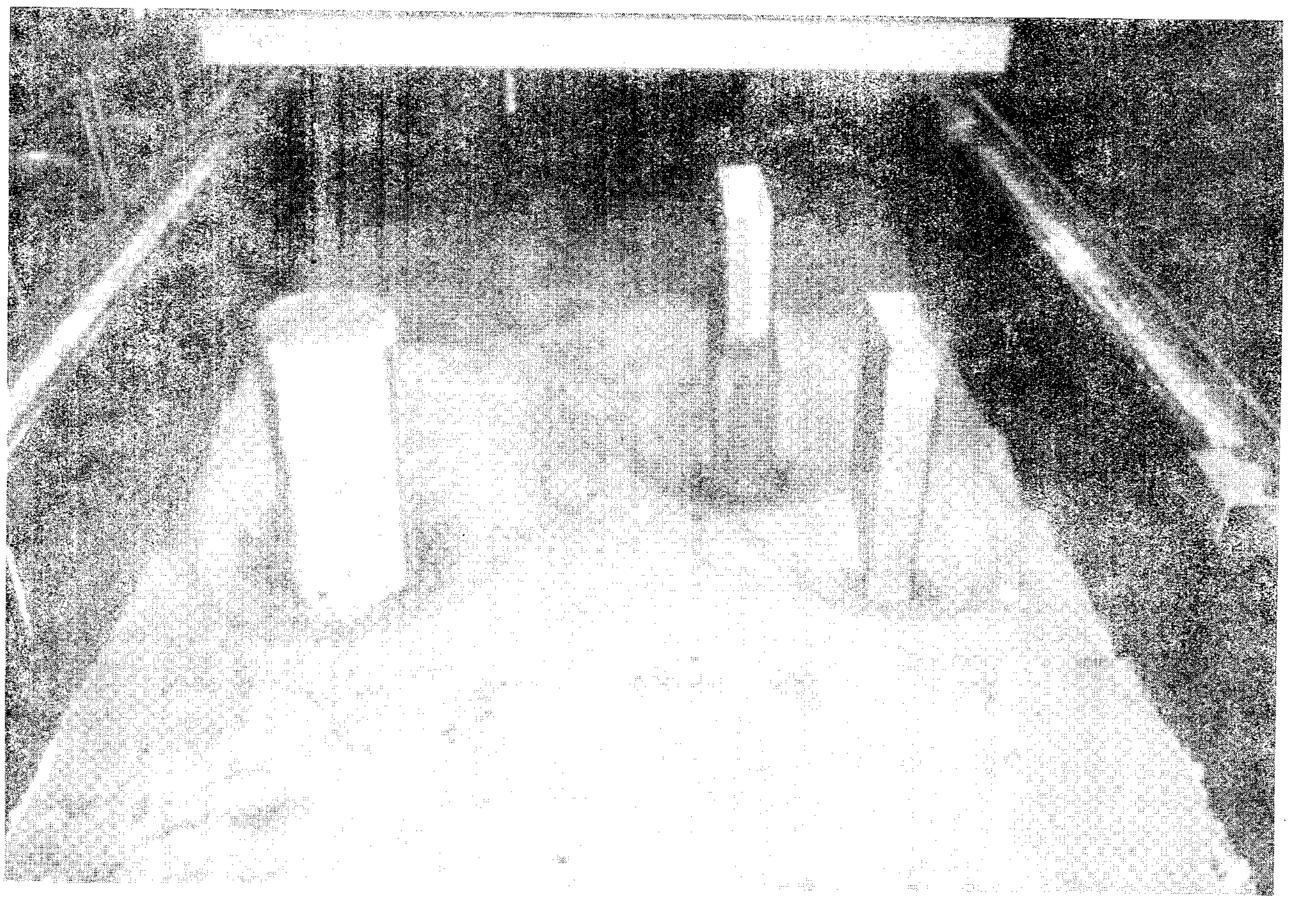
PB91-174052

Lab Report for the Acosta Bridge Scour Study

Publication No. FHWA-RD-89-114

January 1990



U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

FOREWORD

This report describes a laboratory study conducted in FHWA's hydraulics lab to investigate the relative effects that a proposed commuter bridge spanning the St. Johns River in Jacksonville, Florida, will have on scour at two existing bridges. This report describes results of the site specific scour tests at a 1:50 scale as well as several nonsite specific issues including the influence of pile spacing on scour, comparison of equivalent width piers to pile groups and riprap protection of bridge piers which were also investigated. The report will be of interest to hydraulic and geotechnical engineers who deal with scour and scour protection of bridge piers in deep channels.

Sufficient copies of this report are being distributed by an FHWA transmittal memorandum to provide a minimum of one copy to each regional office, division office, and State highway agency. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Thomas J. Pasko

Thomas J. Pasko, Jr., P. E.
Director, Office of Engineering and
Highway Operations Research and Development

NOTICE

Technical Report Documentation Page

1. Report No.	2.	3. Recipient's Catalog No.
FHWA-RD-89-114	PB91-174052	
4. Title and Subtitle LAB REPORT FOR THE ACOSTA BRIDGE SCOUR STUDY		5. Report Date January 1990 6. Performing Organization Code
7. Author(s) Stuart M. Stein, Roger T. Kilgore, and J. Sterling Jones		8. Performing Organization Report No.
9. Performing Organization Name and Address GKY and Associates, Inc. 5411-E Backlick Road Springfield, VA 22151		10. Work Unit No. (TRAIS) NCP 3D3b2152 11. Contract or Grant No. DTFH61-88-C-0004
12. Sponsoring Agency Name and Address Office of Engineering and Highway Operations R&D Federal Highway Administration 6300 Georgetown Pike McLean, VA 22101-2296		13. Type of Report and Period Covered Final Report January 1988-August 1989 14. Sponsoring Agency Code

15. Supplementary Notes

Contract Manager: J. Sterling Jones, HNR-10

Florida DOT personnel involved in Model Study: Shawn McLemore and Alice Rankeillor

Design and fabrication of Models: A. Schallenberger, J. Kurczewski, and C. Johnson

16. Abstract

The purpose of this study is to investigate the relative effects that a proposed commuter bridge spanning the St. Johns River in Jacksonville, Florida, will have on scour at two existing bridges. This was accomplished through physical modeling of the study area performed at the Federal Highway Administration's Hydraulics Laboratory.

Wooden scale models of bridge piers were placed in a sand-filled sump within a flume. Many different configurations of bridge piers were placed in the flume, and the resulting scour holes were measured and compared.

Other, nonsite-specific scour issues were also investigated. Experiments were run which tested the influence that pile spacings have on scour. Scour resulting from equivalent width piers versus pile groups was also investigated. Finally, riprap tests were performed for comparison with empirical formulas for establishing stability.

17. Key Words	18. Distribution Statement
Scour, Modeling, Piers, St. Johns River, Florida, Acosta Bridge, Riprap	No restrictions. This document is available to the public through the National Technical Information Services, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	89	

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

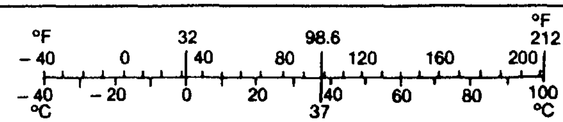
Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
	MASS			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
	MASS			
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
	TEMPERATURE (exact)			
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

Table of Contents

List of Figures

List of Figures (continued)

List of Figures (continued)

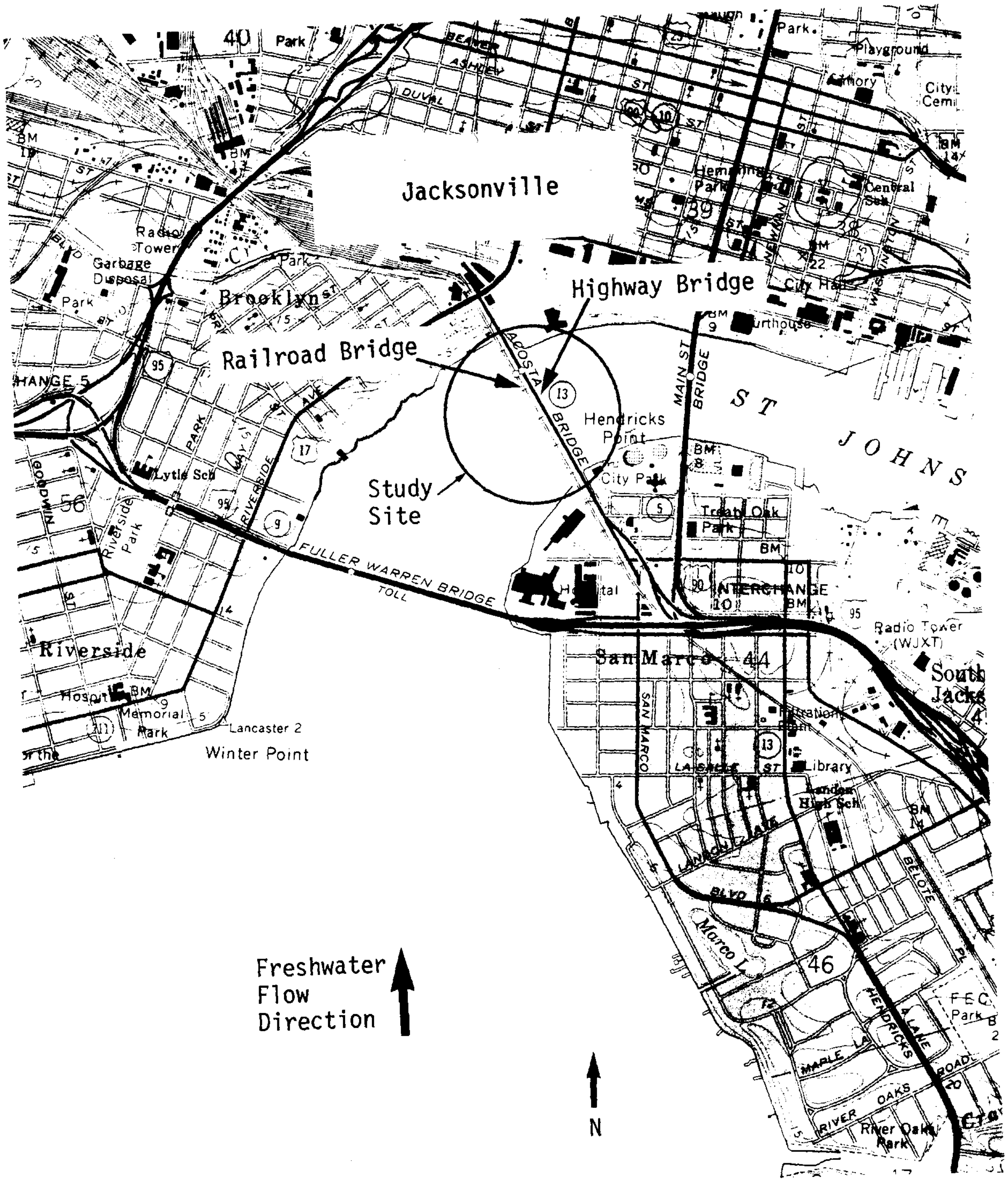
List of Tables

1. Introduction

The purpose of this study is to investigate the relative effect that a proposed Acosta Highway Bridge may have on scour at the existing bridges. The existing Acosta Highway Bridge and the Acosta Railroad Bridge span the St. Johns River in Jacksonville, Florida, as shown in figure 1. The combined narrowing and curvature, and the highly complex flow patterns of the river at this location causes increased velocities which have resulted in scour problems, especially for the railroad bridge. A commuter bridge has been proposed immediately downstream of the existing highway bridge. Since the St. Johns River is tidally influenced in Jacksonville, the proposed bridge could affect scour at the existing bridges when flow travels in the tidal flood direction. Two alternative design options are considered: one incorporating steel design, and the other incorporating concrete design.

This study is not intended to model contraction scour or velocity patterns within the cross section for this site. It is limited to a confined strip of the cross-section to determine the relative effects of flow currents from one pier on scour at other piers.

Other, nonsite-specific scour issues were also investigated. Experiments were run which tested the influence that pile spacings have on scour. Scour resulting from equivalent width piers versus pile groups was also investigated. Finally, riprap tests were performed for comparison with empirical formulas for establishing stability.



Scale 1:24000

Figure 1. Acosta Bridge vicinity map.

2. Description of Study Area

Hydrodynamics

The existing Acosta Highway Bridge is located on the St. Johns River approximately 25 miles upstream of the Atlantic Ocean. The St. Johns River has a mean tidal range of 1.2 feet at Jacksonville.(1) Tidal variation results in an average maximum velocity of 2.7 feet per second (ft/s) in the flood direction and 2.9 ft/s in the ebb direction.(2)

Physical Description

Figure 2 shows a plan view of the existing piers and the piers for the proposed (steel alternative) structure. The existing Acosta Bridge is 81.5 ft downstream of the Railroad Bridge (measured center-line to center-line) and the proposed (steel alternative) bridge would be 91.7 ft downstream of the existing Acosta Highway Bridge (again, measured center-line to center-line). Scour at railroad bridge piers 7 and 8 is of most concern, so the study concentrates on these piers as well as the piers which would have the most influence on scour at piers 7 and 8. These influencing piers include the existing Acosta Highway Bridge piers 5 and 6, the proposed bridge pier (steel alternative bridge pier 5 or concrete alternative bridge pier 4), and the temporary structure next to the steel alternative bridge pier 5.

Cross sections showing the water surface, bed elevations, and pier locations at the railroad bridge, the existing Acosta Highway Bridge, and the proposed (concrete alternative) bridge are presented in figures 3 and 4, respectively. Depths at railroad bridge piers 7 and 8 average approximately 45 ft with a maximum depth at pier 8 of 72 ft. Depths at existing Acosta Highway Bridge piers 5 and 6 average approximately 32 ft with a maximum depth of over 60 ft by pier 6. Depths at the proposed bridge pier 4 (concrete alternative) and pier 5 (steel alternative) average approximately 40 ft.

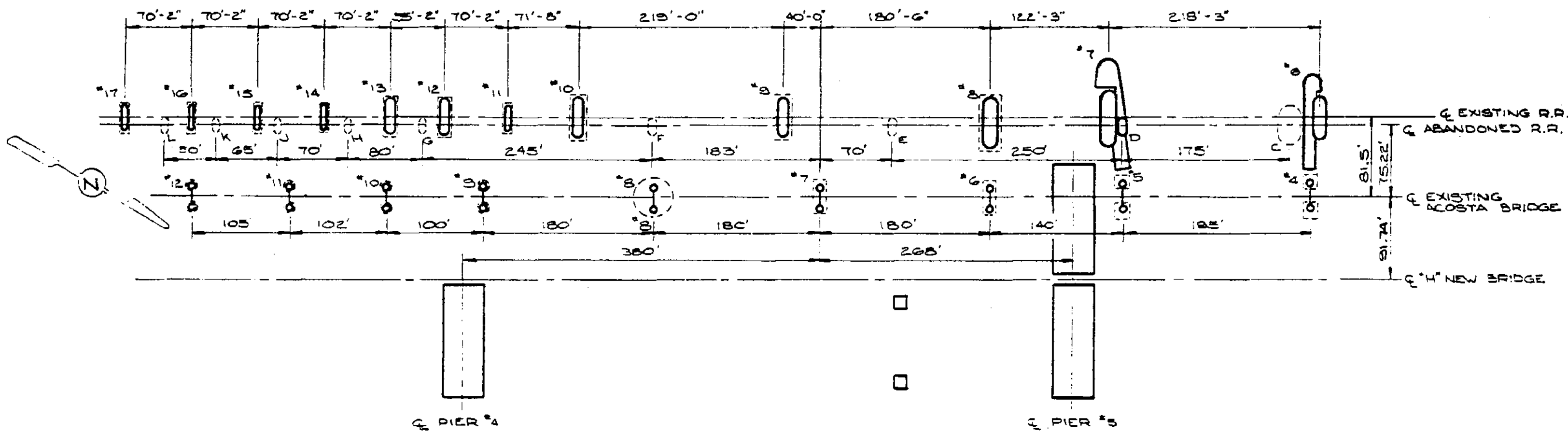
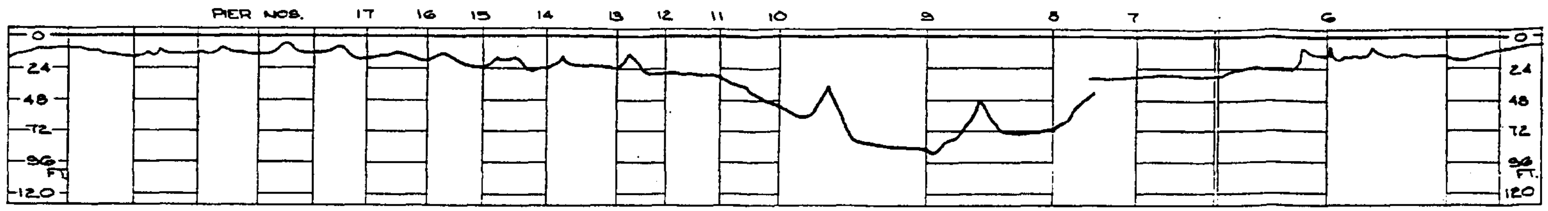
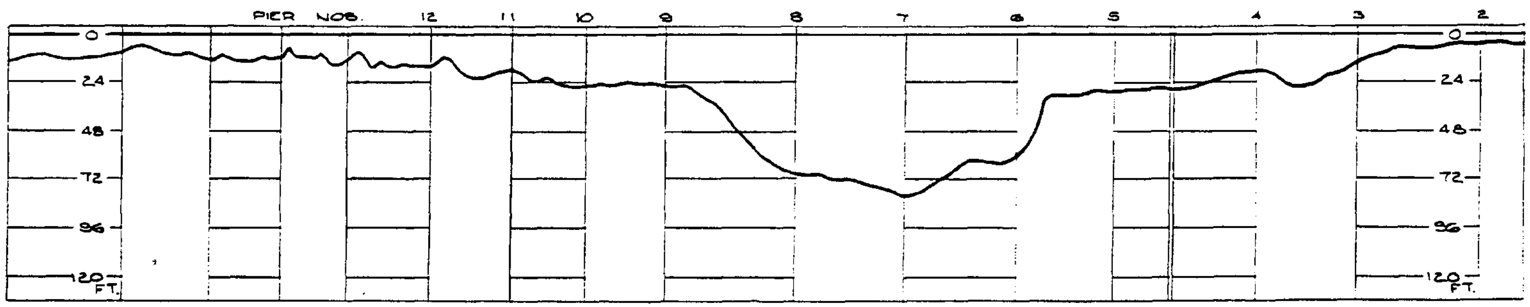


Figure 2. Plan view of existing and proposed bridges.



