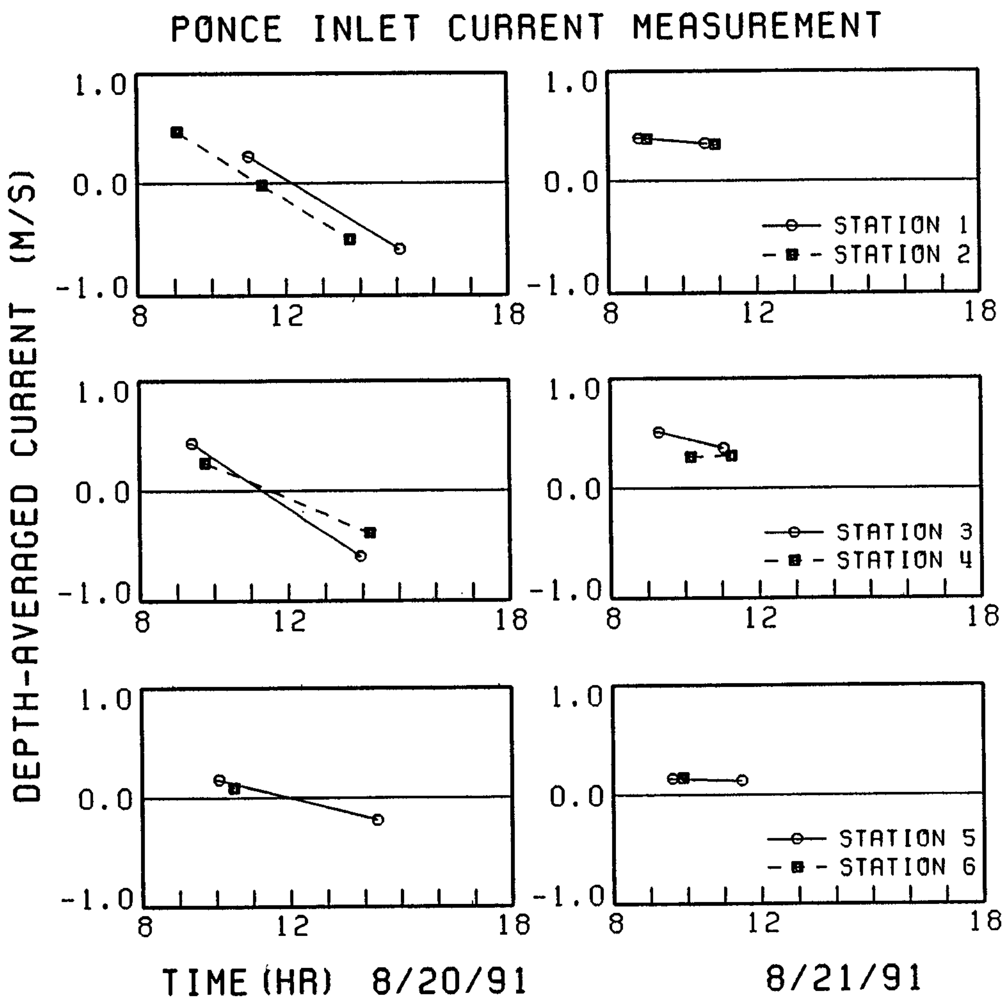

Figure 9: Depth-averaged currents measured at all six stations.

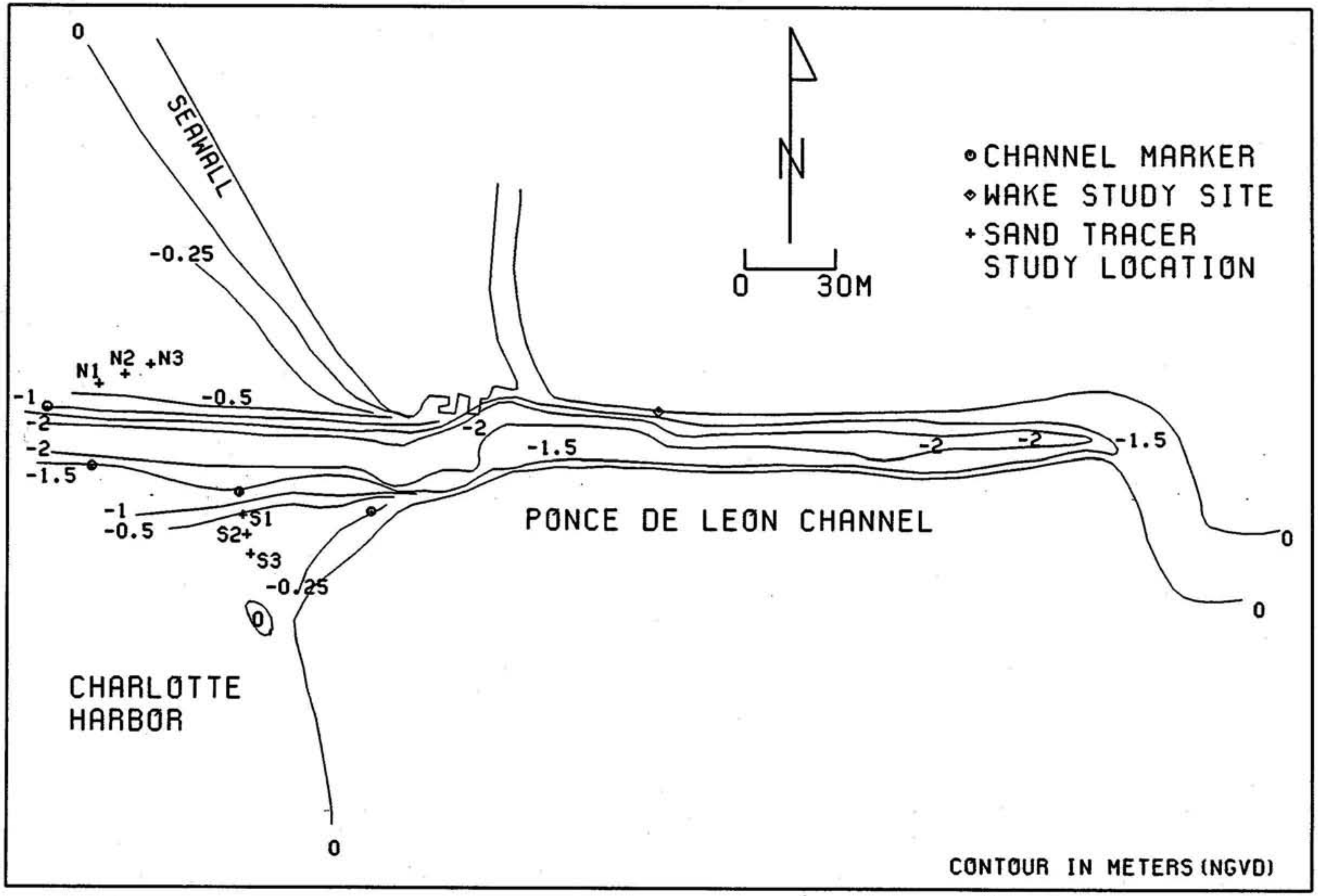

Figure 10: Measured bottom bathymetries.

3.4 Flow Pattern Measurement

The results of the drogue studies are given in Figures 11 and 12 for flood and ebb conditions, respectively. These flow patterns are typical for a tidal entrance, that is, during flood tide flow enters the channel like entering a funnel while during ebb tide flow returns to the bay like a jet causing entrainment from two sides. Both flood and ebb flow patterns show stronger currents along the south bank in agreement with the cross section currents measured in Station 1.

3.5 Sand Tracer Study

The sand tracer study was performed just outside the inlet entrance by the navigation channel. Dyed sand was placed at six locations, three at the north side of the channel and three at the south side (see Figure 10). Samples were taken 24 hours after the initial deposition. The contours of tracer concentration after 24-hour period are plotted in Figures 13 and 14. The overall net transport is towards the entrance into the channel with the exception at location S1 close to the ebb channel where sand is moving offshore.

3.6 Boat Wake Study

Boat wake study was performed in the lower reach of the Ponce channel. Eleven runs were carried out for the study using an 18-ft Mckee craft with different speeds in the range between 2.2 m/s, or 4.3 knots (near idle speed) to 5.2 m/s, or 10.1 knots. Wake height and the wake-induced water particle velocity were recorded by wave gage and flow meter, respectively, installed by the north bank of the channel near inlet entrance (see Figure 10). Three additional wake records were also collected when local boats passing through the channel; one is from a 10 ft small boat, another from a 18-ft median size boat and the third from a 25-ft large size boat. All of them were cruising at idle speed in the order of 2.5 m/s, or 4.8 knots. The time series of the wake height and the bank-normal flow velocity were plotted in Figure 15. The measured peak values of wake height and velocity are listed in Table 1. The wake height increases with increasing boat speed to a certain value, here in the order of 8.5 knots. Beyond this speed the wake height actually decreases as the boat starts to plane on the surface.

The wake-induced maximum height and water particle velocity can also be predicted by some ship wave models. Appendix A presents a simple model developed by Sorensen and Weggel (1984). The model is utilized to predicting the maximum wake information and the results are compared with the measured data.

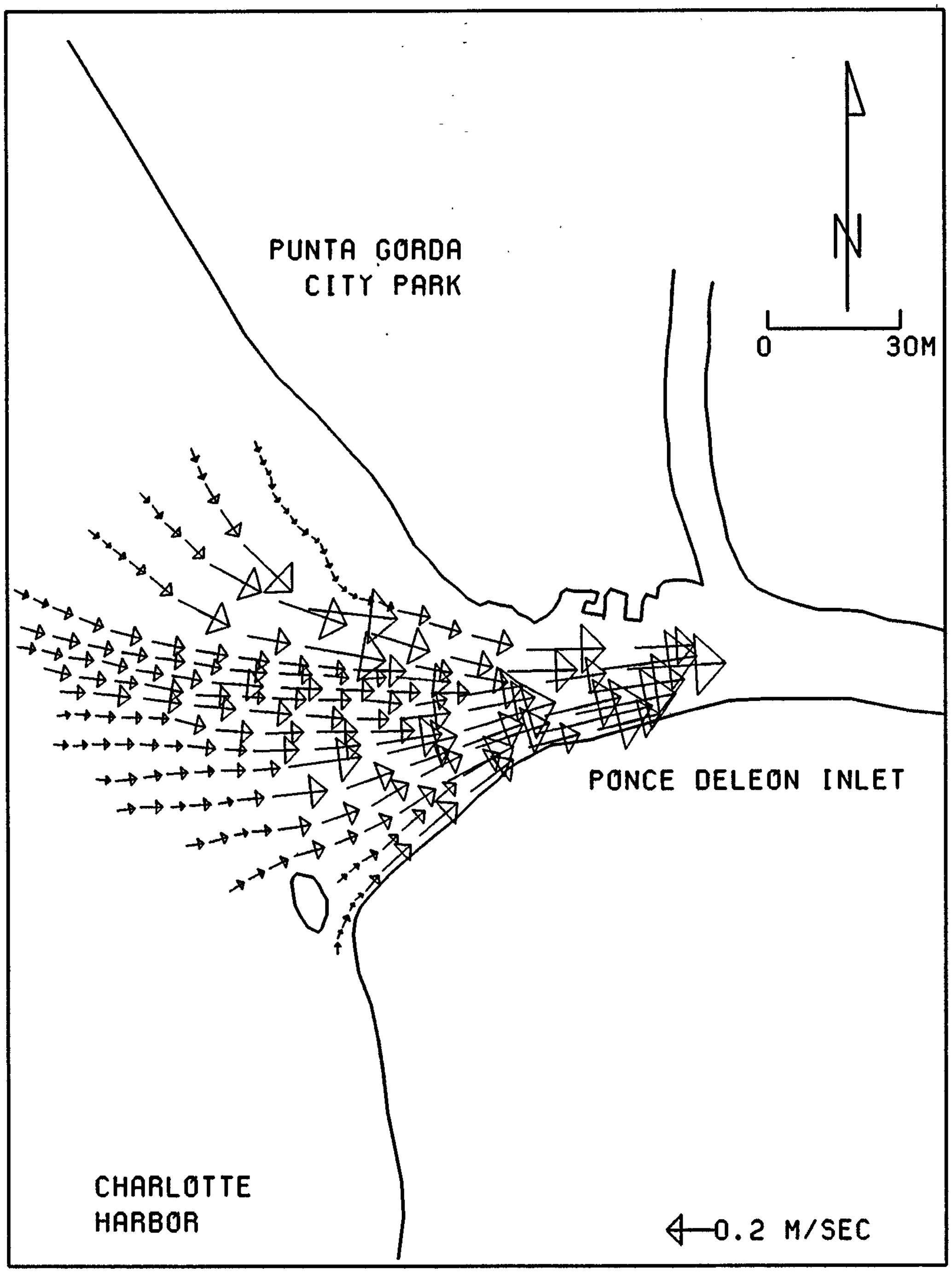

Figure 11: Flood flow pattern measured from drogue study.