Existing railroad bridge (built 1924).



Existing highway bridge (built 1921).

Figure 3. Cross sections at existing bridge.

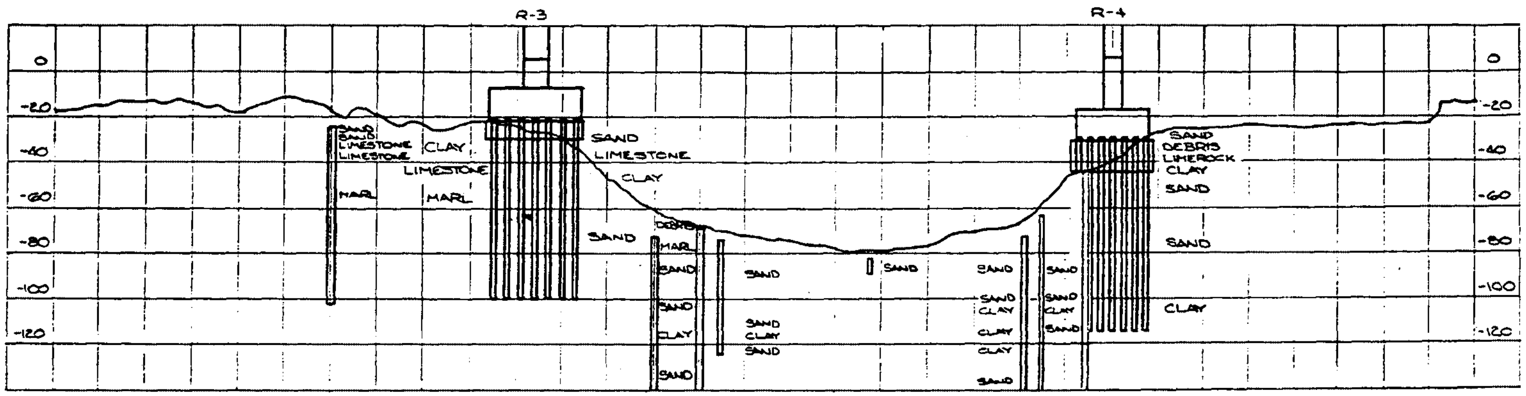


Figure 4. Cross section at proposed concrete alternative bridge.

3. General Procedures

Undistorted wooden scale models of various pier configurations were placed in a sump filled with sand within the flume. The model scale was restricted by the size of the flume. The scale of 1:50 was based on a maximum flume flow depth of about 16 in and a river depth of 70 ft. Depth changes in the prototype were not reproduced in the model. Table 1 includes the prototype to model ratios which are based on the Froude number. The 6 ft wide flume restricted how much of the river cross section could be modeled at a given time.

Table 1. Model parameter ratios.

Parameter	Ratio (Prototype:Model)
Length (Depth)	50:1
Velocity	7.1:1
Unit Flow	354:1
Riprap Size	50:1

Sand was selected as the bed material because it was available and would yield generally applicable information. However, since the bed material is a more complex composite material, as shown in figure 4, quantitative site-specific conclusions regarding absolute depth and extent of scour are limited. The nonuniformity of the bed material also indicates the inadequacy of lightweight model material. The approach to the sump in the flume was plywood with sand glued to the surface so there was continuity in the bottom roughness at the sump. Since there was no method for recirculating sand within the flume, the shear stress at the bed was kept near the critical shear stress (T_c) as shown in figure 5. If the shear stress were higher than the critical shear stress, the sand would wash out of the sump. With an upstream supply of sand, sand would wash into the scour hole as fast as it would wash out. For this reason, the depth of scour is nearly constant for shear stresses over the critical level. A second (larger) sand size was used for a few experiments to ensure that depth of scour is independent of particle size as long as shear stress is at the critical level. The grain size distribution curves for the two sands are given in appendix A.

The flow at which the experiments were run was determined as the minimum flow at which the sand particles just began to move with the given flow depth (about 16 in). Incipient motion is indicative of critical shear stress at the bed. After this threshold flow was determined, a pier was placed into the sand and the time for full development of the scour hole was measured. This time, 4 hours, was the time over which all experiments were run. The flume flow is

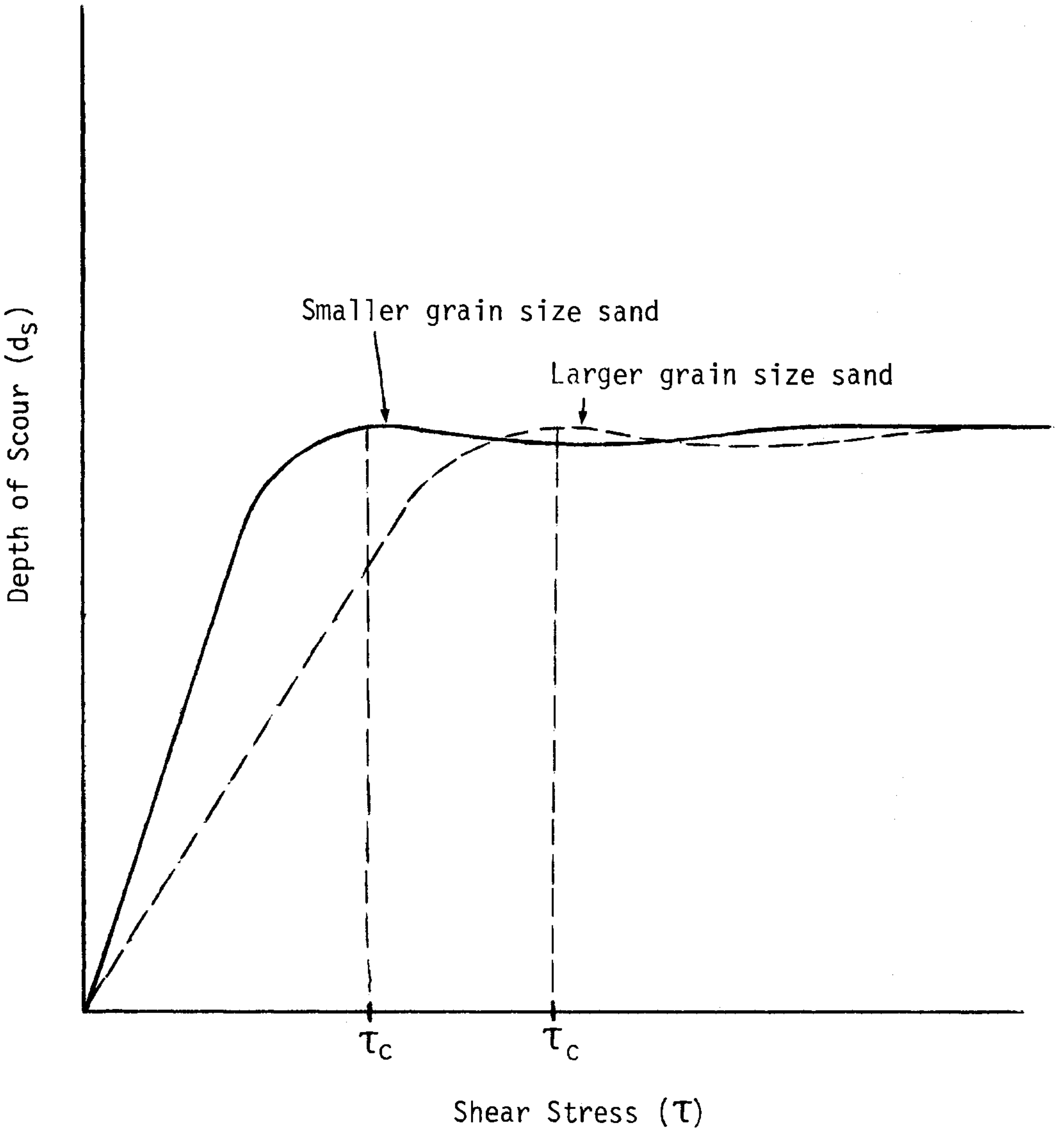


Figure 5. Shear stress versus depth of scour.

unidirectional, unlike the flow in the prototype, so the models were placed so that the flow traveled in the tidal flood direction. This is considered the critical direction for the analysis because it is during the flood flows that the proposed bridge could alter scour at the existing bridges. The two photographs in figure 6 show a preliminary setup with existing highway piers 7 and 8 and railroad pier 9 during and after the experiment.

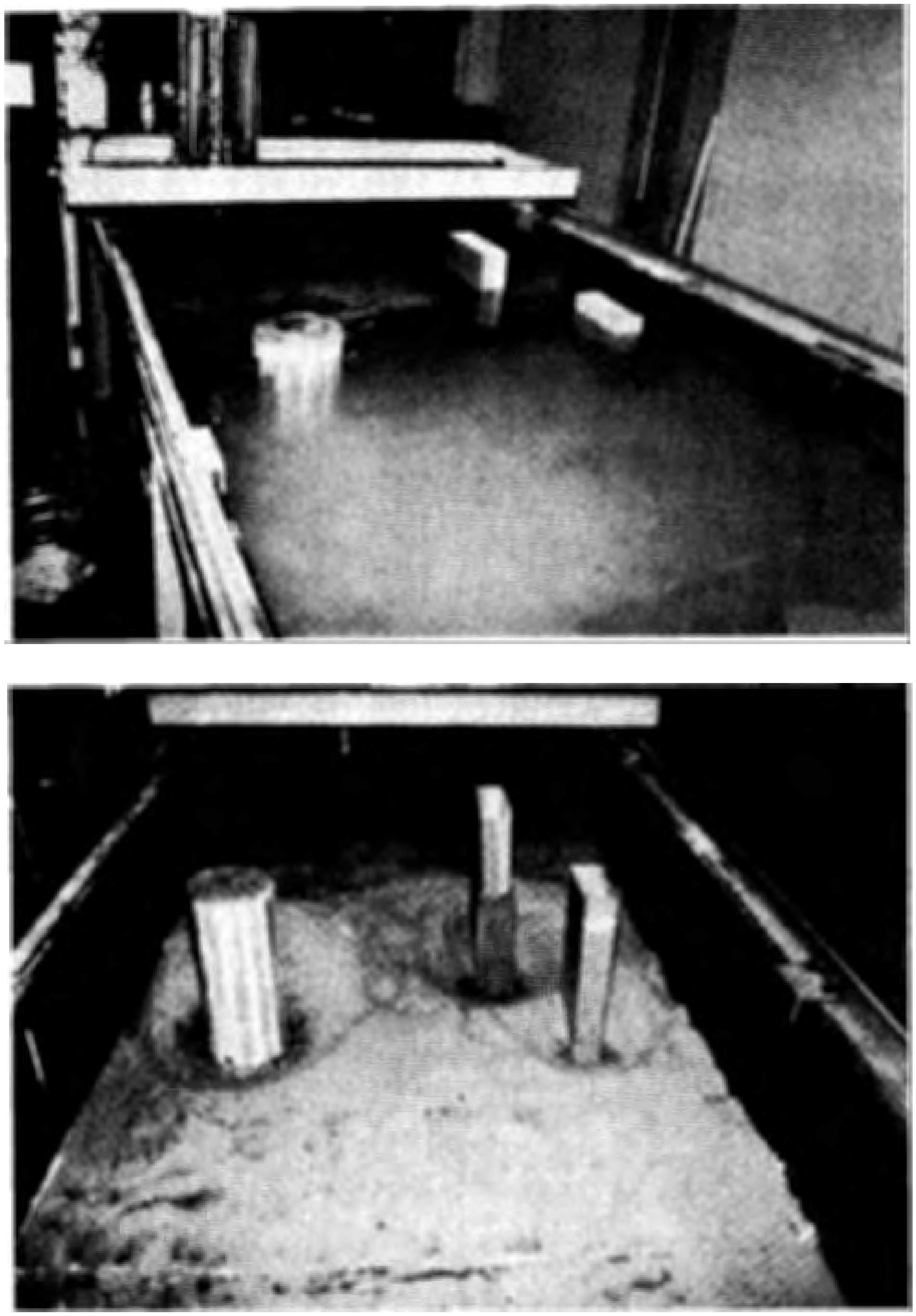


Figure 6. Flume during and after an experiment.

4. Data Acquisition and Results

A list of 36 experiments was compiled in cooperation with the Florida Department of Transportation. This list, along with objectives for individual experiments are included in table 2. The pier configuration for each experiment was centered between the flume sidewalls to minimize the effects of the flow contraction with the sidewalls. Figures 7 through 12 show various model pier configurations within the flume (these are the actual configurations for runs 8, 2a, 7a, 3a, 3b, and 5, respectively). Figure 7 also shows typical locations at which velocity readings were taken.

After the pump valves were adjusted to the proper flow - 7.0 cubic feet per second (ft^3/s) for all runs except 13 through 15b - velocity readings were taken. First, a grid of approach velocities, 5 depths at 5 locations for 25 readings, was taken at an adequate distance in front of the piers to ensure that velocities were not influenced by the piers. This distance was 1.5 ft for most runs, but as much as 2.5 ft for runs which included very wide pier models. The 25 readings were averaged to give a baseline approach velocity for each run. The average approach velocity, as expected, was about 0.9 ft per second (ft/s) (velocity = flow/area = $(7\ ft^3/s)/8\ ft^2 = 0.9$ ft/s). This corresponds to a prototype mean velocity of 6.4 ft/s.

Velocities were then measured at about 0.5 ft in front of the piers (the pitot tube could not get closer), where velocities are low, and maximum velocities were taken at the piers by trial and error of placement and angle of the pitot tube. The high and low velocities were taken at five depths and were compared to the average approach velocity. These velocities were plotted and are included in appendix B.

The extent of scour was measured at the front and both sides, as shown in figure 13, and the maximum depth of the scour hole was taken at the front of each pier which was buried in sand for each run. These scour measurements are included in table 3. It should be reemphasized that the absolute scour depths cannot be transferred to the prototype and may not be representative of the prototype scour. Many of the run configurations which included both existing and proposed piers could not fit entirely within the sand-filled sump. For these runs, the proposed piers were attached to the plywood approach to the sump and scour measurements could not be taken at these piers. In these instances, scour at existing piers was of more concern and these measurements were taken.

Table 2. List of scour study experiments.

Run Number	Objective	Configuration
Site-Specific		
1a	Determine if scour at RR pier 9 is increased by the highway piers; conversely, will scour be reduced by removing highway piers.	Existing Acosta (highway) Bridge piers 7 and 8 and Railroad (RR) Bridge pier 9.
1b	Same as 1a above.	RR 9.
2a	Base line experiment in a series to determine effects of proposed structure on the RR piers.	RR 7 and 8.
2b	Same as 2a with different approach angle.	Repeat 2a with flow oriented at 15° to pier alignment.
3a	Second experiment in a series to determine if it would be better to remove the existing highway piers or leave them.	RR 7 and 8 plus existing highway 5 and 6.
3b	Same as 3a with different approach angle.	Repeat 3a with flow direction oriented at 15° to pier alignment.
3c	Determine if scour is influenced by the pier height.	Repeat 3b with the highway piers cut off to ¼ the flow depth; flow direction at 15°.
4	Determine if the proposed piers will have a significant effect on the RR and existing highway piers.	RR 7 and 8 plus existing highway 5 and 6 plus front half of proposed steel alternative pier (the downstream half in the prototype but the upflow half in the flume) with pile cap on bed.
5	Same as 4 above.	RR 7 and 8 plus existing 5 and 6 plus both halves of proposed steel alternative pier.

Table 2. List of scour study experiments (continued).

Run Number	Objective	Configuration
Site-Specific		
6	Extraneous run.	RR 7 <u>plus</u> both halves of proposed steel alternative pier.
7a	First in a set of two to determine how far out from the piers the effect of the proposed piers will extend.[1]	RR 8 <u>plus</u> both halves of proposed steel alternative pier. Set proposed piers with footers at stream bed elevation.
7b	Same as 7a above.	RR 8 <u>plus</u> both halves of proposed steel alternative pier. Set proposed pier with footers at stream bed elevation and place RR 8 and proposed piers as far apart, transversely, as possible without getting too much side effects.
7c	Base line test for 7b.	RR 8 by itself in the location of 7b.
8	First of a set of three to determine effects of footer location.	Half of proposed steel pier by itself with footer at the waterline.
9a	Same as 8 above.	Same as 8 but with footer at mid-depth.
9b	Same as 8 above.	Same as 8 but with bottom of footer at the stream bed elevation.

[1]If RR pier 9, which is the most vulnerable, might be endangered by the proposed piers, but the flume is not wide enough to model the space between RR pier 9 and the proposed piers. It is expected that RR pier 9 is far enough away not to be affected by the new piers and this can be demonstrate by setting RR 8 at two positions.

Table 2. List of scour study experiments (continued).

Run Number	Objective	Configuration
Site-Specific		
10	Same as 7a but with piles only exposed to water.	RR 8 plus both halves of proposed steel alternative pier with footer at water line. Set RR 8 plus both halves of proposed steel alternative pier. Set proposed pier with footer at water line. Set RR 8 and the proposed pier at normal positions transversely as with run 7a.
11	Same as 7b but with piles only exposed to water.	RR 8 plus both halves of proposed steel alternative pier set with footers at waterline. Set piers at transverse locations the same as in 7b.
12a	Determine if the temporary structure will affect scour at either pier.	RR 9 plus existing highway 7 plus one temporary structure.
12b	Base line for 12a.	RR 9 plus existing highway 7.
16	Determine effects of flow concentrations from concrete pier on scour at the RR and existing highway piers. Not concerned about scour at the new concrete pier for this test.	RR 7 and 8 plus highway 5 and 6 plus one half concrete alternative pier with footer at stream bed elevation.
17	Determine effects of new concrete pier on the RR pier scour when the existing highway piers are removed. Compare with runs 2 and 3.	RR 7 and 8 plus both halves of concrete alternative pier set with footers at stream bed elevation.

Table 2. List of scour study experiments (continued).

Run Number	Objective	Configuration
Rip Rap		
13	Bracket design velocities.	RR 9 by itself with larger sand (higher velocity).
14	Determine riprap stability at higher velocity.	RR 9 with larger sand (higher velocity) with 3/8 in model riprap extended to 10 in around pier.
15a	Same as 14.	RR 9 with larger sand (higher velocity) with 1/4 in model riprap extended to 10 in around pier.
15b	Demonstrate what happens when riprap is not extended far enough.	RR 9 with larger sand with 1/4 in model riprap extended to 5 in around pier.
Footer Location		
18	First of a set of experiments to check effects of footer location.	One half of concrete pier by itself with footer at the waterline.
19	Same as 18 above.	Same as 18 but with footer at mid-depth.
20a	Same as 18 above.	Same as 18 but with bottom of footer at the stream bed.
20b	Same as 18 above.	Same as 18 but with top of footer at the stream bed.

Table 2. List of scour study experiments (continued).

Run Number	Objective	Configuration
Pile Spacing		
21a	First of a set to determine effects of pile spacing and to compare scour at pile groups versus equivalent width rectangular piers.	Modified concrete alternative pier with pile spacing with 1.0 pile diameter between piles. Footer at waterline.
21b	Base line for 21a. Pier length equal twice the width.	Rectangular pier with width equal to total width of piles in 21a.
22a	Same as 21a above.	Same as 21a but with 2.0 pile diameters between piles.
22b	Same as 21a above.	Rectangular pier with width equal to total width of piles in 22a.
23a	Same as 21a above.	Same as 21a but with 0.5 pile diameters between piles.
23b	Same as 21a above.	Rectangular pier with width equal to total width of piles in 23a.

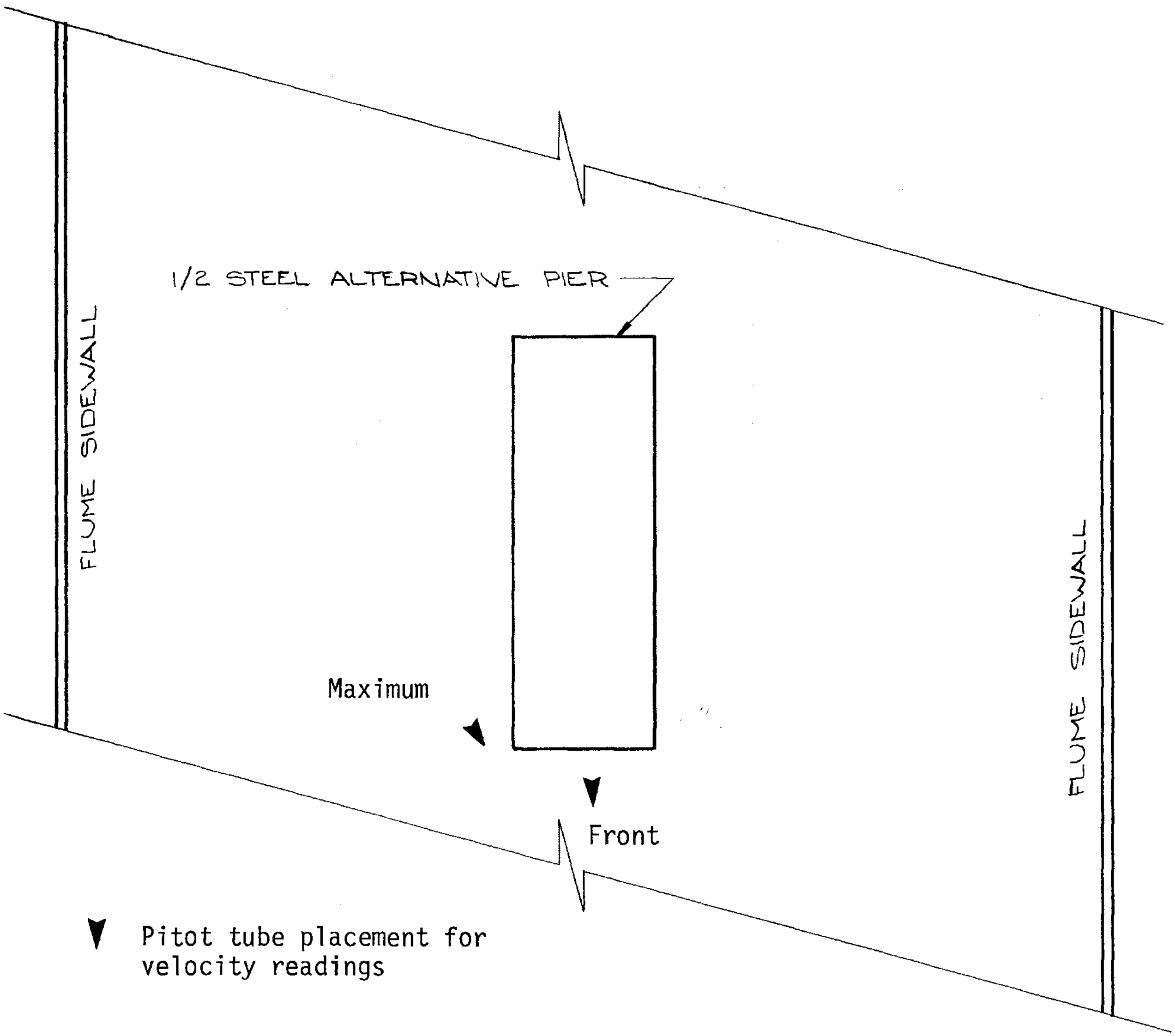


Figure 7. One model flume configuration.

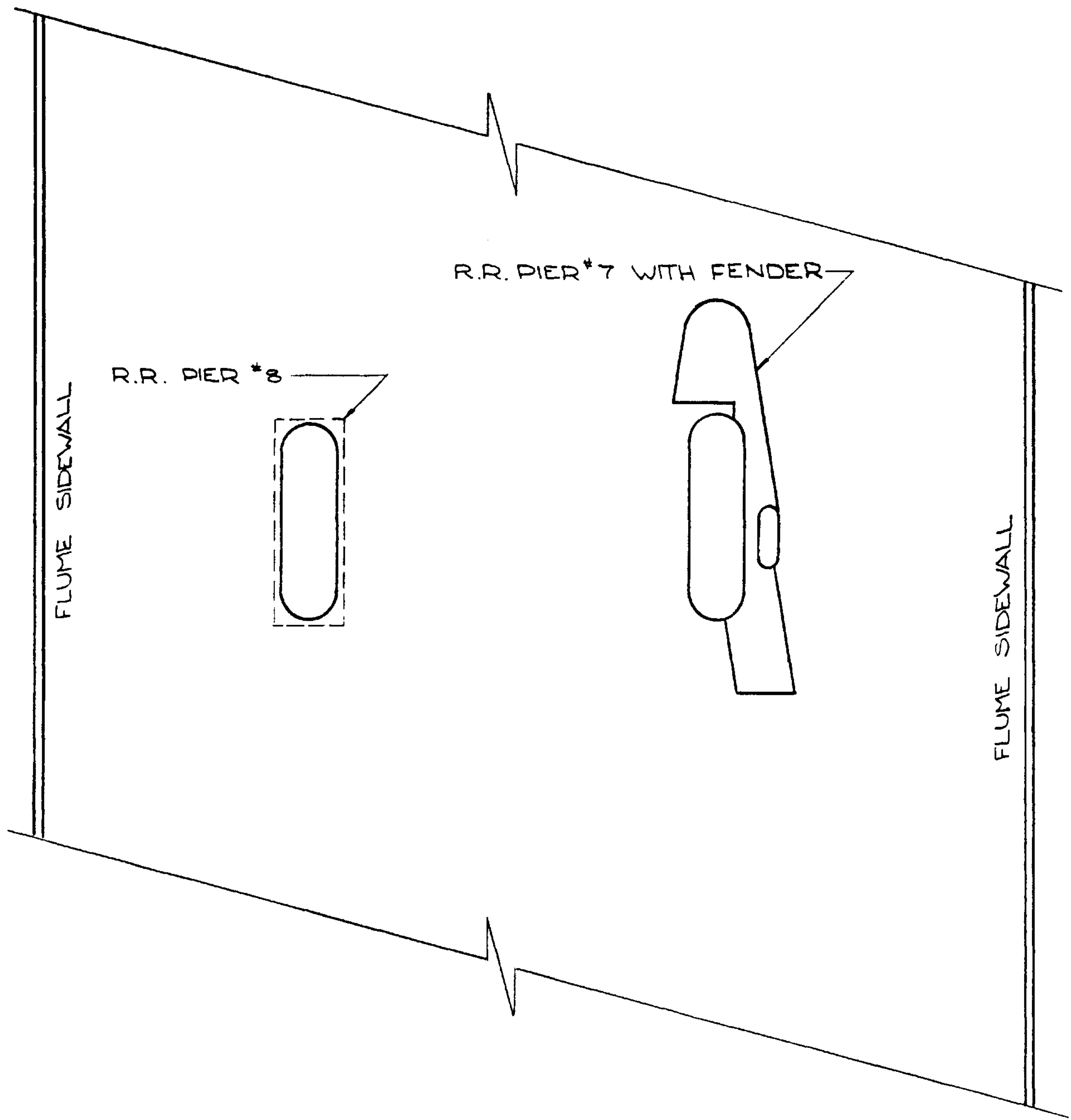


Figure 8 . Two model flume configuration.

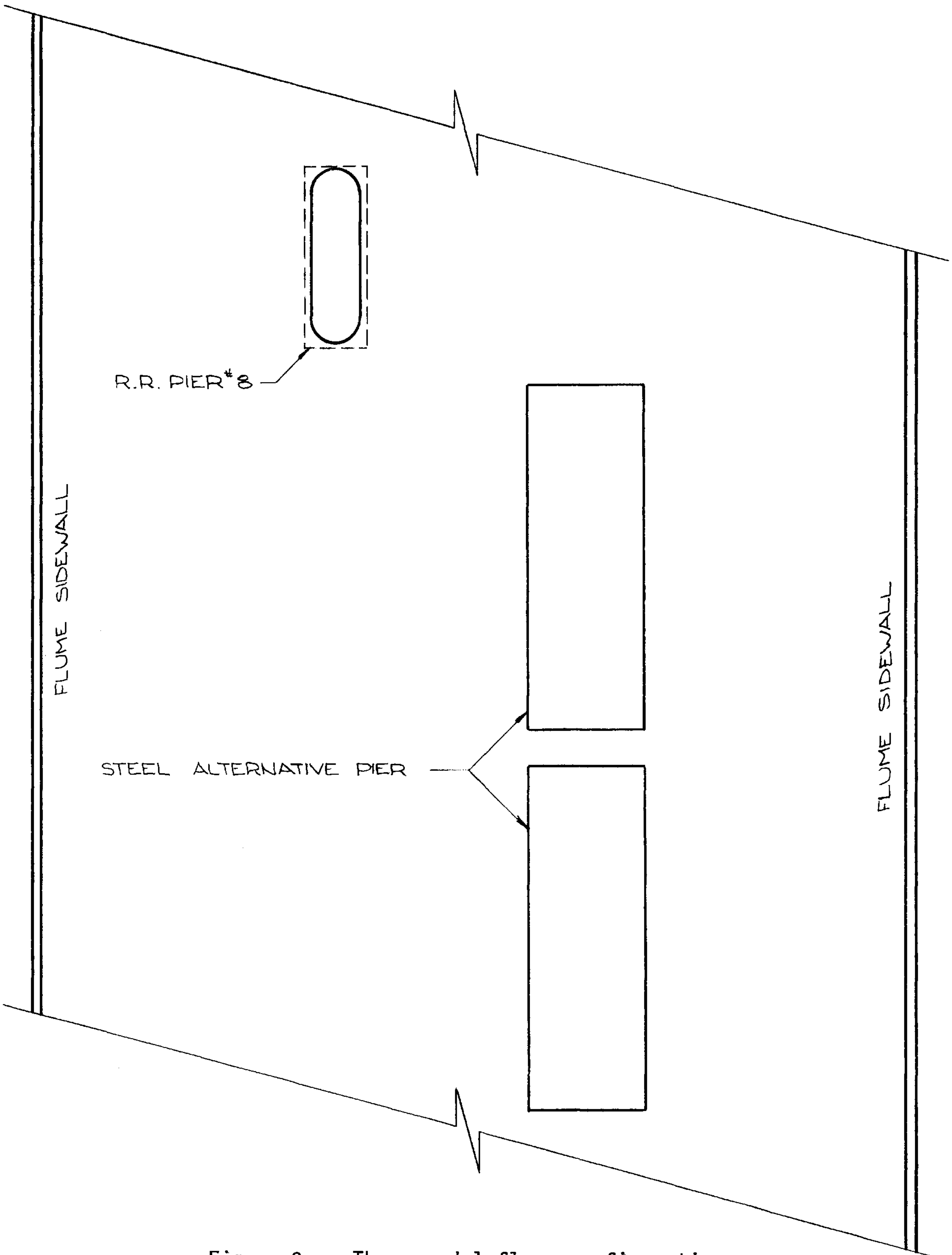


Figure 9 :. Three model flume configuration.

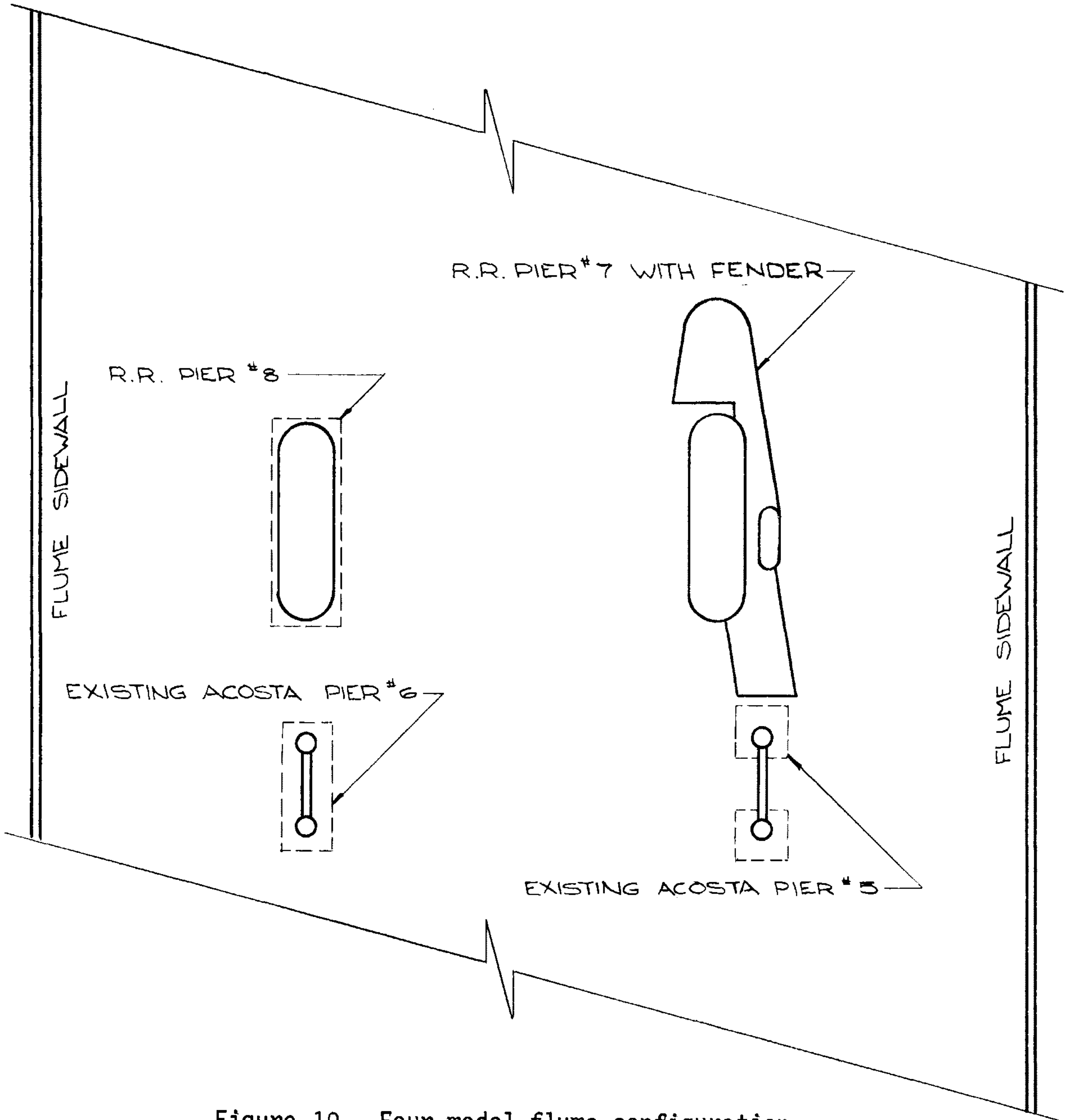


Figure 10. Four model flume configuration.