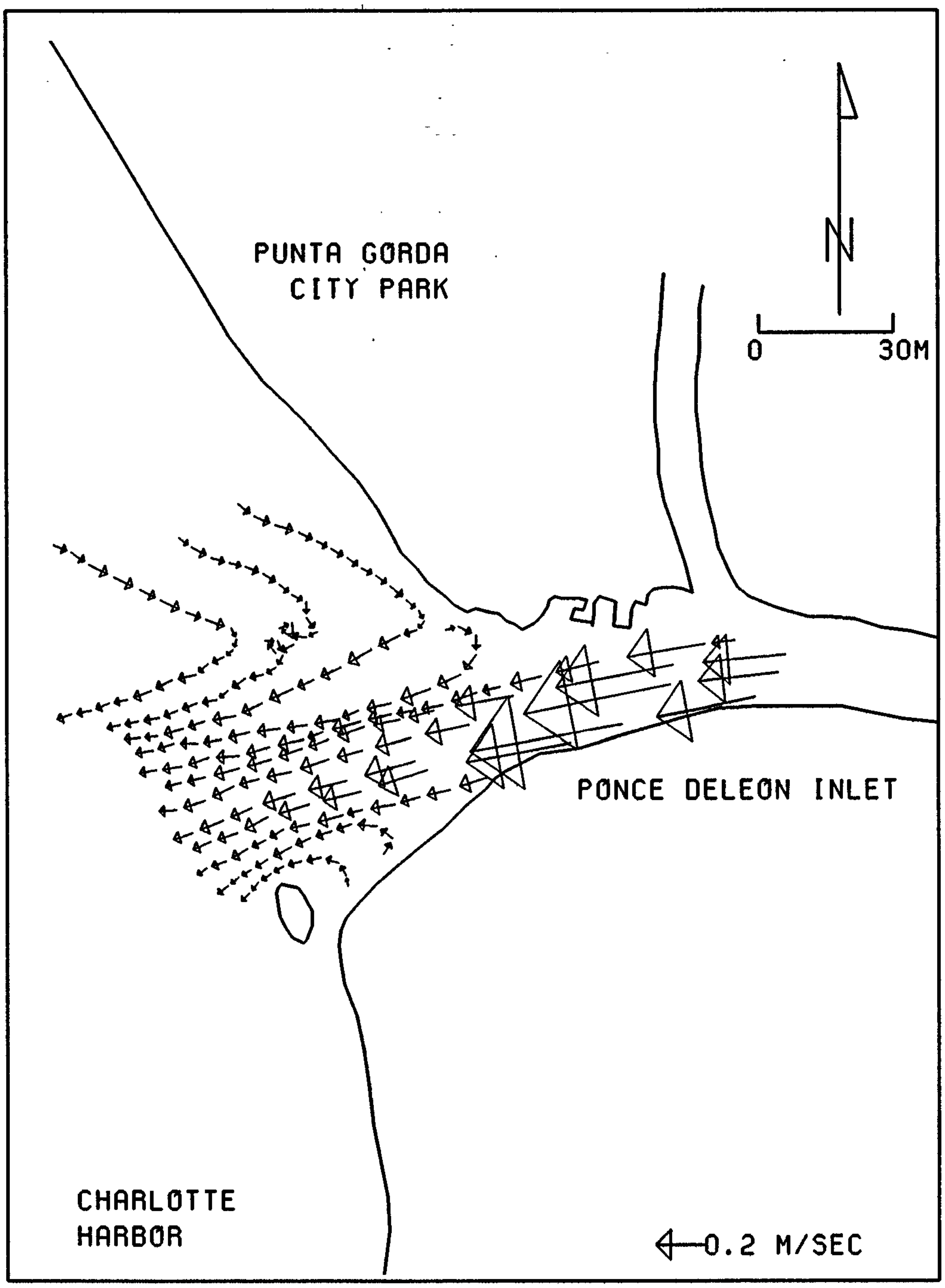

Figure 12: Ebb flow pattern measured from drogue study.

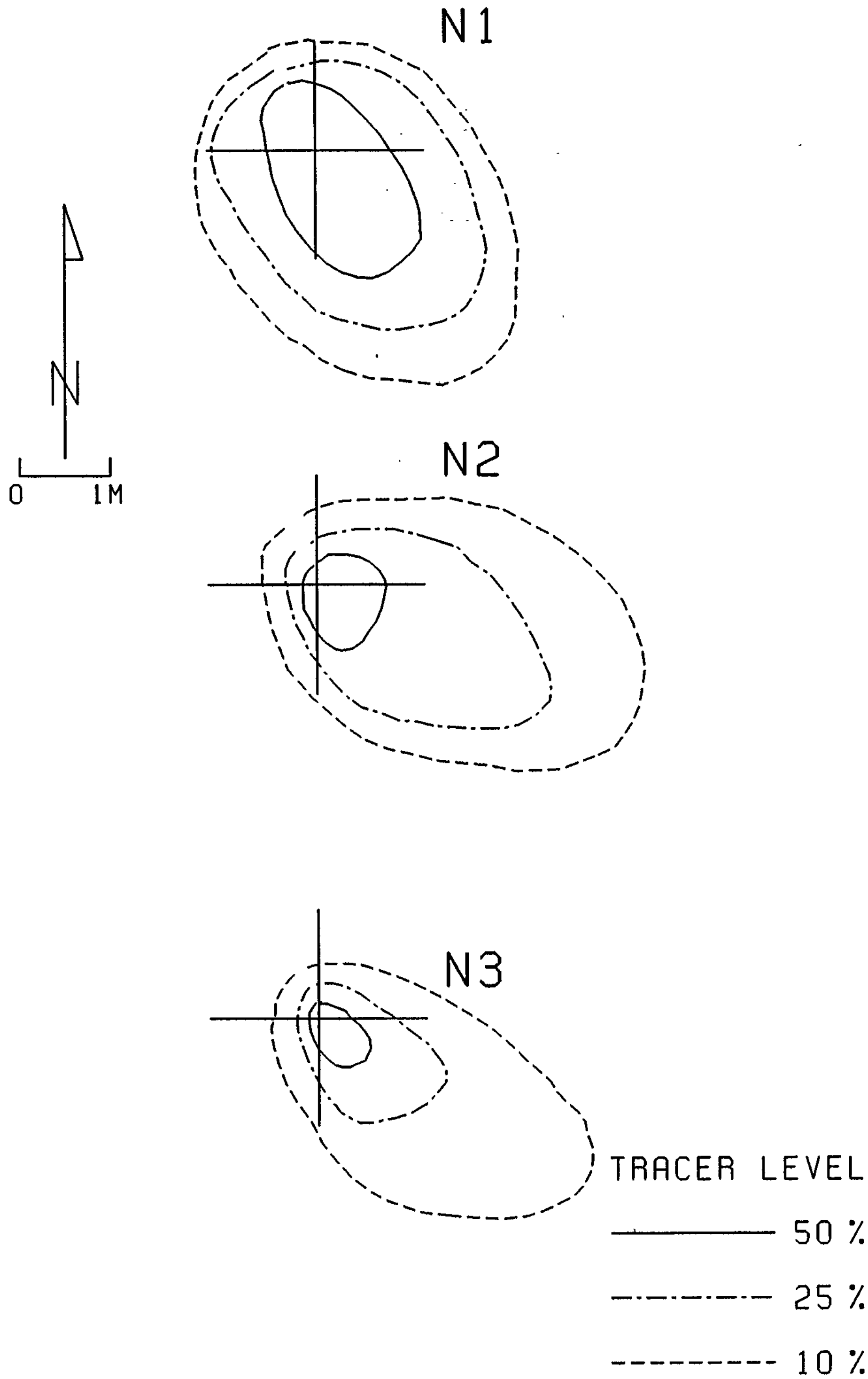

Figure 13: Results of sand tracer study for N1, N2 and N3 locations.