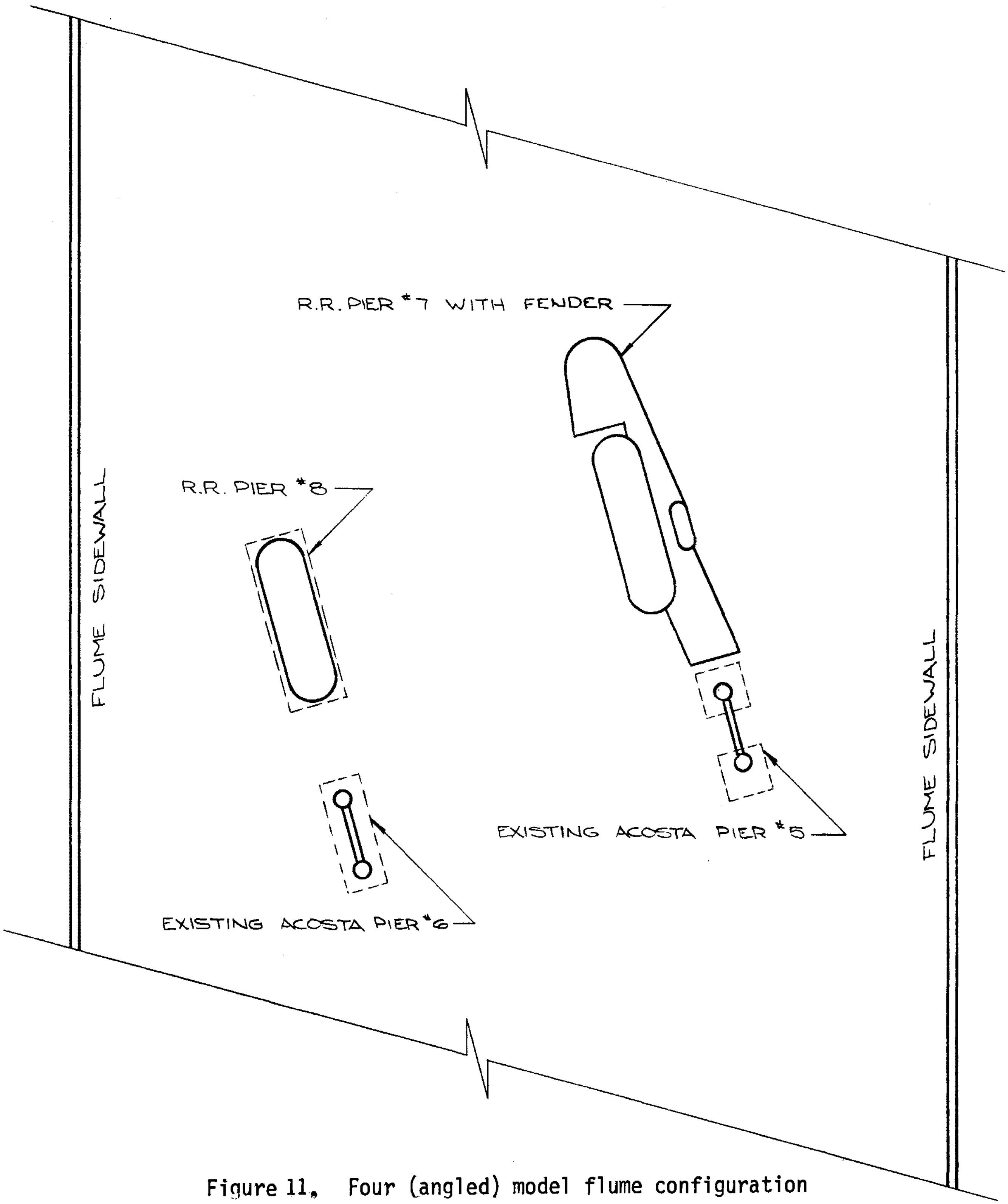


Figure 11. Four (angled) model flume configuration

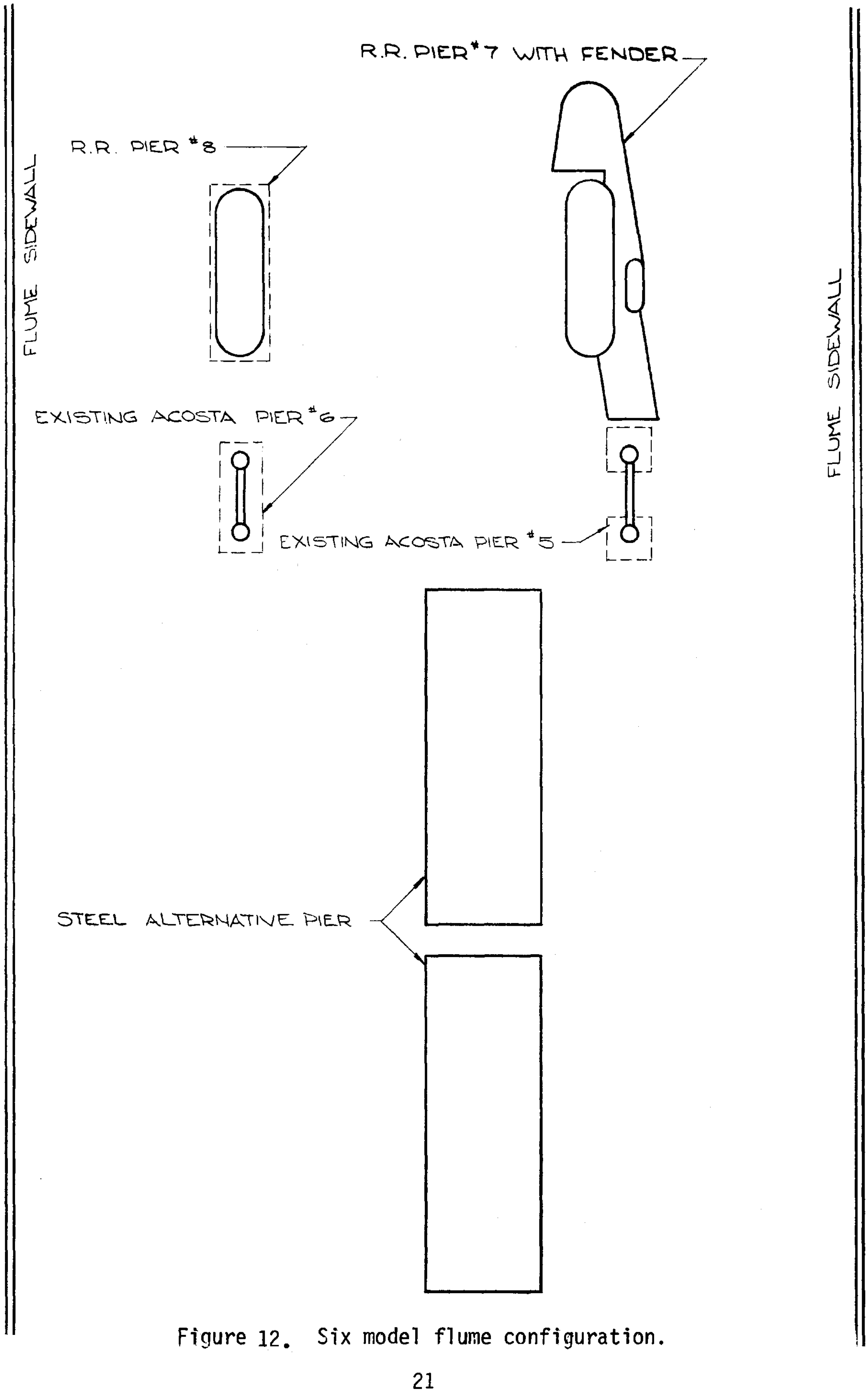
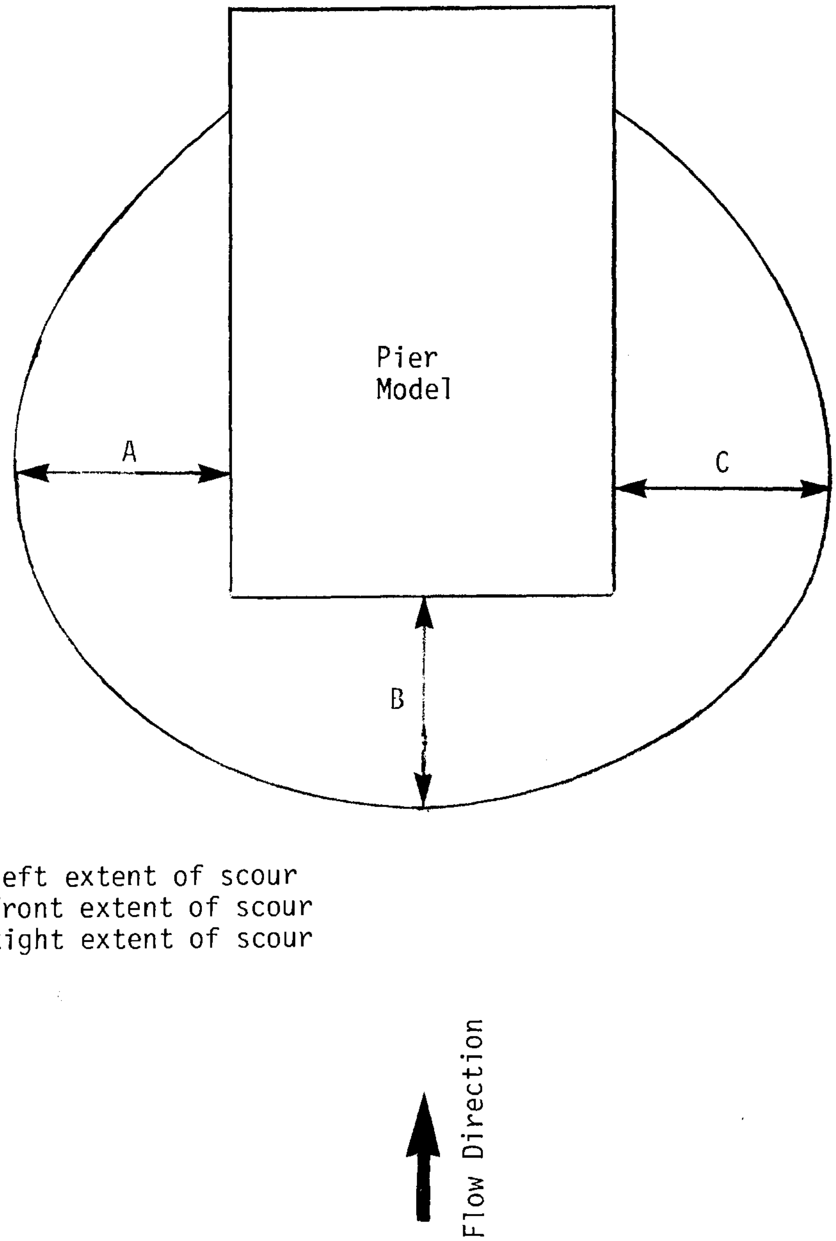


Figure 12. Six model flume configuration.



A - Left extent of scour
B - Front extent of scour
C - Right extent of scour

Figure 13. Extent of scour hole.

Table 3. Scour measurements.[1]

Run No.	Configuration	Pier No.	Pier Width Prototype (ft)	Pier Width Model (ft)	Scour Depth (ft)	Extent of Scour (ft) A Left	Extent of Scour (ft) B Front	Extent of Scour (ft) C Right
Scour runs:								
1a	Existing Acosta (highway) Bridge piers 7 and 8 and Rail-road bridge pier 9.	Highway 7	16	0.32	0.30	0.54	0.67	0.54
		Highway 8	40	0.80	0.33	0.79	0.85	0.79
		RR 9	22	0.44	0.36	0.67	0.88	0.92
1b	RR9	RR9	22	0.44	0.32	Measurements not taken.		
2a	RR 7 and 8.	RR 7	19.5	0.39	0.33	0.62	0.75	0.67
		RR 8	19.5	0.39	0.30	0.58	0.67	0.52
2b	Repeat 2a with flow oriented at 15° to pier alignment.	RR 7	19.5	0.39	0.34	0.67	0.62	0.75
		RR 8	19.5	0.39	0.34	0.58	0.62	0.62
3a	RR 7 and 8 plus existing high-way 5 and 6.	Highway 5	18	0.36	0.38	0.50	0.79	0.92
		Highway 6	16	0.32	0.32	0.50	0.67	0.50
		RR 7	19.5	0.39	0.40	0.58	0.83	--[2]
		RR 8	19.5	0.39	0.24	0.33	--	0.33
3b	Repeat 3a with flow direction oriented at 15° to pier alignment.	Highway 5	18	0.36	0.54	0.75	0.88	--
		Highway 6	16	0.32	0.26	0.54	0.58	0.67
		RR 7[3]	19.5	0.39	0.56	0.67	--	1.08
		RR 8	19.5	0.39	0.40	0.58	--	0.75
3c	Repeat 3b with the highway piers cut off to 1/4 the flow depth; flow direction at 15°.	Highway 5	18	0.36	0.27	0.46	0.67	0.58
		Highway 6	16	0.32	0.16	0.33	0.46	0.46
		RR 7	19.5	0.39	0.34	0.38	0.54	0.58
		RR 8	19.5	0.39	0.31	0.38	0.58	0.62

[1]Flow depth for all runs was 16 in (approximately 70 ft prototype). Normal mean velocities were 0.9 ft/s (7 ft/s prototype) except for runs 13-15. For these runs, normal mean velocities were 1.4 ft/s (10 ft/s prototype).
[2]Not definable
[3]Ran without fender

Table 3. Scour measurements (continued).

Run No.	Configuration	Pier No.	Pier Width Prototype (ft)	Pier Width Model (ft)	Scour Depth (ft)	Extent of Scour (ft) A Left	Extent of Scour (ft) B Front	Extent of Scour (ft) C Right
4	RR 7 and 8 plus existing highway 5 and 6 plus front half of proposed steel alternative pier (the downstream half in the prototype but the upflow half in the flume) with pile cap on bed.	Highway 5	18	0.36	0.36	0.67	0.79	0.75
		Highway 6	16	0.32	0.32	0.62	0.64	0.54
		RR 7	19.5	0.39	0.30	0.46	--	0.58
		RR 8	19.5	0.39	0.12	0.33	--	0.29
5	RR 7 and 8 plus existing 5 and 6 plus both halves of proposed steel alternative pier.	Highway 5	18	0.36	0.47	--	0.90	0.92
		Highway 6	16	0.32	0.37	0.60	0.75	0.54
		RR 7	19.5	0.39	0.46	0.79	0.58	--
		RR 8	19.5	0.39	0.21	0.42	0.48	0.42
6	RR 7 plus both halves of proposed steel alternative pier.	RR 7	19.5	0.39	0.32	0.75	0.79	0.69
7a	RR 8 plus both halves of proposed steel alternative pier. Set proposed piers with footers at stream bed elevation.	RR 8	19.5	0.39	0.32	0.50	0.62	0.58
7b	RR 8 plus both halves of proposed steel alternative pier. Set proposed pier with footers at stream bed elevation and place RR 8 and proposed piers as far apart, transversely, as possible without getting too much side effects.	RR 8	19.5	0.39	0.32	0.54	0.62	0.58
7c	RR 8 by itself in the location of 7b.	RR 8	19.5	0.39	0.29	0.46	0.62	0.54
8	Half of proposed steel pier by itself with footer at the waterline.	Proposed Steel	43	0.86	0.28	0.33	0.35	0.33
9a	Same as 8 but with footer at mid-depth.	Proposed Steel	43	0.86	0.22	Measurements not taken.		

Table 3. Scour measurements (continued).

Run No.	Configuration	Pier No.	Pier Width Prototype (ft)	Pier Width Model (ft)	Scour Depth (ft)	Extent of Scour (ft) A Left	Extent of Scour (ft) B Front	Extent of Scour (ft) C Right
9b	Same as 8 but with bottom of footer at the stream bed elevation.	Proposed Steel	43	0.86	0.20	0.42	0.38	0.38
10	RR 8 <u>plus</u> both halves of proposed steel alternative pier. Set proposed pier with footer at water line. Set RR 8 and the proposed pier at normal positions, transversely, as with run 7a.	RR 8	19.5	0.39	0.35	0.58	0.79	0.69
11	RR 8 <u>plus</u> halves of proposed steel alternative pier set with footers at water line. Set piers at transverse locations the same as in 7b.	RR 8	19.5	0.39	0.40	0.77	0.88	0.71
12a	RR 9 <u>plus</u> existing highway 7 <u>plus</u> one temporary structure.	Highway 7	16	0.32	0.30	0.46	0.58	0.48
		RR 9	22	0.44	0.32	0.58	0.67	0.73
12b	RR 9 <u>plus</u> existing highway 7.	Highway 7	16	0.32	0.29	0.46	0.58	0.48
		RR 9	22	0.44	0.32	0.58	0.67	0.73
16	RR 7 and 8 <u>plus</u> highway 5 and 6 <u>plus</u> one half concrete alternative pier with footer at stream bed elevation.	Highway 5	18	0.36	0.42	0.67	0.73	0.62
		Highway 6	16	0.32	0.34	0.58	0.71	0.58
		RR 7	19.5	0.39	0.28	0.46	0.50	--
		RR 8	19.5	0.39	0.18	0.29	0.38	0.42
17	RR 7 and 8 <u>plus</u> both halves of concrete alternative pier set with footers at stream bed elevations.	RR 7	19.5	0.39	0.34	--	0.79	--
		RR 8	19.5	0.39	0.44	0.71	0.75	0.75
Riprap runs								
13	RR 9 by itself with larger sand (higher velocity).	RR 9	22	0.44	0.34	0.79	0.79	0.79

Table 3. Scour measurements (continued).

Run No.	Configuration	Pier No.	Pier Width Prototype (ft)	Pier Width Model (ft)	Scour Depth (ft)	Extent of Scour (ft) A Left	Extent of Scour (ft) B Front	Extent of Scour (ft) C Right
14	RR 9 with larger sand (higher velocity) with 0.22 in (D_{50}) riprap extended to 10 in around pier.	RR 9	22	0.44	Riprap was stable.			
15a	RR 9 with larger sand (higher velocity) with 0.123 in (D_{50}) model riprap extended to 10 in around pier.	RR 9	22	0.44	Riprap moved at corners.			
15b	RR 9 with larger sand with 0.123 in (D_{50}) riprap extended to 5 in around pier.	RR 9	22	0.44	Riprap moved at corners.			
Footer location runs:								
18	One half of concrete pier by itself with footer at the waterline.	Proposed Concrete	80	1.60	0.38	0.46	0.38	0.42
19	Same as 18 but with footer at mid-depth.	Proposed Concrete	80	1.60	0.35	--	0.50	--
20a	Same as 18 but with bottom of footer at the stream bed.	Proposed Concrete	80	1.60	0.38	0.92	0.62	0.83
20b	Same as 18 but with top of footer at the stream bed.	Proposed Concrete	80	1.60	0.29	0.73	0.71	0.73
Pile spacing runs:								
21a	Modified concrete alternative pier with pile spacing with 1.0 pile diameter between piles. Footer at waterline.	Modified Concrete	80	1.60	0.38	0.50	0.46	0.50
21b	Rectangular pier with width equal to total width of piles in 21a.	Rectangular Pier	35	0.70	0.46	0.83	0.85	0.83

Table 3. Scour measurements (continued).

Run No.	Configuration	Pier No.	Pier Width Prototype (ft)	Pier Width Model (ft)	Scour Depth (ft)	Extent of Scour (ft) A Left	Extent of Scour (ft) B Front	Extent of Scour (ft) C Right
22a	Same as 21a but with 2.0 pile diameters between piles.	Modified Concrete	80	1.6	0.21	0.33	0.32	0.31
22b	Rectangular pier with width equal to total width of piles in 22a.	Rectangular Pier	25	0.50	0.38	0.67	0.86	0.67
23a	Same as 21a but with 0.5 pile diameters between piles.	Modified Concrete	80	1.6	0.33	*[4]	0.58	*
23b	Rectangular pier with width equal to total width of piles in 23a.	Rectangular Pier	45	0.90	0.42	*	1.11	*
Reruns of pile spacing runs:								
R21a	Rerun of 21a with fine uniform sand	Modified Concrete	80	1.6	0.33	0.58	0.54	0.50
R21b	Rerun of 21b with fine uniform sand	Rectangular Pier	35	0.70	0.67	1.08	1.14	1.12
R22a	Rerun of 22a with fine uniform sand	Modified Concrete	80	1.60	0.28	0.37	0.37	0.35
R22b	Rerun of 22b with fine uniform sand	Rectangular Pier	25	0.50	0.61	0.86	1.06	0.89
R23a	Rerun of 23a with fine uniform sand	Modified Concrete	80	1.60	0.42	0.70	0.64	0.75
R23b	Rerun of 23b with fine uniform sand	Rectangular Pier	45	0.90	0.76	1.45	1.43	1.45

[4]Scour extended to flume sidewall

Site-Specific Scour Runs

Most of the runs were performed to evaluate scour issues specific to the Acosta Bridge site. The scour measurements from runs 1a and 1b indicate that scour depth is increased (10 percent) at railroad pier 9 with the presence of existing highway piers 7 and 8.

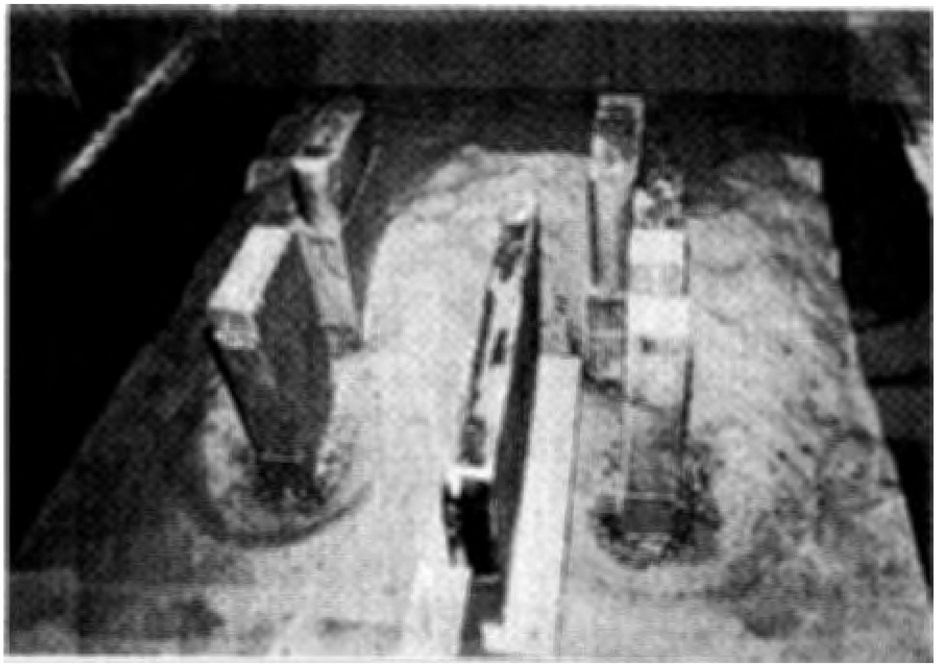
In comparing the scour measurements from run 2a and run 3a, it appears that scour at railroad pier 7 is hardly influenced by the presence of the existing highway piers whereas scour at railroad pier 8 is lessened with the presence of the existing highway piers (a 20 percent scour depth reduction). This might be because existing highway pier 6 is directly in front of railroad pier 8 and, thus, provides some scour protection, while existing highway pier 6 is not directly in front of railroad pier 7.

With a 15 degree pier alignment (runs 2b and 3b) to simulate the curvature of the St. Johns River at this location, scour at the railroad piers is by the existing highway piers. Railroad pier 7 exhibited a 60 percent increase in depth and an increase in extent on the right side, and railroad pier 8 exhibited a 20 percent increase in depth but no increase in extent with the presence of the existing highway piers. Scour cannot be compared for these runs at railroad pier 7 because the fender was not used in run 3b. The 15 degree alignment results in a smaller effective cross-sectional area for flow which causes higher velocities by the piers which should increase the scour. With the 15 degree pier orientation, cutting the existing highway piers to 25 percent of the flow depth (run 3c) reduced scour depth by 15 percent at railroad pier 7, but increased scour depth by approximately 30 percent at railroad pier 8.

The presence of half of the steel alternative pier (run 4) had no significant affect on scour at the existing highway piers and decreased scour at the railroad piers compared to run 3a. Scour depth decreased by 25 percent at railroad pier 7 and 50 percent at railroad pier 8, but scour extent did not change. The presence of both halves of the steel alternative pier (run 5) slightly increased scour at the existing highway piers and railroad pier 7 and had no significant effect on scour at railroad pier 8. Scour depth increased by approximately 25 percent at existing highway pier 5, 15 percent at existing highway pier 6, 15 percent at railroad pier 7, and decreased by 10 percent at railroad pier 8. Figure 14 shows a front and side view of the scour holes which were developed in run 5. The white lines toward the bottom of the piers represent the sand line before the experiment was run. Run 6 was an extraneous run that was not analyzed.

The results of runs 7a, 7b, and 7c indicate that the proposed steel pier should not affect scour at railroad pier 8. The depth and extent of scour was virtually the same at railroad pier 8 for the three runs, with the depth of scour being very slightly smaller (approximately 10 percent less scour depth) without the presence of the proposed steel pier.

There were contradictory results in determining the effect of the footer elevation on scour at the proposed steel pier (runs 8, 9a, and 9b). The depth of scour decreased approximately 30 percent and the extent of scour increased as the footer was moved from the waterline to the stream bed elevation.



Front view of run 5

Side view of run 5

Figure 14. Views of a six model configuration (run 5).

The depth of scour around railroad pier 8 actually increased as this pier was set further away from the proposed steel pier (runs 10 and 11). By setting the piers further apart transversely, a flow contraction might have been created between the piers and the flume sidewalls which could result in more scour. This phenomenon would not occur in the prototype, and the results were expected to be similar to those of runs 7a and 7b. Therefore, runs 10 and 11 are inconclusive with regard to the expected effect of the proposed piers on railroad pier 8.

As expected, the results of runs 12a and 12b indicate that the temporary structure should not affect scour at existing highway pier 7 nor railroad pier 9. The temporary structure has only three 2 ft piles projecting in the direction of flow, which is minimal considering it is over 100 ft from existing highway pier 7.

The presence of half of the concrete alternative pier (run 16) resulted in only slightly more scour around existing highway piers 5 and 6 (an increase of approximately 10 percent and 5 percent in scour depth, respectively) and less scour around railroad piers 7 and 8 (a decrease of approximately 30 percent and 25 percent in scour depth, respectively). This might be the result of a contraction being formed and flow being channeled between the concrete alternative pier and the existing highway piers to an area between the railroad piers, as shown in figure 15, instead of alongside the existing piers. Half of the concrete alternative pier had virtually the same effect on scour at existing highway piers 5 and 6 and railroad piers 7 and 8 as half of the steel alternative pier (run 4).

A comparison of runs 17 and 2a indicates that the concrete alternative pier has no influence on scour at railroad pier 7 and increases the scour at railroad pier 8. The depth of scour increased 45 percent at railroad pier 8. A similar comparison between runs 17 and 3a indicates that there is slightly less scour around railroad pier 7 (a decrease of 15 percent in scour depth) and much more scour around railroad pier 8 (an increase of 85 percent in scour depth) with the concrete alternative pier than with existing highway piers 5 and 6.

Footer Location Runs

Runs 18 through 20b investigate the effects of footer location, as shown in figure 16, on scour. The location of the footer of the concrete alternative pier greatly influenced the extent of scour but did not have much of an affect on the depth of scour as long as the footer was on or above the bed (runs 18, 19, 20a, and 20b). The extent of scour doubled as the bottom of the footer was lowered from the waterline to the bed surface, while scour depth remained the same. The scour depth decreased approximately 25 percent when the footer was lowered further so that the top of the footer was at the bed surface. This also slightly decreased the extent of scour, although the extent of scour was greater than that experienced with the footer at the water surface or at mid-depth.

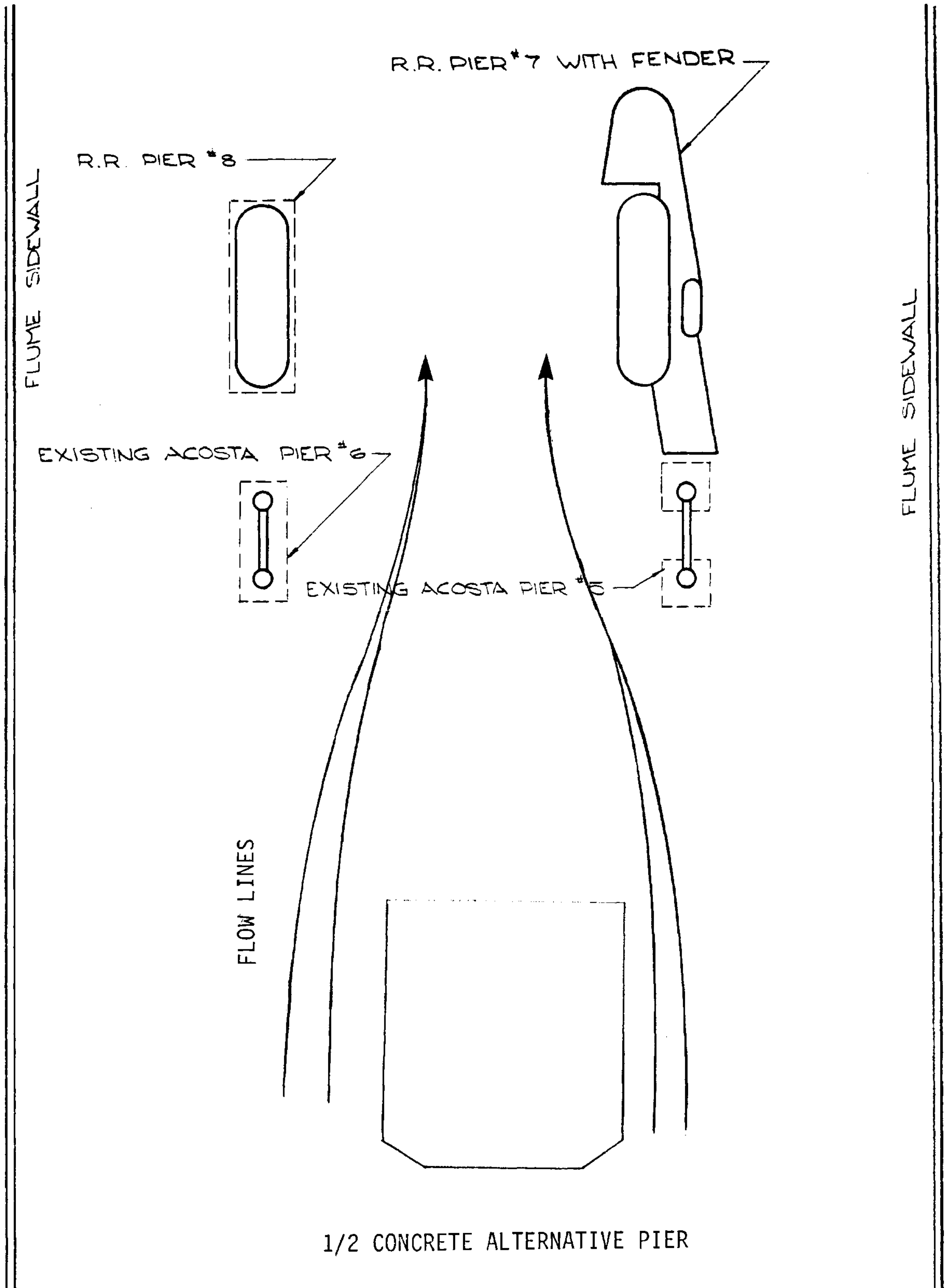
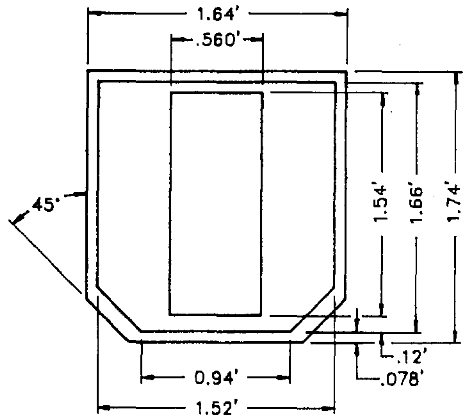
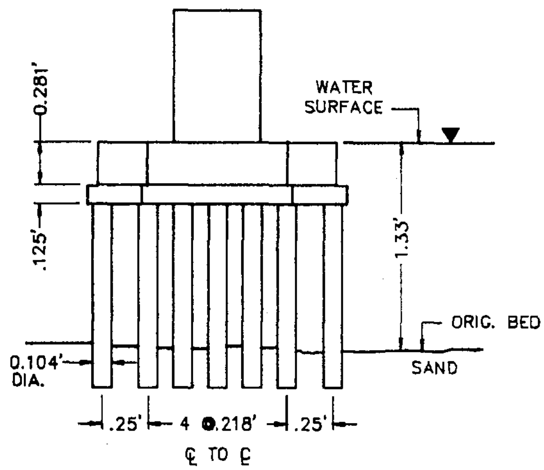


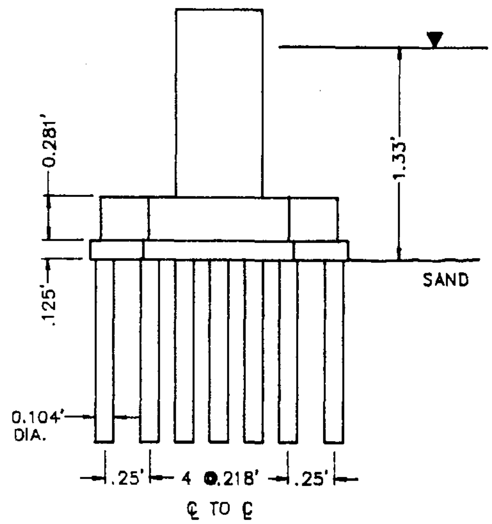
Figure 15. Possible flow contraction in run 16.