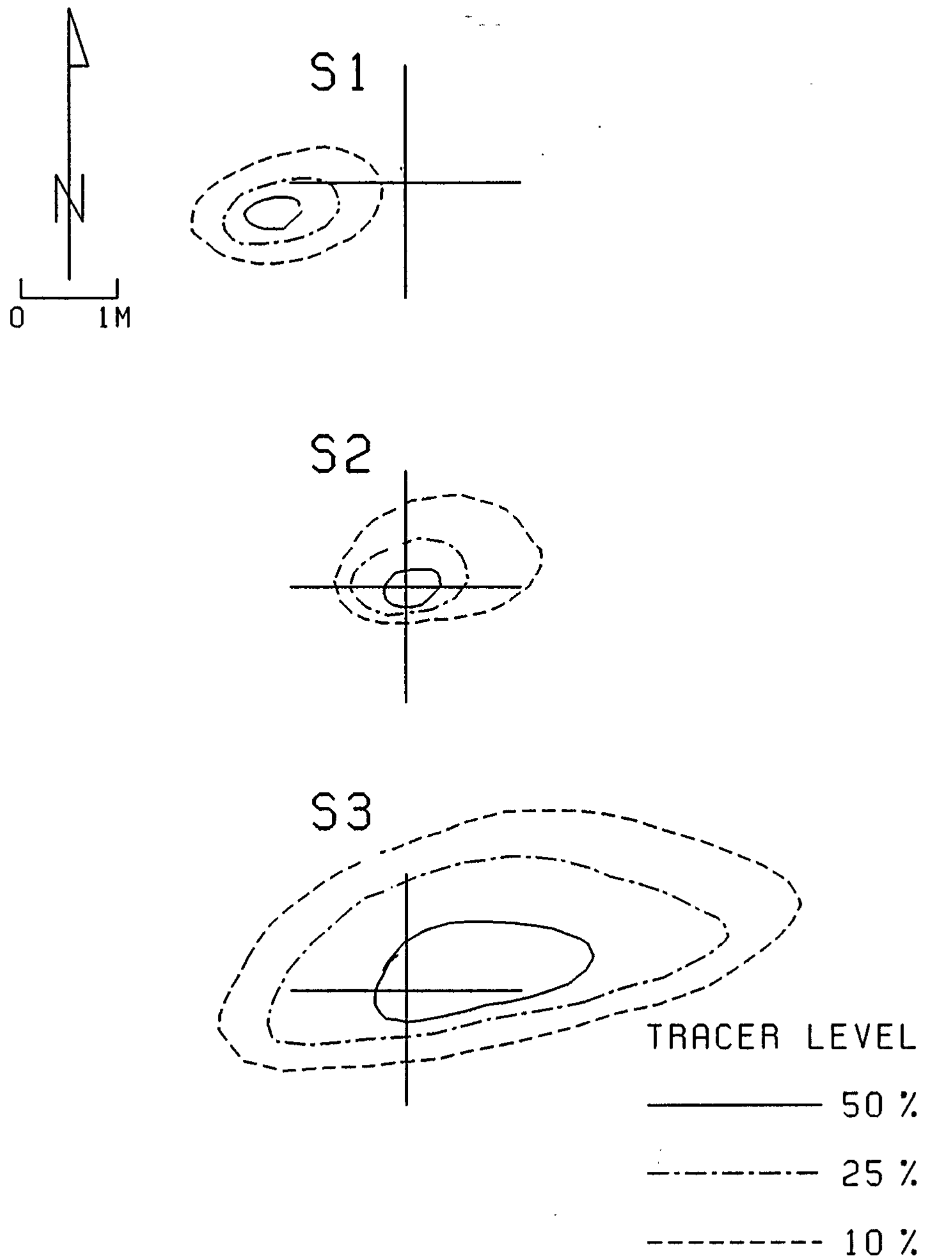

Figure 14: Results of sand tracer study for S1, S2 and S3 locations.

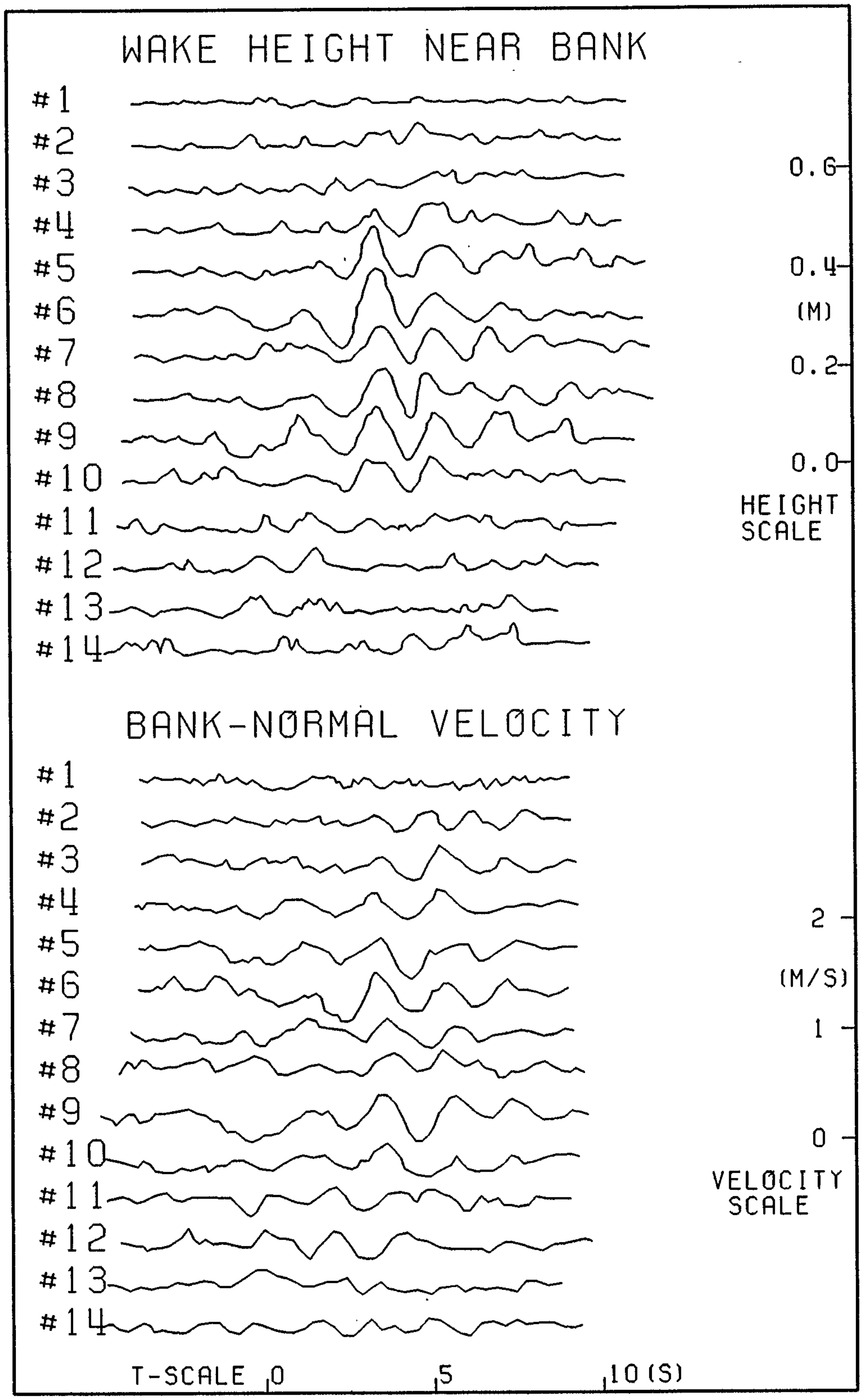

Figure 15: Measured boat wake heights and wake-induced velocities.

Table 1: Measured wake height and bank-normal velocity vs. boat speed.

Exp.	Boat Speed		Boat Wake	
I.D.	(Knot)	(m/sec)	H*(m)	V*(m/sec)
1	4.3	2.2	0.020	0.112
2	5.5	2.8	0.043	0.168
3	6.3	3.2	0.036	0.256
4	6.7	3.5	0.069	0.231
5	7.5	3.9	0.105	0.383
6	8.6	4.4	0.161	0.346
7	10.1	5.2	0.075	0.273
8	8.9	4.6	0.101	0.254
9	7.5	3.9	0.112	0.415
10	6.8	3.6	0.072	0.300
11	5.7	2.9	0.038	0.211
12	4.8	2.5	0.048	0.264
13	4.8	2.5	0.041	0.152
14	4.8	2.5	0.048	0.140

* Maximal height and velocity measured in each experiment.

4 Wind Wave Estimation

One of the factors causing bank erosion is the wind waves generated in the Charlotte Harbor penetrating into the channel. The entrance of the channel is located in an exposed area. The fetch in the south west orientation is approximately 20 km (12.5 miles). Based on this fetch length the magnitudes of the wind generated waves near the entrance were estimated for various wind speeds. The results are given in Table 2 together with the wave-induced water particle velocity along the bank near the entrance. The detail of the method was given in the 1984 report concerning nearby Bass Inlet in Charlotte Harbor (Wang, et.al.,1984).

5 Bank Erosion Assessment

The potential of bank erosion is assessed here based on the concept of excess shear stress, that is, if the flow-induced shear stress is larger than the soil resistance, the bank will be erosional, otherwise, the bank will be stable. This excess shear

Table 2: Prediction of wave-induced along-bank velocity.

Souther Wind		Waves in Channel*		
(mph)	(m/sec)	H(m)	T(s)	U^{**}_{max}(m/sec)
10	4.47	0.28	2.03	0.45
20	8.94	0.56	2.89	0.73
30	13.41	0.77	3.46	0.96
40	17.88	0.94	3.91	1.15
50	22.35	1.09	4.28	1.30

* based on 20Km long and 4m deep fetch.
**estimated at surface level in a 2m-deep channel.

stress is computed from the following equation:

$$\tau_b \propto \tau - \tau_c = K(u_*^2 - u_{*,c}^2) \tag{1}$$

where u_* is the flow-induced shear velocity, $u_{*,c}$ is the critical shear velocity and K is an erosion coefficient. This equation states the excess shear stress is proportional to the difference of the shear velocity squared.

The critical shear velocity $u_{*,c}$ for the local bank material is in the order of 0.01 to 0.02 m/s without consideration of the added resistance due to vegetations. The quantity of u_* can be related to the mean stream velocity $\bar{u}$ by the relationship:

$$u_* = \sqrt{f/8} \cdot \bar{u} \tag{2}$$

where f is the bank friction coefficient. For a common sandy bank with or without minor vegetation growths the value of f is in the order of 0.01 to 0.03. For banks covered with mangroves such as in the Ponce channel the value of f is much larger. For flood water over mangrove growth field, f is around 0.12 based on the experiment carried out by Wang (1983). This f will yield the flow-induced stress 4 to 12 times larger than the corresponding flow-induced stress over sandy surface. However, the resistance is also increased due to the presence of the mangroves. The actual flow along the bank soil could be further retarded as the flow meanders through the mangroves. In all likelihood, the presence of mangroves should reduce the net erosional force on the soil. For lack of actual data, the treatment here is to assume that the additional flow stress induced by the presence of the mangroves is compensated by the additional resistance provided by the mangroves. A value of f=0.02 is, therefore, used to calculate the bank erosional stress with bank resistance force calculated on the basis of no vegetation. This flow-induced shear stress vs. the mean velocity is plotted in Figure 16. The critical soil resistance is also plotted in the same figure as dash lines. When the flow-induced stress exceeds the critical stress bank erosion will occur otherwise the bank is stable. According to this graph, the channel becomes stable if the current velocity is less than, say, 0.4 m/s.

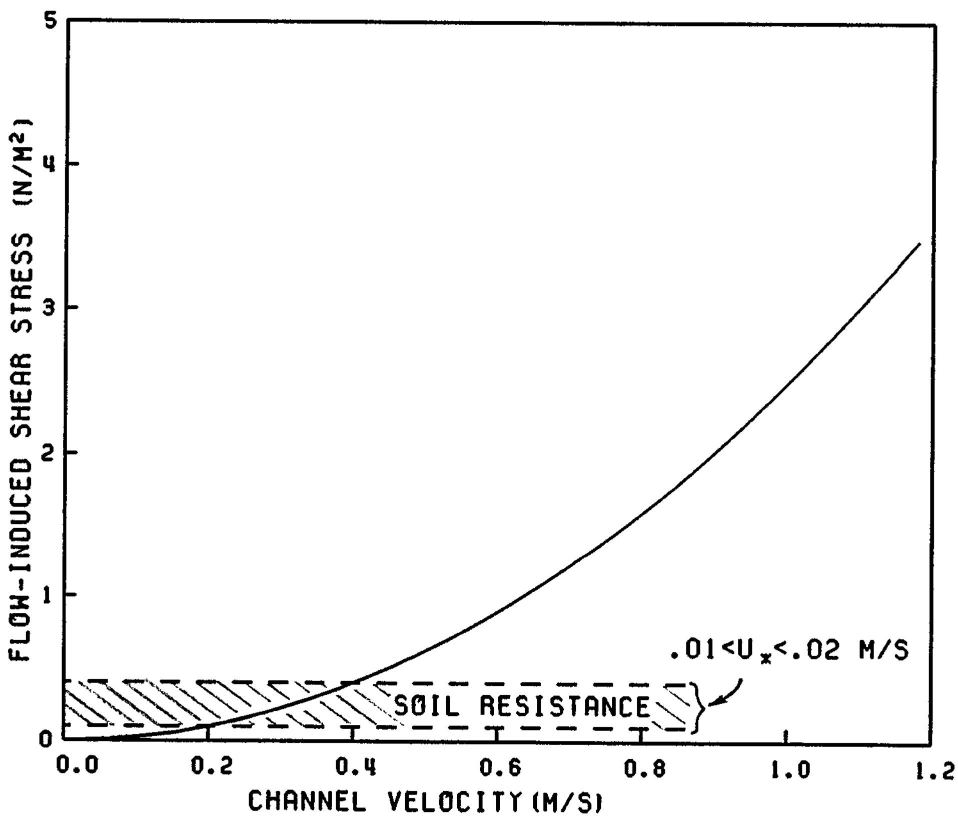

Figure 16: Flow-induced shear stress vs. mean velocity.

Table 3: Boat traffic statistics at Ponce de Leon channel of February 91.

Weekday	Week*					Sum	Ave.
	1st	2nd	3rd	4th	5th		
Sunday		49	32	69	57	207	52
Monday		90	5	25	99	219	55
Tuesday	56	45	49	79	79	308	62
Wednesday	48	64	32	49		193	48
Thursday	39	42	35	31		147	37
Friday	79	57	80	40		256	64
Saturday	124	94	133	125		476	119
Sum	346	441	366	418	235	1806	
Ave.	69	63	52	60	78		

* week of the month

It is apparent that magnitudes of flow currents in the Ponce channel are affected by several factors, including tides, wind waves, runoff, and boat traffic, etc. Of the three major factors considered – tides, wind waves and boat traffic, the boat-wake induced current which is on the order of 0.2 m/s clearly will not cause any serious bank erosion. Moreover, the boat traffic only constitutes a very small fraction of the total duration when it is compared with the tides and wind waves (based upon the one-month boat traffic statistics given in Table 3, furnished by the City of Punta Gorda). Therefore, the effect of boat wake alone is negligible in Ponce channel.

The tidal current in the lower reach of the channel near the entrance was measured to be in the vicinity of 0.65 m/s along the south bank and 0.45 m/s along the north bank. This current strength is a moderate erosional force which, by itself, might not constitute a major threat to bank erosion. The wind-wave induced current estimated in this region is, in general, in the same order of magnitude as the tidal induced current. For a steady 10 mph SW wind, for instance, the amplitude of the wave-induced current is about the same as the tidal current. This wave-induced current when superimposed upon the tidal current may become a major erosional force to the bank since the erosional stress is proportional to the square of the current speed. Thus, for a 10 mph SW wind, the peak erosional stress is more than tripled and for a 20 mph wind, the peak erosional stress increases by 6 fold, etc. The excess stress due to the combined tide and wave effect for various wind speed is indicated in Figure 17. This combined erosional stress is further compounded by the wave-induced oscillatory pressure force acting upon the soil. Since soil is particularly vulnerable to negative pressure, this oscillatory pressure force could play an important role in bank erosion.

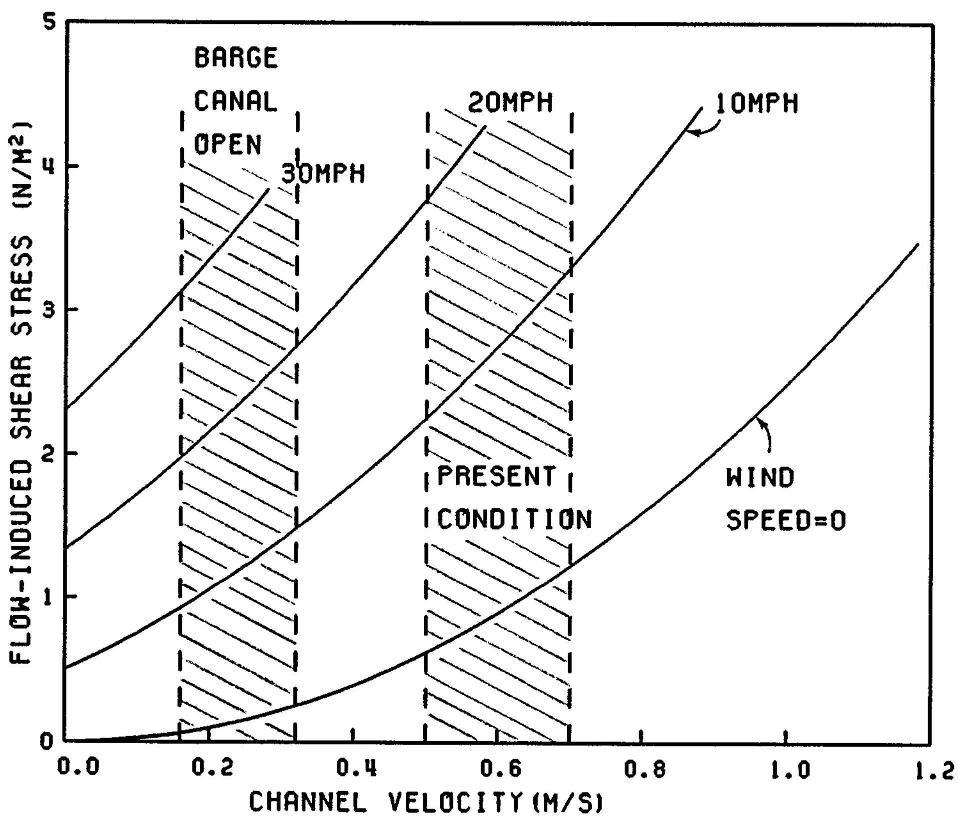

Figure 17: Excess stress due to combined tide-wave effect for various wind speed.

In the middle reach of the channel the tidal current is the main erosional force and the erosion should be moderate as explained earlier. The added wind wave effect in this reach should be minor.

Judging from the current strength, the erosional pressure should be small in the upper reach of the channel and also in the perimeter canal.

6 Effects of Barge Canal on Tidal Hydraulics

Currently the barge canal was blocked off at both ends. Thus, the vast canal network shown in Figure 1 behaves like a tidal reservoir that exchanges bay water through only two openings: the Ponce de Leon channel on the east and the canal opening at Colony Pt. on the north. To understand the hydrodynamics in the system, a numerical model was employed. The details of the numerical model were described in a companion report concerning the Canal Network in Burnt Store Isles, Punta Gorda (Wang, et. al., 1992). The model was calibrated and adjusted using the measured field data on current and discharge. The model was then used to examine the flow behavior in the study area.

6.1 Existing Condition

Under the existing condition, flows enter the canal network through Ponce de Leon channel and through Colony Pt. opening during flood period as shown in Figure 18. These two flows join together at the upper end of the Ponce channel and converge into the perimeter canal between the Ponce channel and the barge canal and then to the upper stretch of the canal system. The discharge through the Colony Pt. is almost the same as that through the Ponce Inlet, both in the order of 20 m^3/s. The discharge in the perimeter canal is slightly less than the total as part of the discharge is being stored in the connecting ponds along the Ponce channel. During ebb, the flow reverses its direction and splits into the two outlets in almost same proportion as the flood flow as shown in Figure 19. The current pattern and magnitudes during the flood and ebb are shown in Figures 20 and 21, respectively. Since the average cross-sectional area in the perimeter channel is considerably larger than that of the Ponce channel, the velocity is small in the perimeter canal even though the discharge is almost double that in the Ponce channel.

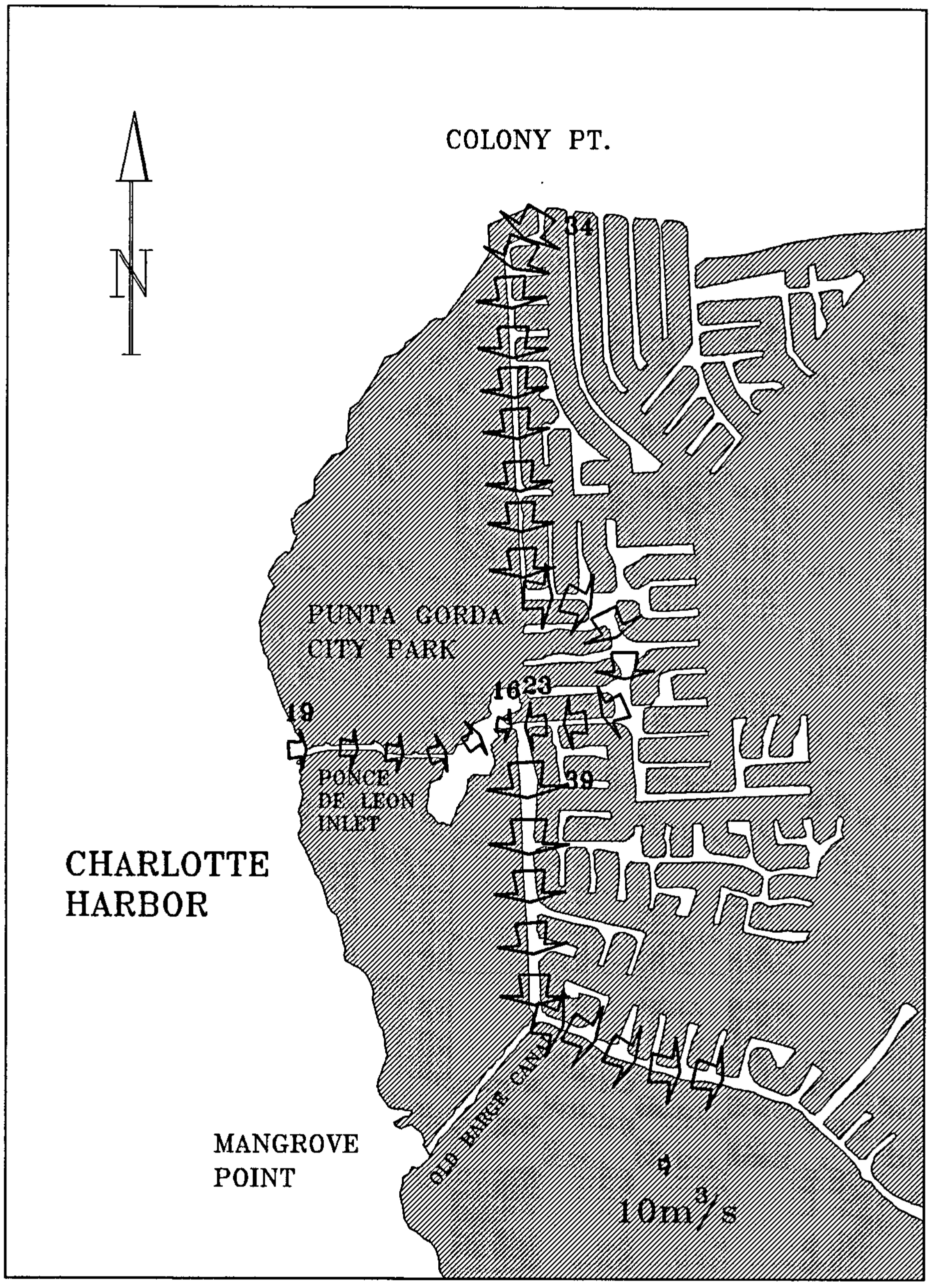

Figure 18: Time-averaged flood flow pattern under existing condition.