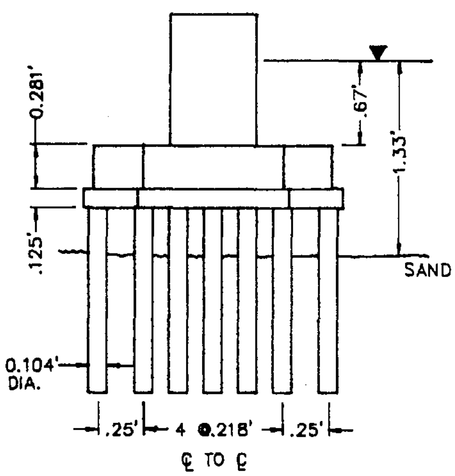
PLAN VIEW



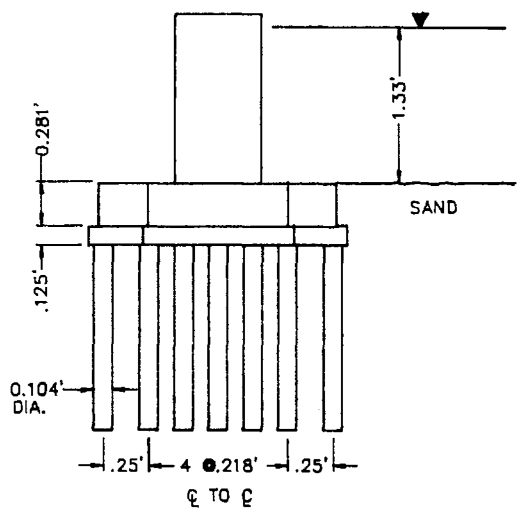
A. ELEV. VIEW FOOTER AT WATER SURFACE. TEST RUN 18.



C. ELEV. VIEW BTM. OF FOOTER AT BED LEVEL. TEST RUN 20A.



B. ELEV. VIEW FOOTER AT MID DEPTH TEST RUN 19.



D. ELEV. VIEW TOP OF FOOTER AT BED LEVEL. TEST RUN 20B.

Figure 16. Footer locations used in test runs 18,19, & 20.

Pile Spacing Runs

Experiments were performed to compare scour at pile groups versus scour at equivalent width rectangular piers, and also to investigate the effects of pile spacing on scour. The results of runs 21a and 21b, 22a and 22b, and 23a and 23b indicate that assuming a rectangular pier width equal to the net total width of piles obstructing the flow is a conservative assumption for estimating scour for pile groups. For example, run 21a had seven piles with a center to center spacing of 2.0 pile diameters while run 21b had an equivalent rectangle with a width of 7.0 pile diameters. Run 23a had 9 piles at 1.5 pile diameter spacing, center to center, while run 23b had an equivalent rectangle with a width of 9 pile diameters. Run 22 had a center to center spacing of 3.0 pile diameters and the piles started acting independently rather than as a mass. At the larger spacing, the piles would act like independent piers in a crossing, but the series was not extended that far. Test series 21 through 23 were rerun because some of the results were inconsistent. The reruns are labeled R21a through R23b, where the "R" prefix indicates a rerun. The bed material used for the original runs was no longer available so a different sand was used to the reruns. The original bed material had D_{50} equal to 0.48 mm; the bed material used for the reruns had D_{50} equal to 0.36 mm. Results from both sets of experiments showed that it would be conservative to assume a rectangular pier width equal to the total projected width of the pile group ignoring the space between piles to estimate scour for a pile group. The results of the reruns showed scour from a pile group to be less than 60 percent of the scour for a corresponding "equivalent" rectangular pier. For the range tested, the greater the pile spacing, the lower the ratio of scour for the pile group to scour for the "equivalent" rectangular pier. Two precautions need to be noted. First, debris could make a group of piles act as a bigger obstruction to the flow; second, staggered rows of piles effectively increase the equivalent rectangle width. The rows of piles were not staggered for this series of tests.

Riprap Runs

The objective of the various riprap experiments (runs 13-15b) was not necessarily to provide site-specific sizing guidance, but rather to compare the results to various empirical equations which have been developed as guidance for riprap stability. These runs were performed with a larger sand. The flow rate to cause incipient motion for the larger sand at a flow depth of 16 in was 11 ft^3/s. This corresponds to an average approach velocity of approximately 1.4 ft/s which is equivalent to 9.9 ft/s prototype. The grain size distributions for the two riprap sizes which were used are included in appendix A.

Some preliminary tests were done with riprap. Figure 17 shows existing highway pier 7 and railroad pier 9 with 0.123-in (D_{50}) riprap extended 10 in (42 ft prototype) from both piers with Shawn McLemore of the Florida Department of Transportation looking on. The preliminary experiments indicated that the scour would extend about 8 in from the piers without the riprap. Figure 18 shows that the riprap was stable and that there was no scour outside of the riprap. By cutting the extension of riprap to 5 in from each pier, scour did develop outside of the riprap, as shown in figure 19. Although the riprap itself did not scour,the bed material beyond the riprap apron scoured approximately the same as it would have (at that distance from the pier) had the riprap not been in place.

Figure 17. Riprap experiment before flume operation.

Figure 18. Riprap experiment after flume operation.

Figure 19. Scour around riprap.

The depth of scour at railroad pier 9 in run 13 was 0.34 ft. This is similar to, and bracketed by, scour depth measurements at railroad pier 9 for other runs (0.36 ft for run 1b and 0.32 ft for runs 1a, 12a and 12b) which indicates that the experiments were run at near critical shear stress so scour should be near maximum regardless of grain size.

The 0.22-in (D_{50}) riprap extended 10 in from railroad pier 9 in run 14 was stable and provided scour protection. The 0.123-in (D_{50}) riprap extended 10 in from railroad pier 9 in run 15a moved slightly at the higher velocity. The depth of scour using this riprap was 0.08 ft and scour occurred only at the front corners of railroad pier 9. The riprap extended 5 in from railroad pier 9 in run 15b exhibited a scour depth of 0.15 ft and scour occurred at the front and sides of the pier. The depth of scour at the edges and front of the riprap in run 15b was less than the depth of scour that occurred at 5 in (or 0.41 ft) from the pier in run 1a with no riprap. Compare 0.15 ft with 0.36 (0.88-0.41)/0.88 = 0.19 ft assuming a straight line from the deepest point to the extent of scour at the front of the pier. Runs 14, 15a, and 15b indicate that 0.22-in riprap extended 10 in (42 ft prototype) provided adequate scour protection at railroad pier 9.

It is suggested that the following equations, and the corresponding references, be consulted in sizing riprapping around piers. Generally, the maximum velocity by a pier was about 1.5 multiplied by the average approach velocity. This maximum velocity was applied in using the equations and might be considered in design of riprap around piers.

Ishbash developed two equations for riprap stability. Equation (1) is for stones allowed to roll to find a "seat," and equation (2) is for loose top stones that will be removed with a minimum velocity.[3,4,5]

$$D_{50} = 0.69 \frac{V^2}{2g\ (SG-1)} \qquad (1)$$

$$D_{50} = 1.38 \frac{V^2}{2g\ (SG-1)} \qquad (2)$$

where:

D_{50} = mean riprap diameter, in feet,
V = velocity, in ft/s,
SG = specific gravity of riprap,
g = gravitational acceleration, 32 ft/s^2.

Shield's criterion, as proposed for the revised FHWA HEC-11, results in the following:[6]

$$D_{50} = 0.00176 \left(\frac{SF}{K_1}\right)^{1.5} \frac{V^3}{d^{0.5}(SG-1)^{1.5}} \qquad (3)$$

where:

D_{50} = mean riprap diameter, in feet,

SF = stability factor, 1.2 or 1.6 for straight and curved channels, respectively,

K_1 = side slope correction,

$$= \left[1 - \frac{\sin^2}{\sin^2\Phi}\right]^{0.5} \qquad (4)$$

= bank side slope, in degrees,

Φ = angle of repose, in degrees,

V = average (in vertical) velocity, in ft/s,

d = depth, in feet.

Maynord developed equations (5) and (6) in terms of D_{30} since he considers D_{30} to be a better size parameter than D_{50} for design of riprap.[7] Since the other riprap equations are in terms of D_{50}, equation (7) was used as an approximation to facilitate comparing Maynord's results to our observations and to other methods.

$$D_{30} = (SF) \frac{0.00312\ V^{2.5}}{d^{0.25}(SG-1)^{1.25}} \quad \text{(for straight reaches)} \qquad (5)$$

$$D_{30} = (SF) \frac{0.00391\ V^{2.5}}{d^{0.25}(SG-1)^{1.25}} \quad \text{(for curved reaches)} \qquad (6)$$

$$D_{50} = 1.25 D_{30} \qquad (7)$$

where:

D_{30} = diameter at which 30 percent of riprap is smaller, in feet,

D_{50} = mean riprap diameter, in feet.

V = average (in vertical) velocity, in ft/s,

d = depth, in feet,

SF = stability factor,

SG = specific gravity of riprap.

A stability factor (SF) of 1.2 is suggested for design, and the velocity at the outside of a bend is suggested to be determined by multiplying the average velocity in the main channel by 1.53. No side slope correction was found to be needed for side slopes as steep as 2:1.[7]

The typical methodology for sizing riprap for bridge pier protection is to adjust the approach velocity to account for the effects of the bridge pier, and then to use one of the riprap design equations previously described. Recommendations in the literature are to multiply the approach velocity by 1.5 or 2.0.[8,5] In table 4, values of 1.5, 1.75, 2.0, and 2.5 are used to adjust the approach velocity for comparison between predicted sizes from the previously described equations and observed sizes from the model study. The average approach velocity was 1.4 ft/s and the near-bed approach velocity which was used in the second set of Ishbash computations was taken as 0.72 ft/s as described in the following discussion. The stability (or safety) factor used in the riprap equations was 1.0.

The Ishbash equations predicted significantly larger riprap sizes than were observed to be at incipient motion when the average velocity was used. This overprediction is attributed to the selection of the average rather than the near-bed velocity, and the near-bed velocity is a better representation of the conditions experienced in this model study. Ishbash conducted his experiments

Table 4. Riprap sizing comparison.[1]

	D_{50} (inches)			
Velocity Multiplier	1.5	1.75	2.0	2.5
Observed Incipient Motion	0.22	0.22	0.22	0.22
Ishbash Equation (1) with Average Velocity	0.30	0.40	0.53	0.82
Ishbash Equation (2) with Average Velocity	0.60	0.80	1.06	1.66
Proposed HEC-11/Shield's Equation (3)	0.06	0.09	0.14	0.21
Maynord Equation (5), Modified with Equation (7)	0.12	0.15	0.20	0.32
Ishbash Equation (1) with Near-Bed Velocity	0.08	0.10	0.14	0.22
Ishbash Equation (2) with Near-Bed Velocity	0.16	0.21	0.28	0.43

[1]The depth was 1.33 ft and the approach velocity was 1.4 ft/s; model pier width was 5.28 in (b/D_{50} = 24); SF was used as 1.0 in the equations; SG of model riprap was 2.90.

by dropping prototype riprap stones in water flowing over the crest of an embankment. The vertical velocity distribution for such a flow condition is relatively uniform, so the average velocity that was recorded was close to the near-bed velocity which would cause riprap instability. This is not the case for the physical model or the St. Johns River, where the near-bed velocity would be significantly lower than the average velocity.

The Corps of Engineers recommends using a point velocity taken at a distance D_{50} above the bed as the near-bed velocity to be used in the Ishbash equations.(9) Using this point in the log velocity distribution equation presented by Christiansen(10) in the consultant report for the Acosta Bridge project provides a method for relating the near-bed velocity to the average velocity. The log velocity distribution is as follows:

$$\frac{v}{v_s} = 2.5 \ln \left(\frac{29.7y}{k} + 1 \right) \tag{8}$$

where:

v	=	local velocity at a distance **y** from a fixed boundary, in ft/s,
v_s	=	shear velocity (which is cancelled out in this analysis), in ft/s,
y	=	distance from fixed boundary, in feet,
k	=	representative roughness size (D_{50} in this analysis), in feet.

The 0.22-in model riprap would have scaled to 0.91 ft in prototype dimensions, but the model riprap had a specific gravity of 2.90. Riprap usually has a specific gravity close to 2.65; the observed size would have been larger with that specific gravity. Using Shield's criteria as a basis for adjustment, the expected size for the lower specific gravity can be determined as follows:

$$\frac{D_{50}\text{ Adjusted}}{D_{50}\text{ Observed}} = \frac{(SG-1)^{1.5}\text{ Actual}}{(SG-1)^{1.5}\text{ Adjusted}}$$

$$D_{50}\text{ Adjusted} = (1.90/1.65)^{1.5}\ D_{50}\text{ Observed}$$

$$= 1.23 \times 0.22 = 0.27\text{ in}$$

The adjusted size then would scale to 1.12 ft in prototype dimensions. Applying a stability factor of 1.2 as defined in the Shield's relationship, Equation (3), results in a prototype size of 1.5 ft.

Since designers are more likely to be able to predict an average velocity than a point velocity or the shear velocity, a convenient way to use the log velocity distribution equation is to set it in terms of an average velocity. Assume that the point velocity is equal to the average velocity at y = 0.368 * depth. Then equation (8) can be manipulated to yield:

$$\frac{v}{V_{AVG}} = \left[\ln\left(\frac{29.7y}{D_{50}} + 1\right)\right] / \left[\ln\left(\frac{10.93 \times \text{depth}}{D_{50}} + 1\right)\right]$$

For y = D_{50} and depth = 1.33 ft, this relationship yields a near-bed velocity = 0.51 V_{AVG} = 0.72 ft/s for the model study. Using the near-bed velocity in the Ishbash equations bring them more in line with other equations and in general agreement with the observed results.

5. Conclusions

Physical models were used to qualitatively investigate the effects that a proposed commuter bridge may have on scour at the existing Acosta Highway Bridge and the railroad bridge. The results of the study can only be used qualitatively because there were limitations in the modeling procedures. The actual site is tidally influenced, so flow travels in both the ebb (downstream) and flood (upstream) direction. Since the flow in the flume is unidirectional, the models were placed such that the flow was in the flood direction because this is the direction in which the proposed bridge could affect scour at the existing bridges. Another limitation was in modeling the bend in the river at which the bridges are located. Most of the runs were performed with the bridges perpendicular to the flow direction, but there were a few runs in which the bridge piers were at a 15 degree angle to the flow direction to try to simulate some of the effects of the bend. Concerns over the sand grain size and the fact that there was no upstream supply of sand (no sand replenishment) were mitigated by keeping the shear stress near the critical shear stress. This should have resulted in near maximum scour depth for all runs.

This study was not intended to model contraction scour or velocity patterns within the cross section for this site. Only a confined strip of the cross section was modeled to determine the relative effects of flow currents from one pier on scour at others.

The results of the experiments indicate that the proposed steel bridge will increase scour at existing highway bridge piers 5 and 6 and railroad pier 7, and slightly decrease the scour at railroad pier 8. Results concerning railroad pier 9 are inconclusive. The proposed temporary structures to be used with the steel alternative will not affect scour at either the existing bridge piers or the railroad piers. The proposed concrete bridge will not influence scour at railroad pier 7 and will increase scour at railroad pier 8.

If the flow direction is aligned with the piers, the existing Acosta piers actually provide some protection to railroad pier 8 from additional scour caused by the proposed new piers. However, if the flow is oriented at 15 degrees to the alignment of the piers, the existing Acosta piers could actually worsen the scour if they were left in place.

Nonsite-specific scour issues were also investigated in this study. Footer location greatly influences the extent of scour but does not have much affect on the depth of scour if the footer is on or above the bed, based on test results. Experiments which were performed to compare scour at pile groups versus scour at equivalent width rectangular piers indicate that assuming a rectangular pier width equal to the net total width of piles obstructing the flow is a conservative assumption for estimating scour for pile groups.

The 0.22-in (D_{50}) model riprap was found to be stable, but had a specific gravity of 2.90. After adjusting to a more typical specific gravity of 2.65 and applying a stability factor of 1.2, this material would scale to approximatley 1.5 ft in prototype dimensions. Using a velocity multiplier that ranged from 1.75 to 2.5, the uniform flow riprap equations gave comparable riprap sizes with the exception of the Ishbash equations when a depth average velocity was used. The Ishbash equations are appropriately used with a near-bed velocity and tend to overpredict riprap sizes when the depth average velocity is used for a deep channel.