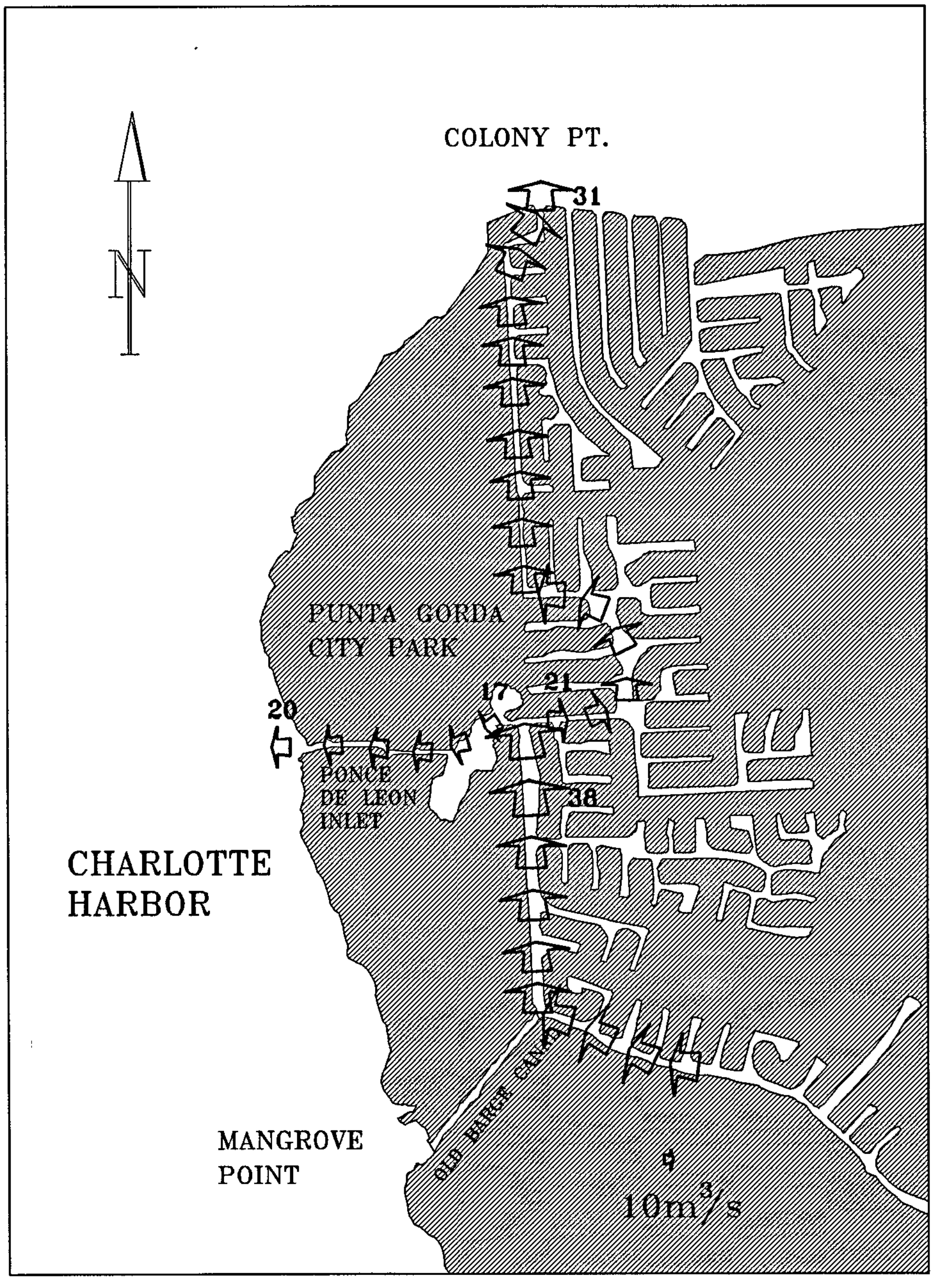

Figure 19: Time-averaged ebb flow pattern under existing condition.

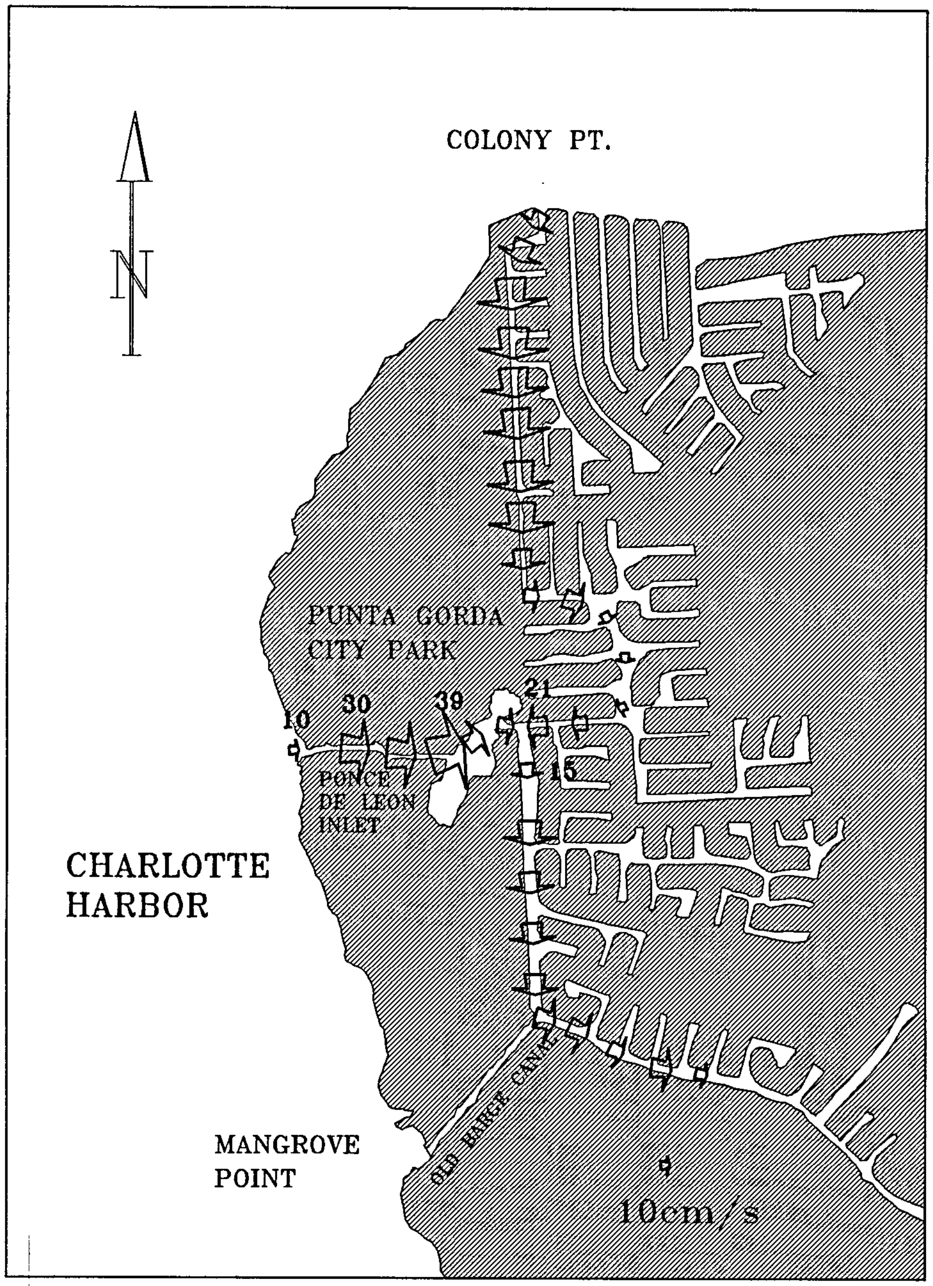

Figure 20: Current vectors of average flood flow under existing condition.

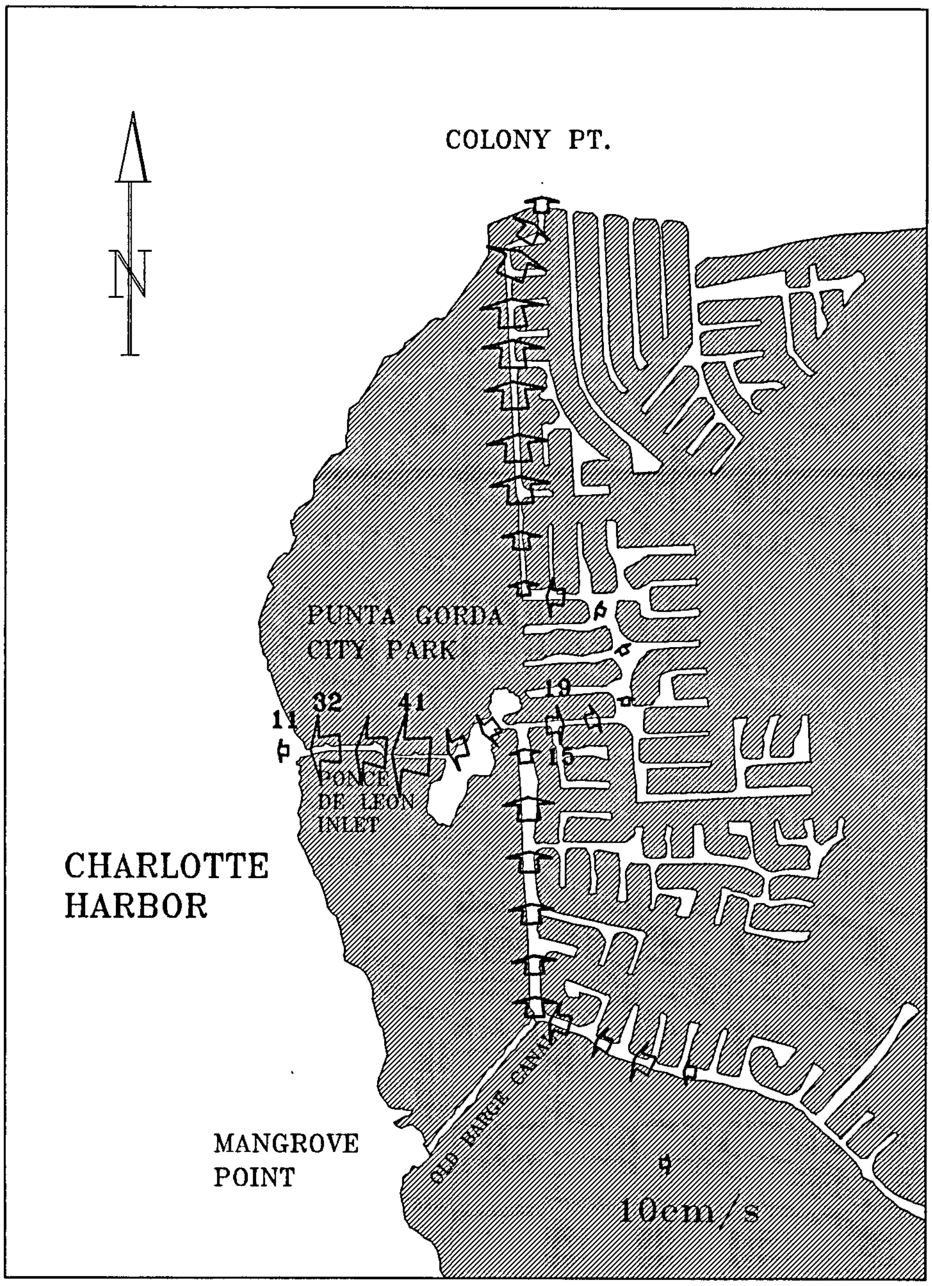

Figure 21: Current vectors of average ebb flow under existing condition.

6.2 Effect of Reopening the Barge Channel

The numerical model was used to examine the effect of opening the barge channel to allow exchange of water between Charlotte Harbor and the canal network. Assumption was made that the tidal condition at the barge canal entrance is the same as the Ponce channel entrance. The cross section of the barge canal was estimated from the aerial photo assuming a uniform depth of 2 m. The resulting flow discharges and velocities are shown in Figures 22 to 25. It is seen that reopening the barge canal has quite an influence on the tidal flow in the network. Most significantly, the discharge and the current speed in the Ponce channel are now reduced by half. This current speed alone poses no threat to bank erosion. The current and discharge in the perimeter canal as well as in the channel leading to Colony Pt. are also reduced.

7 Conclusions and Recommendations

This study aimed at identifying the major forces to cause the shoaling and bank erosion at Ponce de Leon channel. The effect of reopening an old barge canal was also investigated as to reduce the tidal forces in the Ponce channel. The study was accomplished through field measurements augmented with a numerical model. Based on the findings, proper remedial solutions were then presented.

7.1 Conclusions

Three factors were identified that could contribute to the bank erosion of Ponce channel; they are tidal induced current, wind waves penetrating from the Charlotte Harbor and wakes caused by boat traffic. Field experiments coupled with analysis and numerical modeling were conducted to establish the magnitude of influence of each factor.

The major conclusions are:

a. For the types and sizes of the boats seen in the study region and traffic frequency the boat wake alone has a negligible effect on bank erosion in the Ponce channel.

b. Bank erosion is most visible in the lower reach of the Ponce channel near the Charlotte Harbor entrance. It was determined that the combined wind wave and tidal current force is the major cause. Of the two factors, wind wave appears to play a more important role because of its dynamic nature. For

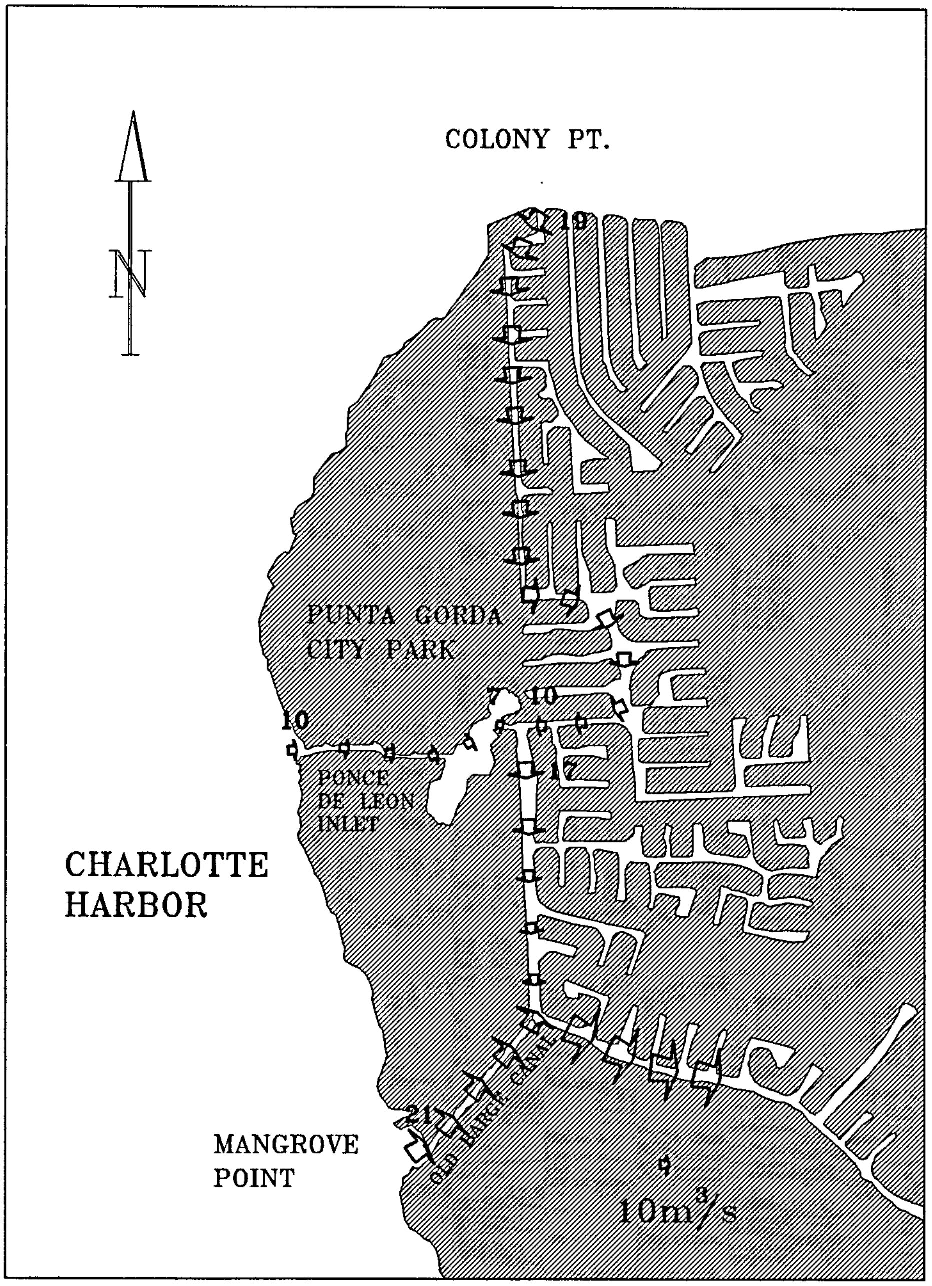

Figure 22: Time-averaged flood flow pattern under reopening of barge canal.

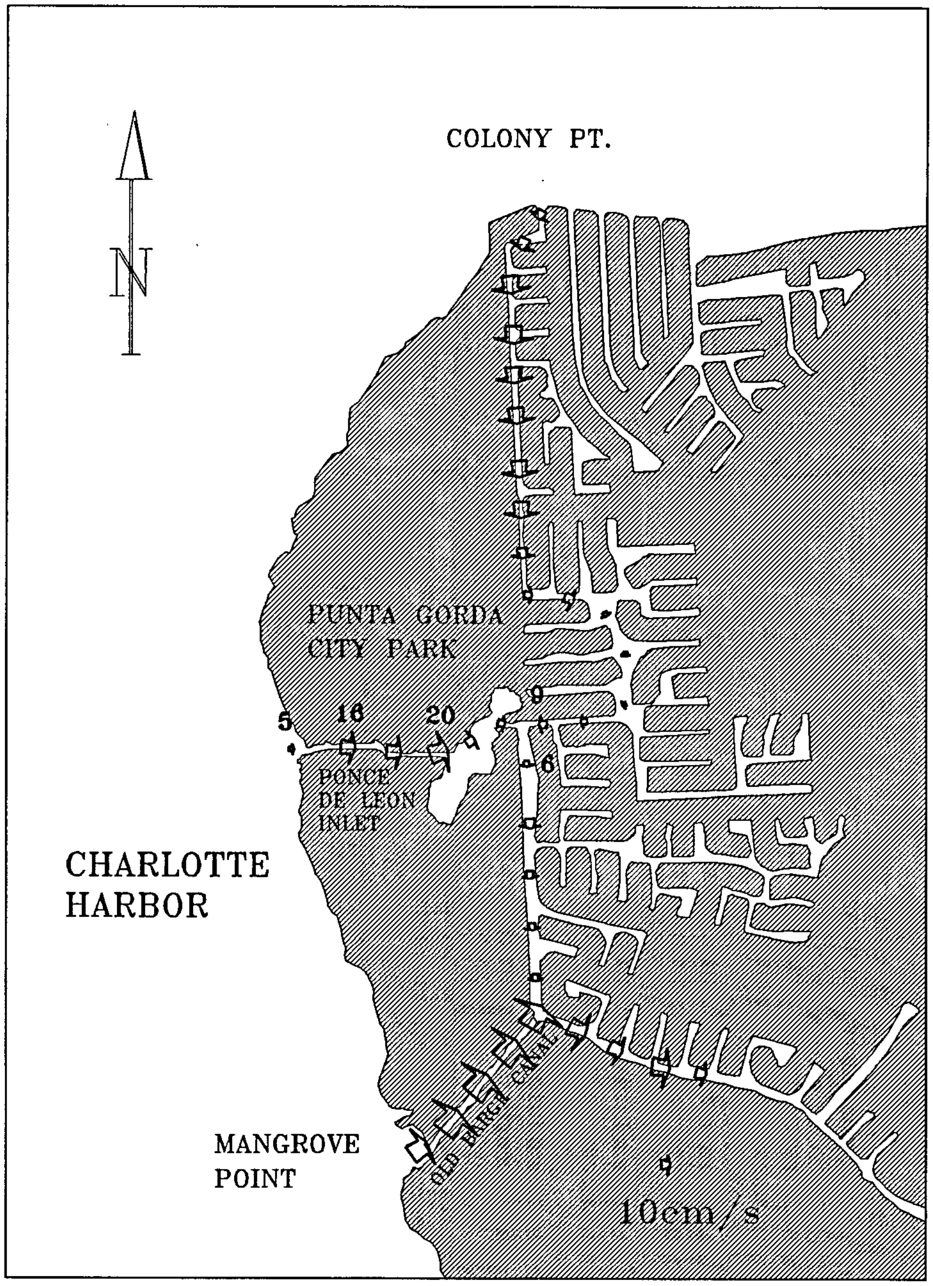

Figure 23: Current vectors of average flood flow under reopening of barge canal.