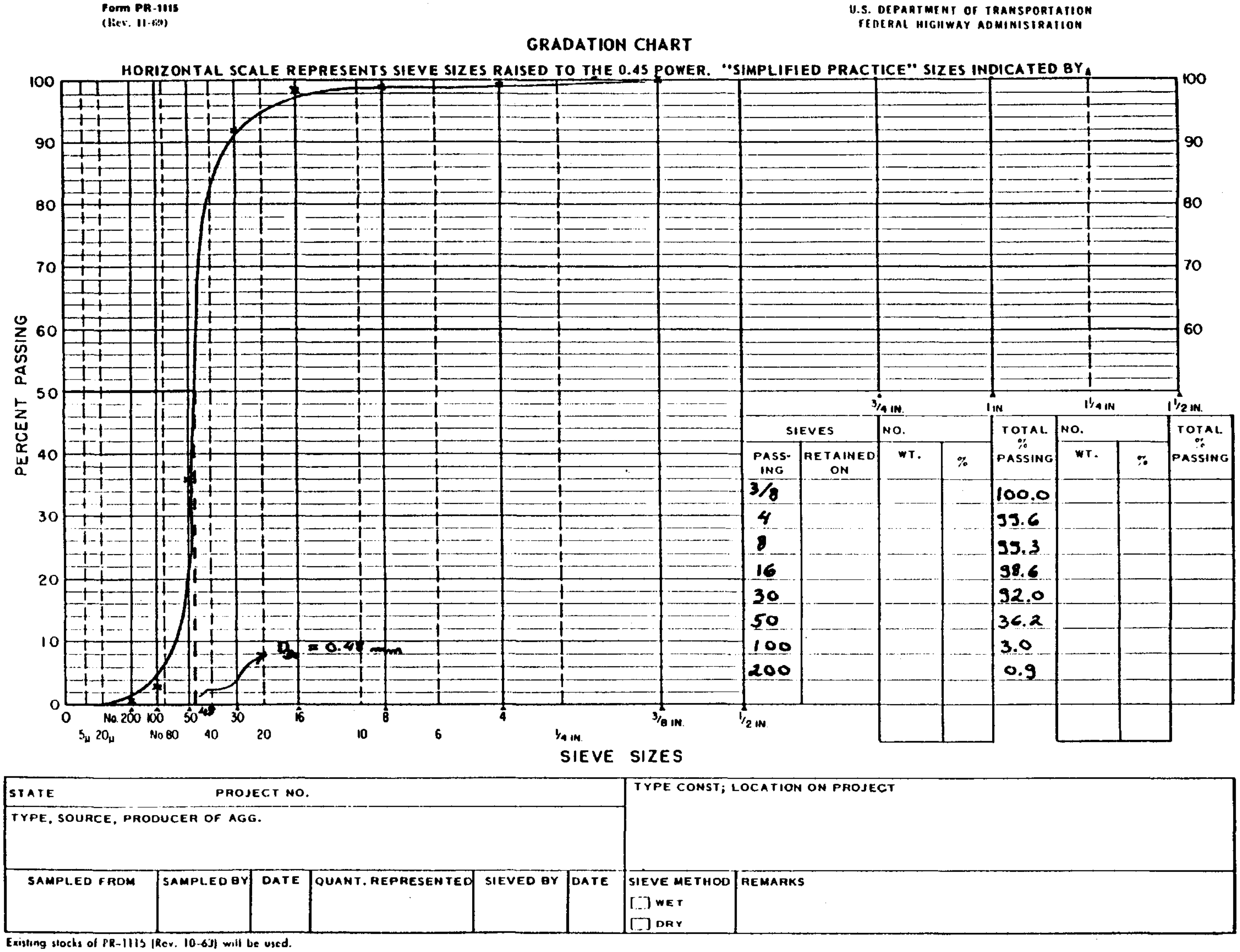


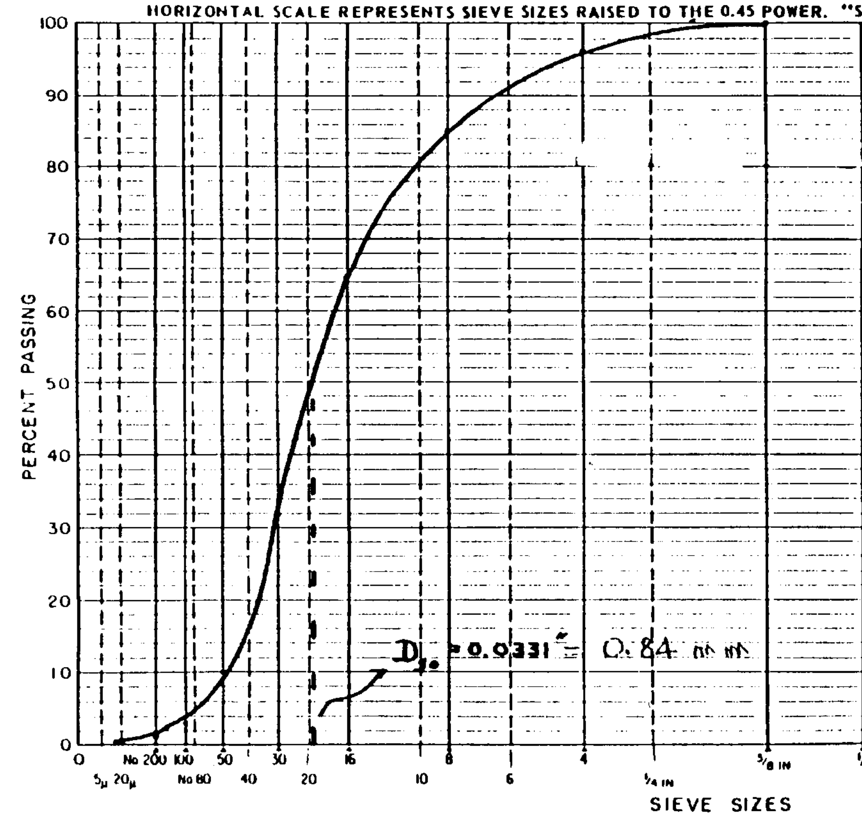
Figure 20. Gradation chart for fine sand.

Form PR 1115
(Rev. 11-69)

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

GRADATION CHART

HORIZONTAL SCALE REPRESENTS SIEVE SIZES RAISED TO THE 0.45 POWER. "SIMPLIFIED PRACTICE" SIZES INDICATED BY



SIEVES PASSING	SIEVES RETAINED ON	NO. WT. (g)	NO. %	TOTAL % PASSING	NO. WT.	NO. %	TOTAL % PASSING
3/8"		73.3		100.0			
4		241.6		96.3			
8		440.0		85.2			
16		718.7		65.0			
30		474.4		31.3			
50		147.6		10.0			
100		42.1		3.2			
200		12.1		1.3			

3/4 IN. 1 IN 1 1/4 IN 1 1/2 IN

STATE	PROJECT NO.	TYPE CONST; LOCATION ON PROJECT
TYPE, SOURCE, PRODUCER OF AGG.		

SAMPLED FROM	SAMPLED BY	DATE	QUANT. REPRESENTED	SIEVED BY	DATE	SIEVE METHOD	REMARKS
						[ ] WET [ ] DRY	

Previous edition of PR 1115 (Rev. 10-63) will be used

Figure 21. Gradation chart for coarse sand.

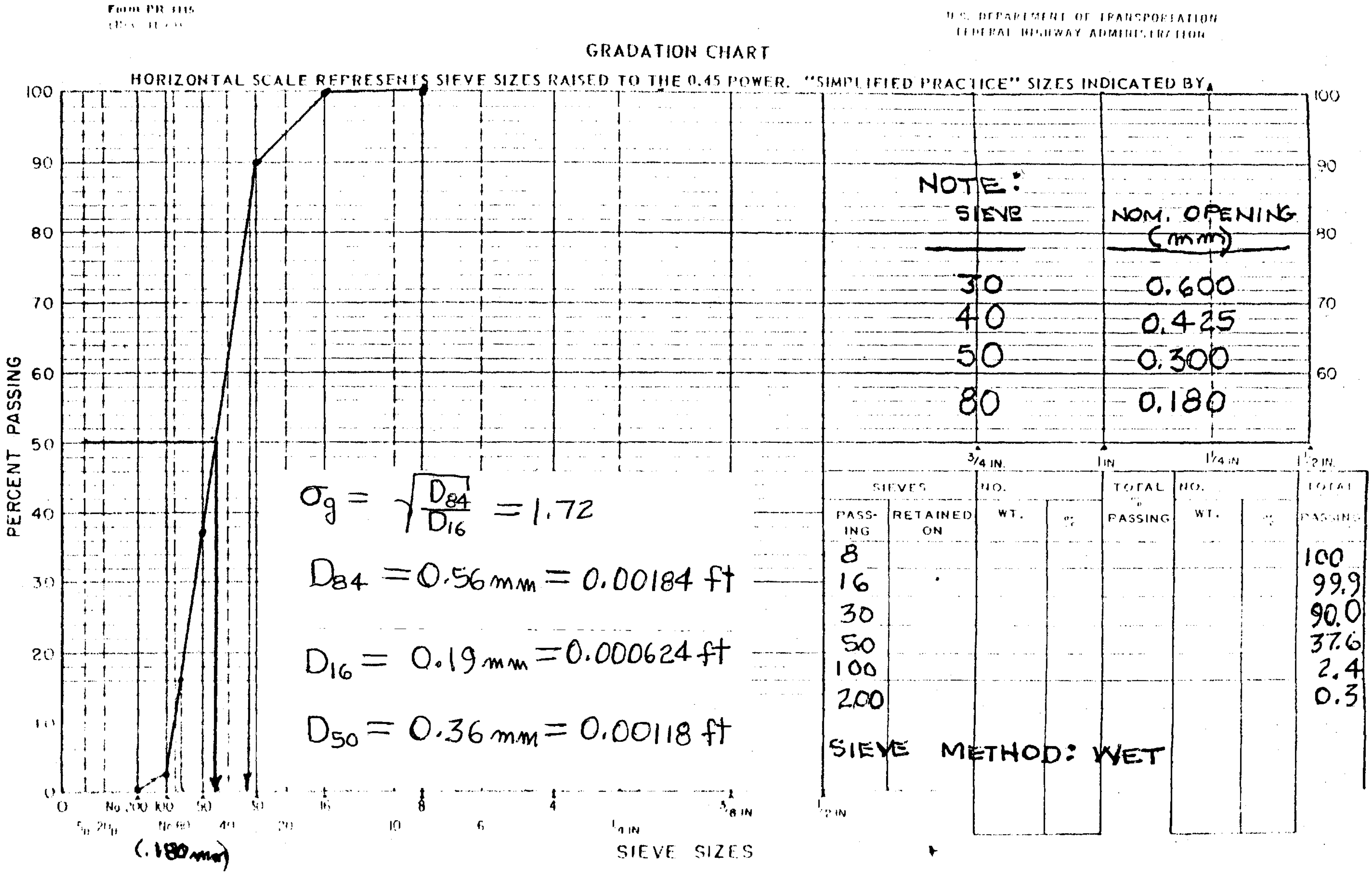


Figure 22. Gradation chart for fine uniform sand used in pile spacing reruns (R20A-R23B)

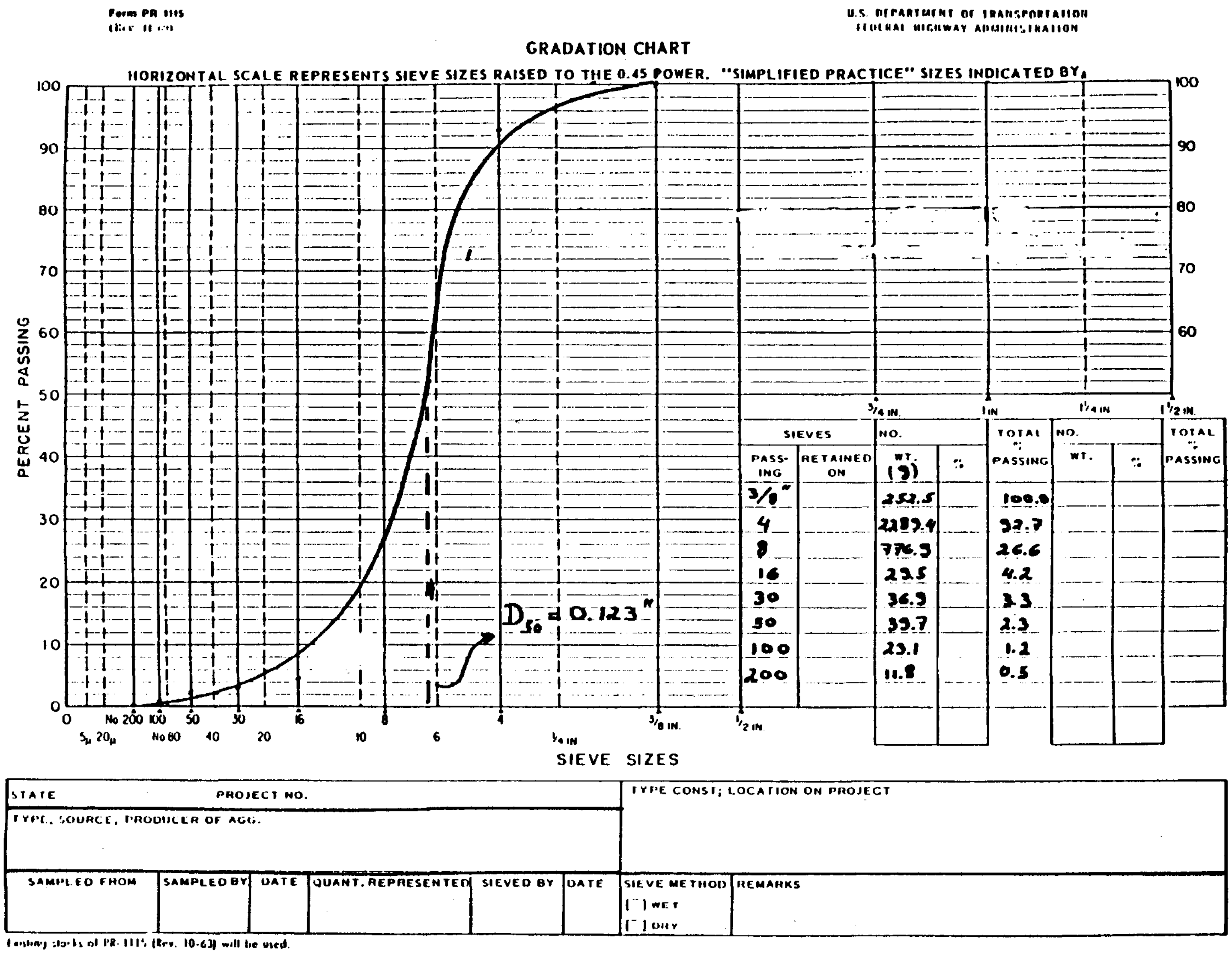


Figure 23. Gradation chart for 0.123 in (D_{50}) riprap.

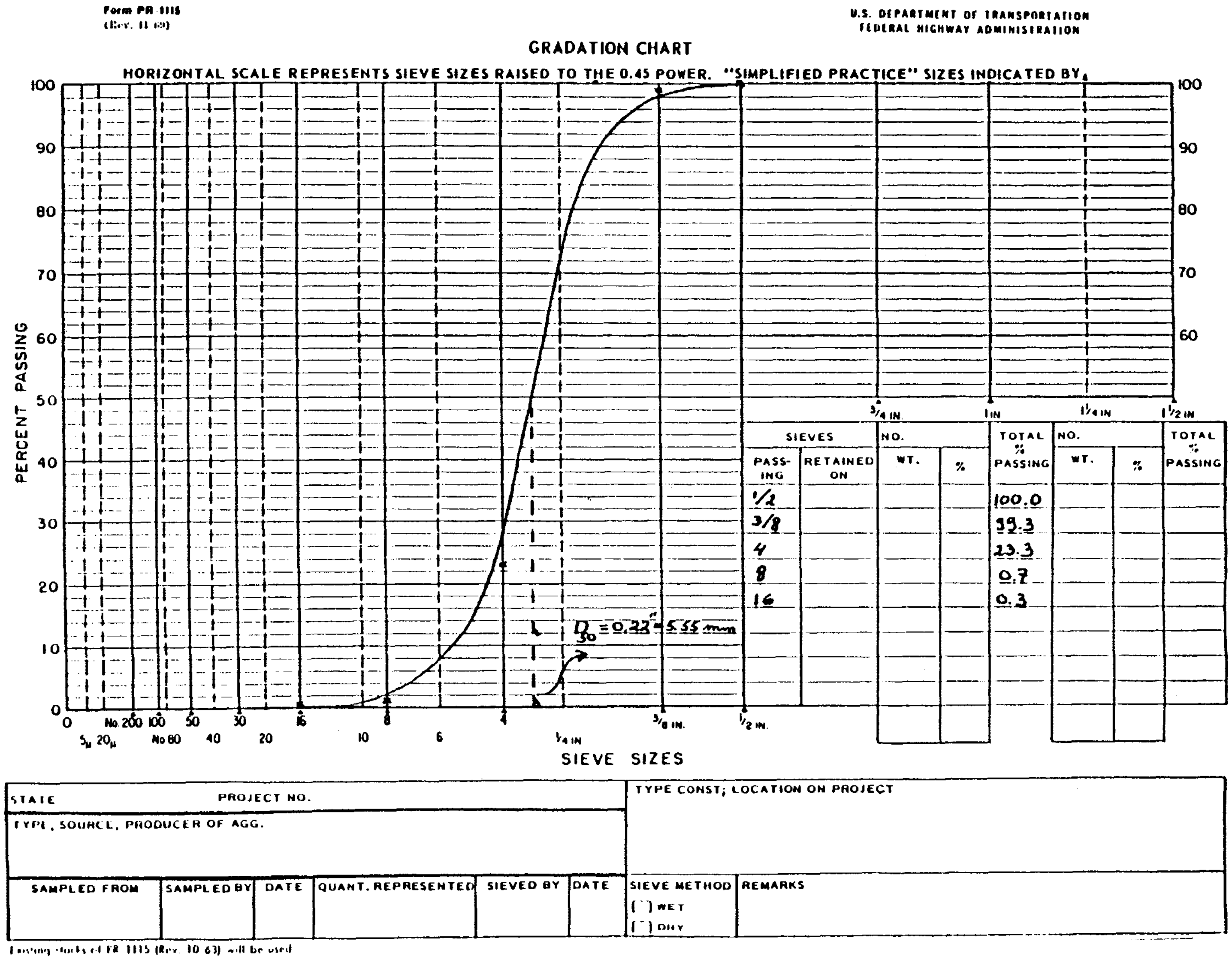


Figure 24. Gradation chart for 0.22 in (D_{50}) riprap.