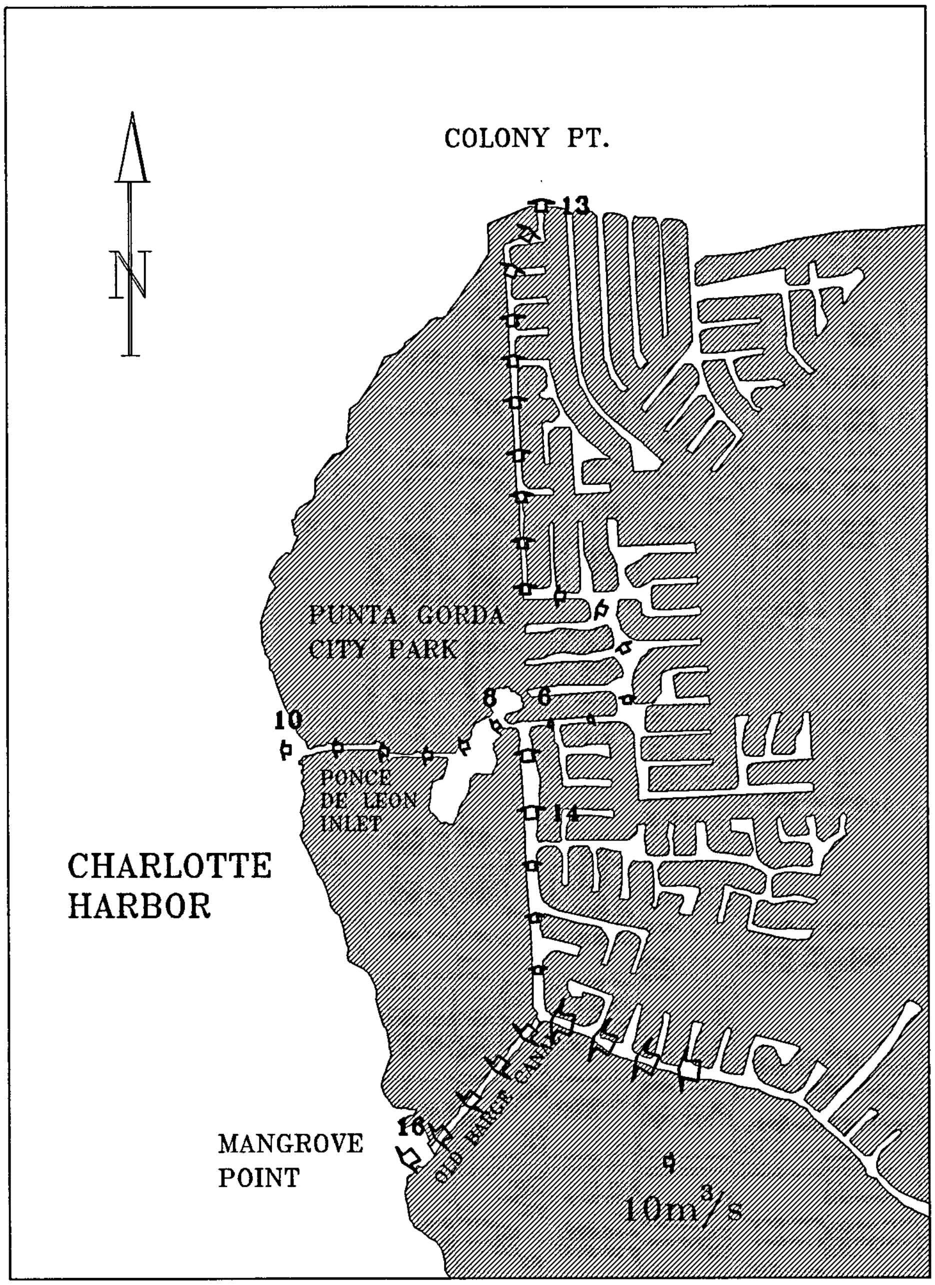

Figure 24: Time-averaged ebb flow pattern under reopening of barge canal.

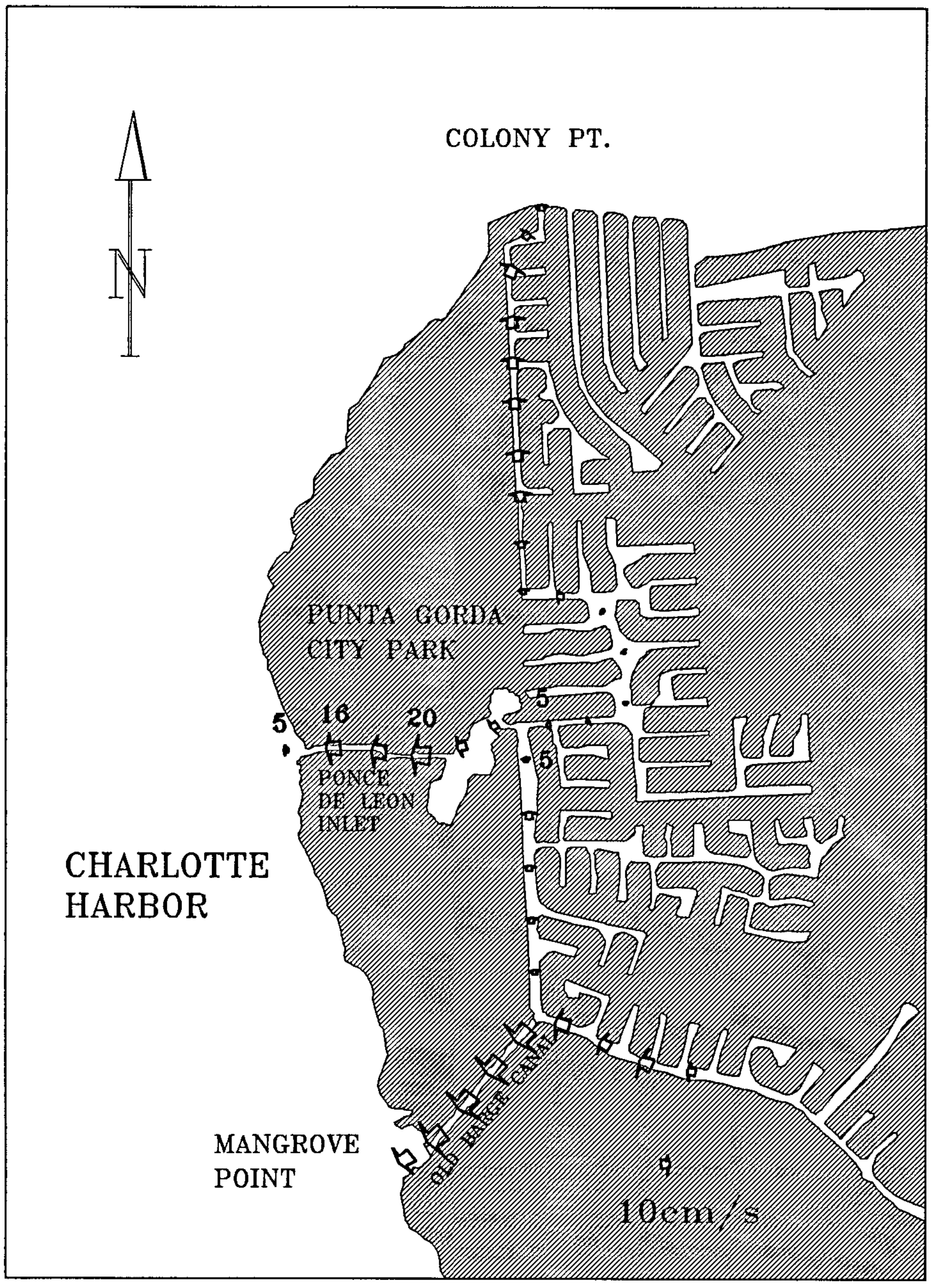

Figure 25: Current vectors of average ebb flow under reopening of barge canal.

the present cross-sectional channel condition the tidal current alone is only a moderate erosional force.

c. In the middle and upper reach of the channel, the tidal current strength is the main erosional force. The areal-averaged tidal current strength in the range of 0.4 to 0.6 m/s is a moderate erosional force for pure sandy soil. With the mangrove growth providing the additional protection the erosional potential should be further reduced.

For reopening the barge canal, it was found that the tidal current strength in the Ponce channel could be reduced by a half. This current reduction appears to be sufficient to render the middle and upper reach non-erosional. It would also reduce but not eliminate bank erosion in the lower reach as the wind-wave induced force will remain to be an important erosional factor. A side effect of reopening the barge canal is the marked reduction of current speed in the channel leading to the Colony Pt. exit. This reduction in current magnitudes could help reducing bank erosion and channel scouring.

It appears that in the development of the canal network of Punta Gorda isles and the closure of the barge canal, the current strength in the Ponce channel increased considerably owing to the compound effects of increased tidal storage (because of the creation of canal network) and reduced entrance area (because of the closure of barge canal). This causes bank erosion and the enlargement of the cross section in the Ponce channel. If wind waves were absent, the channel would eventually become stabilized when the cross section grows to be sufficiently large. However, with waves that can easily penetrate into the channel owing to the enlarged channel entrance, the channel could face more erosional pressure since the combined wave and tidal current force is larger than the summation of the individuals as explained earlier. Thus, the erosion worsens as the entrance becomes wider since, now, the wind wave could become a major erosional force to the channel.

7.2 Recommendations

The Ponce channel entrance is deteriorating caused by erosion. The adjacent shores, particularly, on the south side is losing soil as well as vegetations. As a consequence, the entrance becomes wider resulting more wave energy penetration into the channel. Material is also being carried into the channel causing shoaling just inside the entrance. This trend is likely to continue which will further aggravate the bank erosion inside the channel because of the wave energy increase. It is recommended that the entrance be stabilized with jetty structures. A jetty on the south side is considered essential. The need of a short north side jetty or bank protection measures should be evaluated. The possible jetty orientation may be aligned with the 1 m depth contours. Figure 26 shows a schematic of the jetty configurations. Further engineering and environmental information are required for optimum configuration design.

Since the barge canal closing has been established as one of the major contributing factors to the Ponce channel bank erosion, partial or complete reopening of barge canal is recommended. However, the degree and shape of the opening must be carefully determined so as not to induce excessive shoaling in the Ponce channel and/or causing adverse effects in the canal network under storm conditions such as flooding.

The most direct method of reducing erosion is to provide bank protection. This option shall be considered if reopening of the barge canal is not acceptable. The main purpose here is simply to reduce the wave and current forces impinging on the bank. Since the channel bank has no clearly defined surface the conventional stone riprap protection may not be suitable. More flexible structures should be considered. These include gabions, piles or the combination of them. Gabion structure is probably more effective and economical but piles might be aesthetically more pleasing. The extent of the protection depends upon the extent of wind wave penetration which could be as deep as 150 m into the channel under the present channel entrance condition. This bank protection, together with the jettied channel entrance, should also prevent further erosion at the middle and upper reach of the Ponce channel since less wave forces are expected in these regions.

This study is based on limited field data and simple models. It gives a reasonably accurate description on the hydraulic behavior of the region and clearly identifies the major erosional forces. The recommendations are more descriptive as quantitative. More engineering and environmental information together with better defined constraints are required to arrive at design configurations.

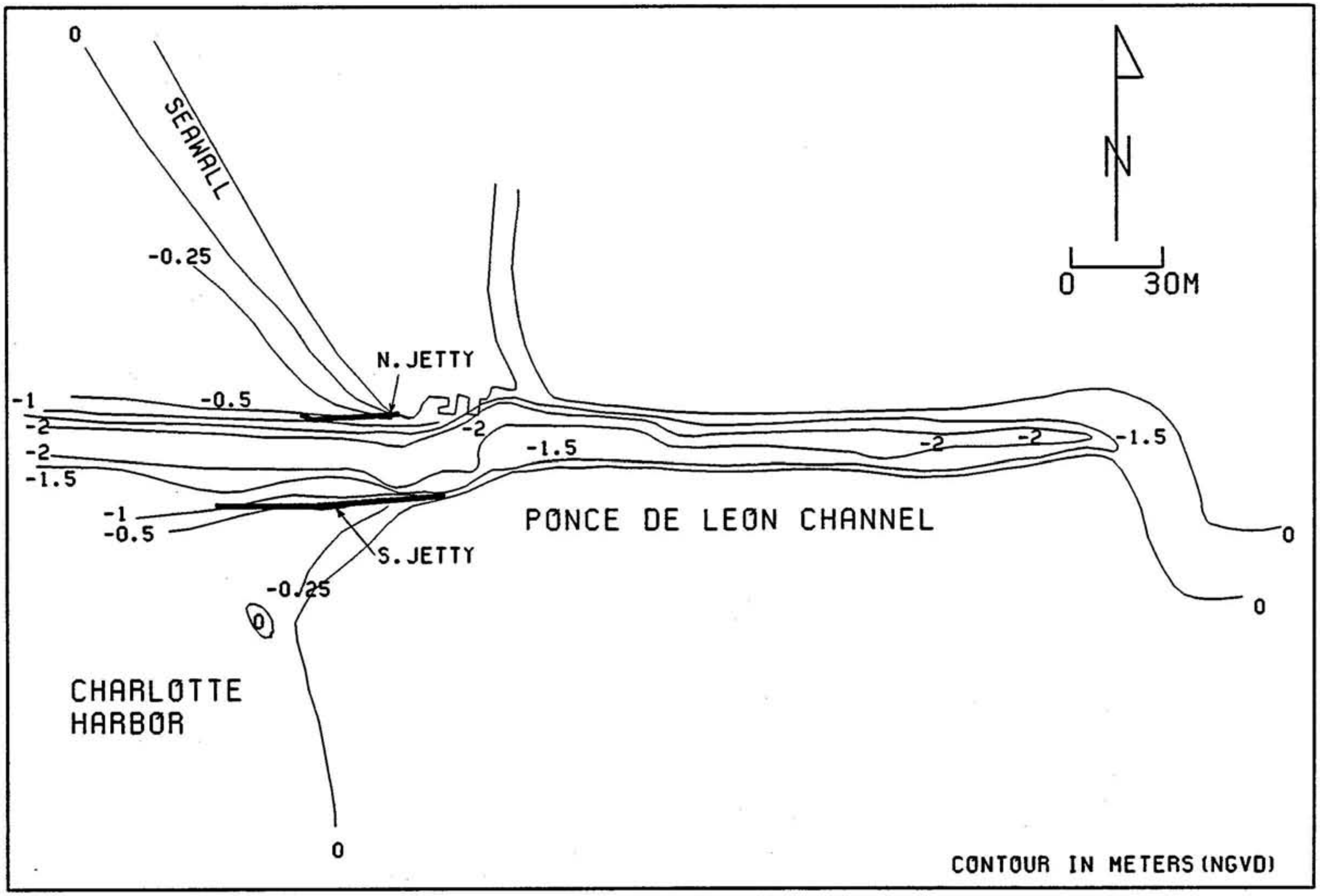

Figure 26: A schematic of possible layout of jetty configurations.

References

[1] Wang, S.Y. 1983 "Friction in Hurricane-Induced Flooding," Coastal and Oceanographic Engineering Department, University of Florida. UFL/COEL-83/011.

[2] Wang, H. 1984 "Evaluation of Channel Shoaling and Adjacent Beach Erosion, Punta Gorda, Florida," Coastal and Oceanographic Engineering Department, University of Florida. UFL/COEL-84/010.

[3] Wang, H., J.L. Lee, and L. Lin, 1992 "Flow and Transport Characteristics at a Canal Network, Burnt Store Isles, Punta Gorda, Florida", Coastal and Oceanographic Engineering Department, University of Florida. UFL/COEL-92/009.

[4] Sorensen, R.M. 1973, "Ship-generated Waves," Advances in Hydroscience, Academic Press, New York, Vol.9. pp.49-83.

[5] Sorensen, R.M. and J.R. Weggel 1984, "Development of Ship Wave Design Information," Proceedings, 19th ICCE, Houston, TX. pp.3227-3243.

[6] Weggel, J.R. and R.M. Sorensen 1986, "Ship Wave Prediction for Port and Channel Design," Ports '86, Oakland, CA. pp.797-814.

A Ship Wave Prediction Model

A.1 Prediction of Maximum Wake Height

Sorensen and Weggel (1984) presented the interim model based on an analysis of prototype ship wave data obtained from the seven different ships given by Sorenson (1973). The model predicts the maximum wake height, H_m, as a function of ship speed (U), water depth (h), distance from the ship sailing line (x), and the volume of water displaced by the ship (V).

The ship wave predictor model equation is expressed in terms of dimensionless variables, Froude number, dimensionless wave height, dimensionless distance from sailing line, and dimensionless depth:

$$\begin{aligned} F &= \frac{U}{\sqrt{gh}} \\ H_m^* &= \frac{H_m}{V^{1/3}} \\ x^* &= \frac{x}{V^{1/3}} \\ h^* &= \frac{h}{V^{1/3}} \end{aligned}$$

The model is given by

$$H_m^* = \alpha x^{*n}$$

where α and n are given in terms of F and h^* as

$$\log_{10}\alpha = -\frac{0.6}{F} + 0.75F^{-1.125}\log_{10}h^* + (2.6531F - 1.95)(\log_{10}h^*)^2,$$

$$n = \beta h^{*\delta}.$$

For $0.20 \leq F < 0.55$,

$$\begin{aligned} \beta &= -0.225F^{-0.699} \\ \delta &= -0.118F^{-0.356} \end{aligned}$$

For $0.55 \leq F < 0.9$,

$$\begin{aligned} \beta &= -0.342 \\ \delta &= -0.146 \end{aligned}$$

Generally, a linear adjustment is suggested to the predicted wave heights to reproduce the measured wave heights:

$$H_m(\text{measured}) = aH_m(\text{predicted}) - b.$$

The comparison of predicted maximum wake heights with experimental data is shown in Figure A.1.

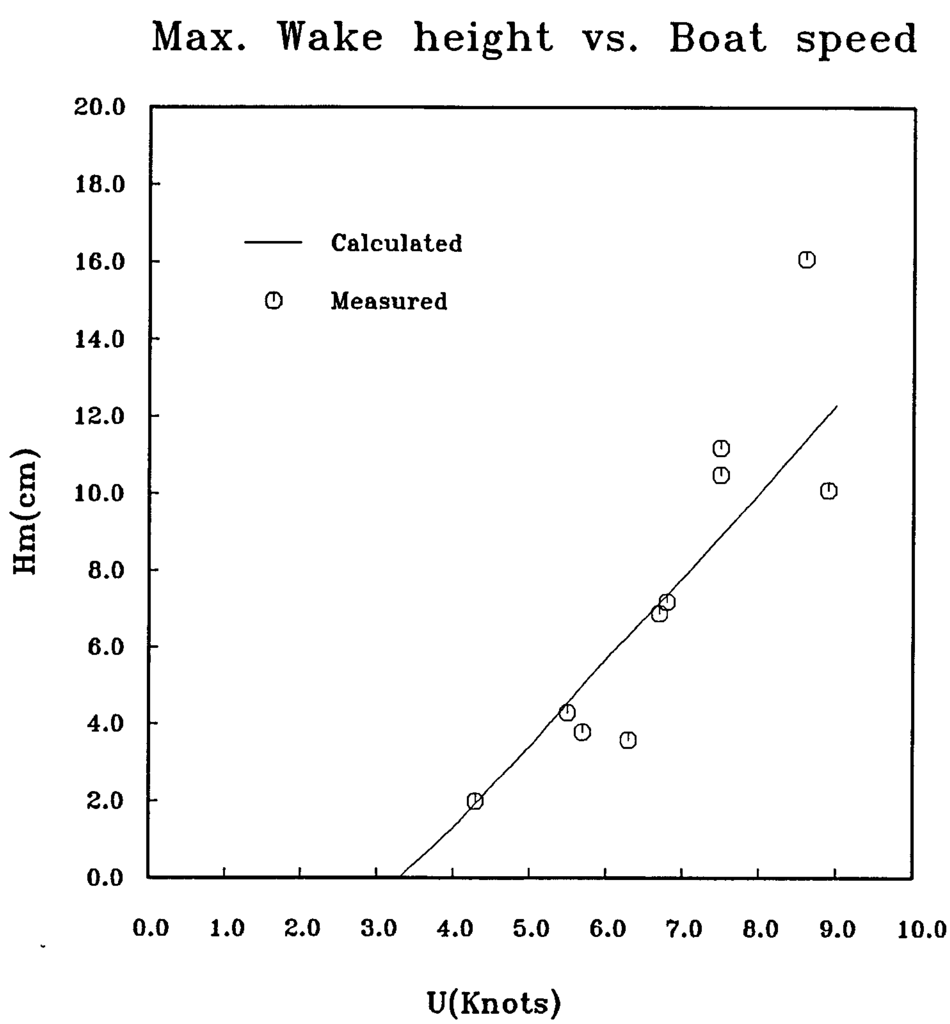

Figure A.1: Comparison of predicted and measured maximum wake heights.

A.2 Prediction of Maximum Wake-induced Velocity

The maximum wake-induced velocity is defined as

$$V_m = g\frac{H_m}{C}$$

according to the linear wave theory. The phase speed, C, can be approximated by

$$C = U\cos\theta$$

where θ is an angle of propagation of diverging wave in degrees and given by an empirical equation (Weggel and Sorensen, 1986):

$$\theta = 35.267[1 - \exp(1 - 12 + 12F)]$$

Figure A.2 shows an example of comparison of predicted maximum wake-induced velocities with measured experimental data.

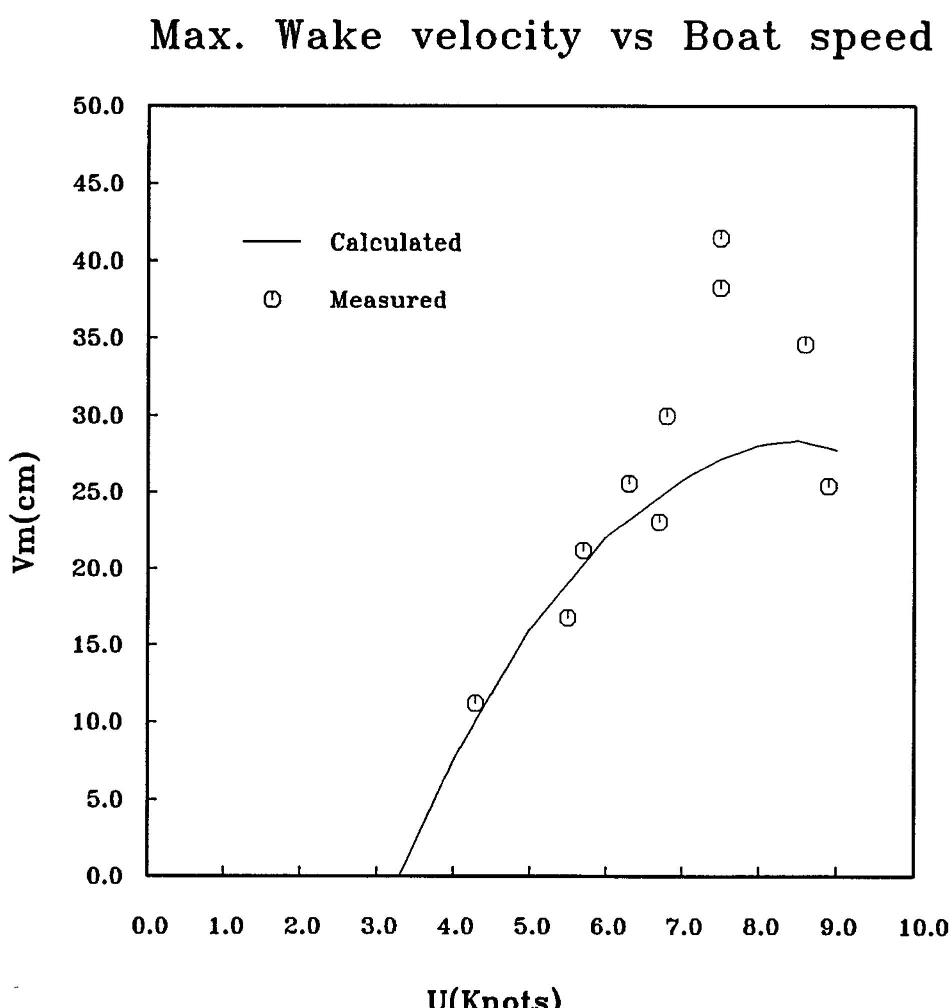

Figure A.2: Comparison of predicted and measured maximum wake velocities.