Appendix B. Velocity Plots

The maximum velocities in the following graphs include some velocity increase due to the loss of flow area when the pier models are placed in the confined flume.

The point velocity represented by the solid data symbols were measured directly in front of the pier with the back of the pitot tube right against the front of the pier. The so called "maximum velocity" was measured at the upstream corner of the pier and was obtained by rotating the pitot tube until it appeared to be oriented directly in line with the flow currents around the pier. See figure 7 for an illustration of the points of measurement.

A two-dimensional velocity probe was not available for this study, so the front velocity was just a measure of the component in the general flow direction and did not include the diving component. Likewise the so called "maximum velocity" did not include the diving component or other secondary currents associated with vortices around the pier. These measurements were taken to provide some insight into what was happening around the pier and should not be taken to represent the maximum that could occur.

In a later study, it was determined that the most meaningful measurements were a near-bed velocity around the pier and a corresponding near-bed velocity in the approach flow. A reasonable assumption is that the average velocity around the pier (if it could be measured) is a multiple of the average approach velocity in proportion to the ratio of the bed velocities (which can be measured more readily).

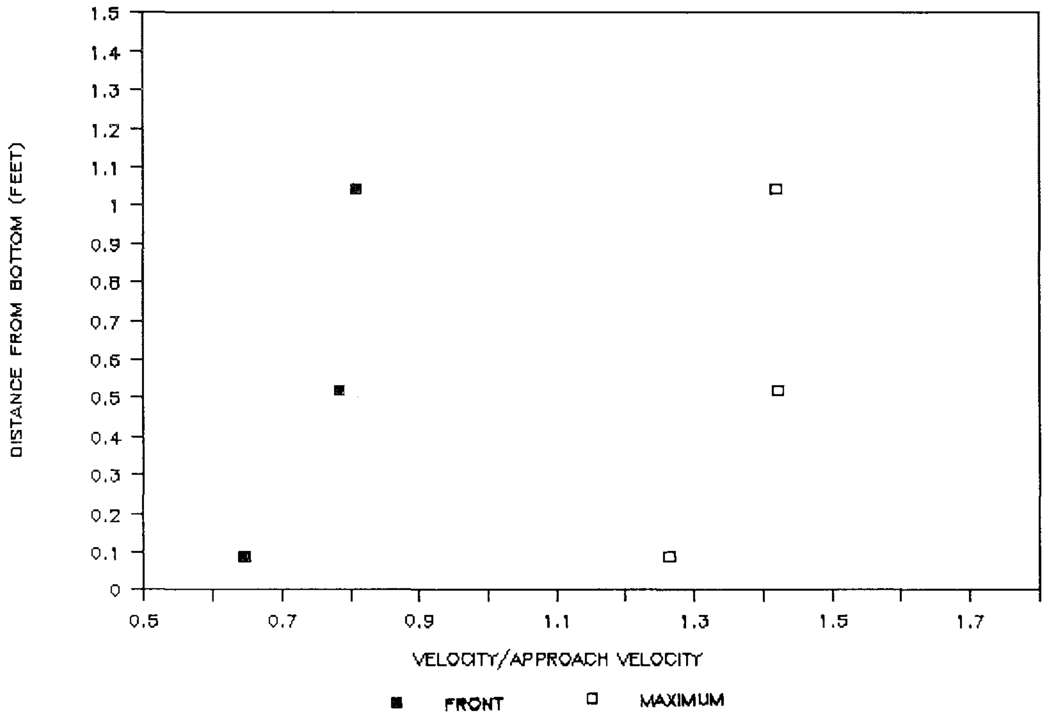


Figure 25. Velocity at railroad pier 7 (run 1a).

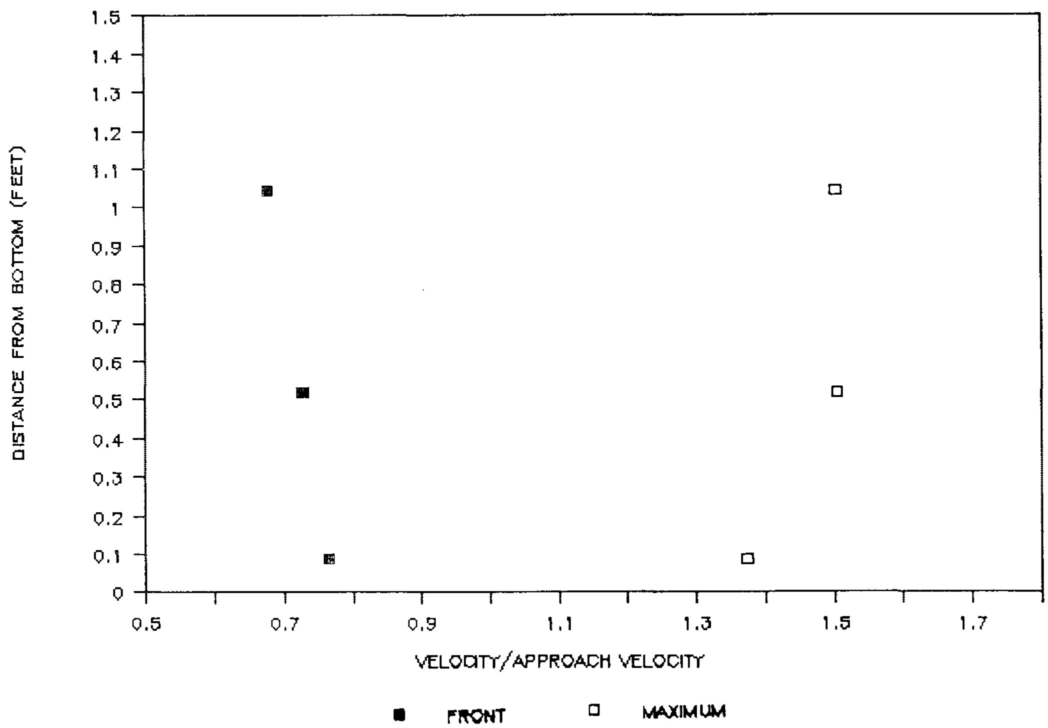


Figure 26. Velocity at railroad pier 8 (run 1a).

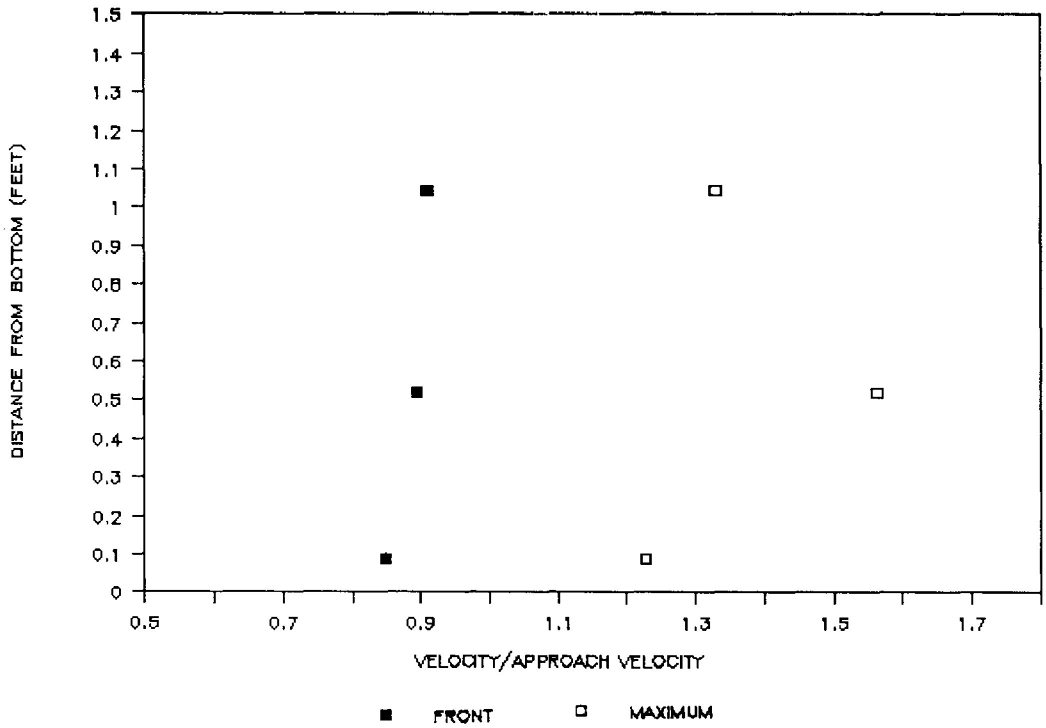


Figure 27. Velocity at railroad pier 9 (run 1a).

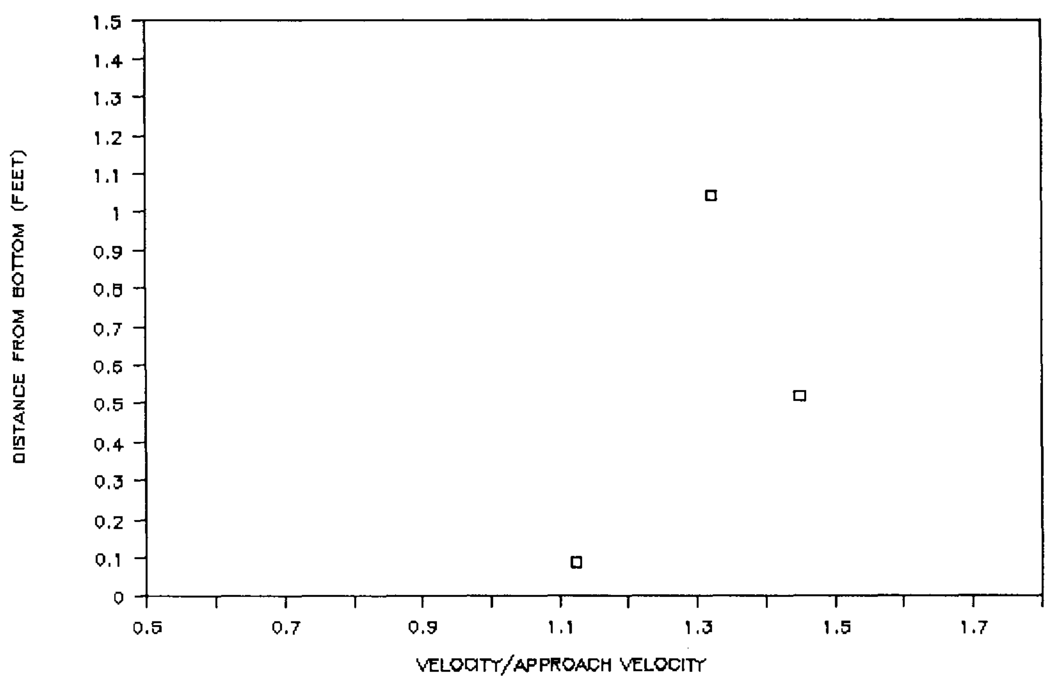


Figure 28. Velocity at center of highway piers 7 and 8 (run 1a).

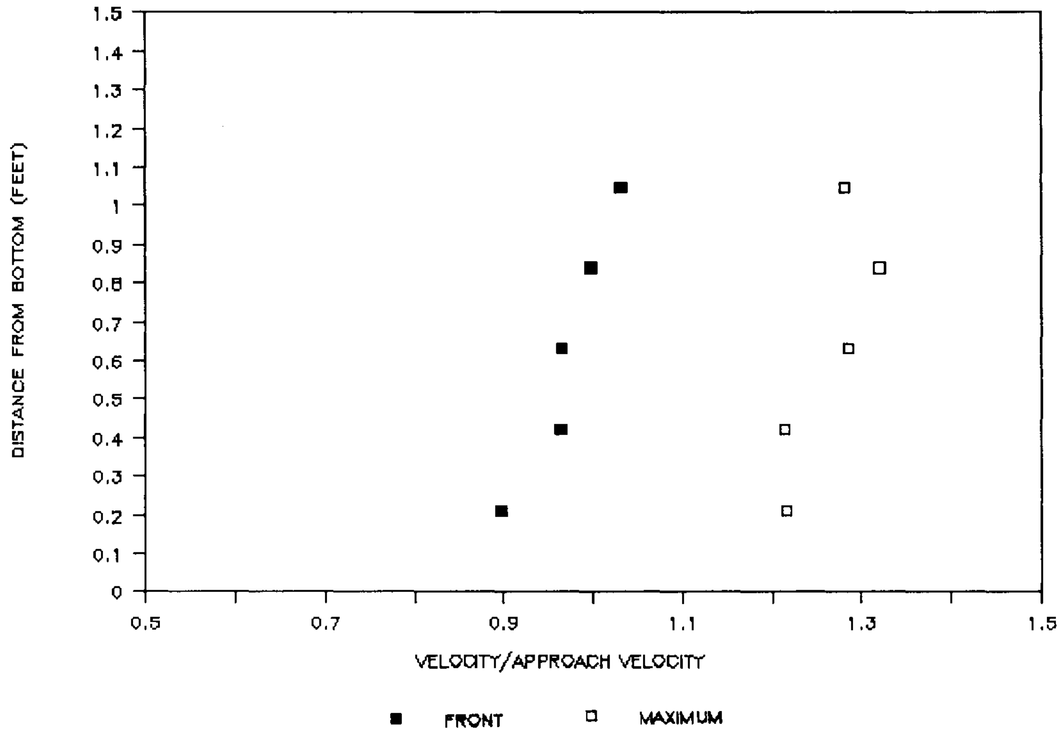


Figure 29. Velocity at railroad pier 7 (run 2).

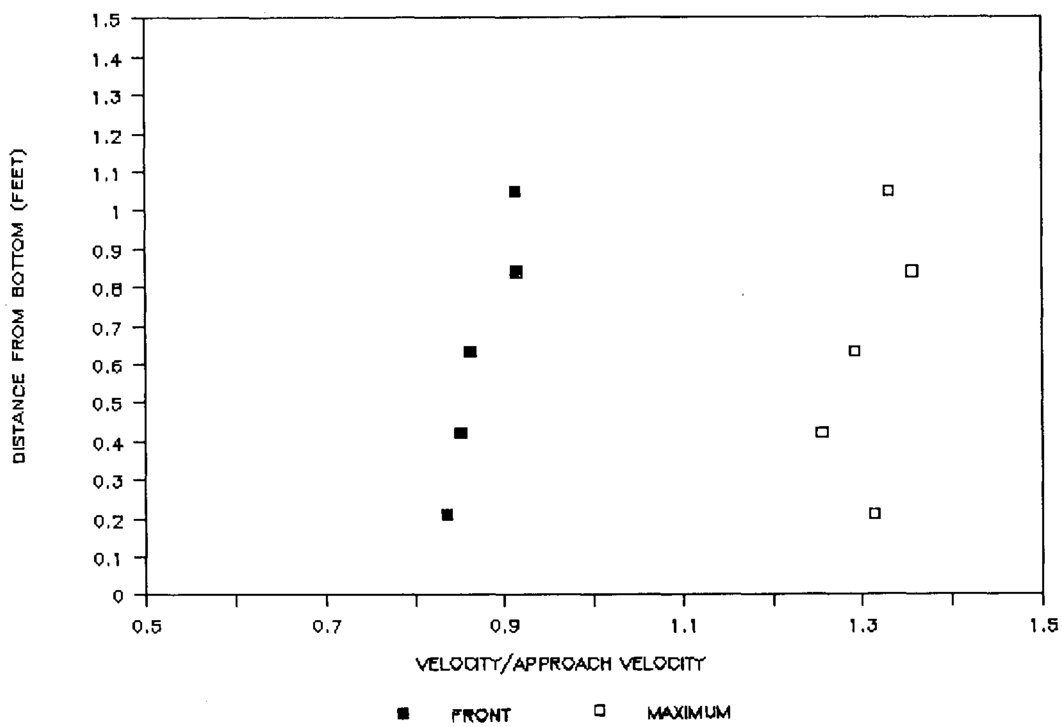


Figure 30. Velocity at railroad pier 8 (run 2).

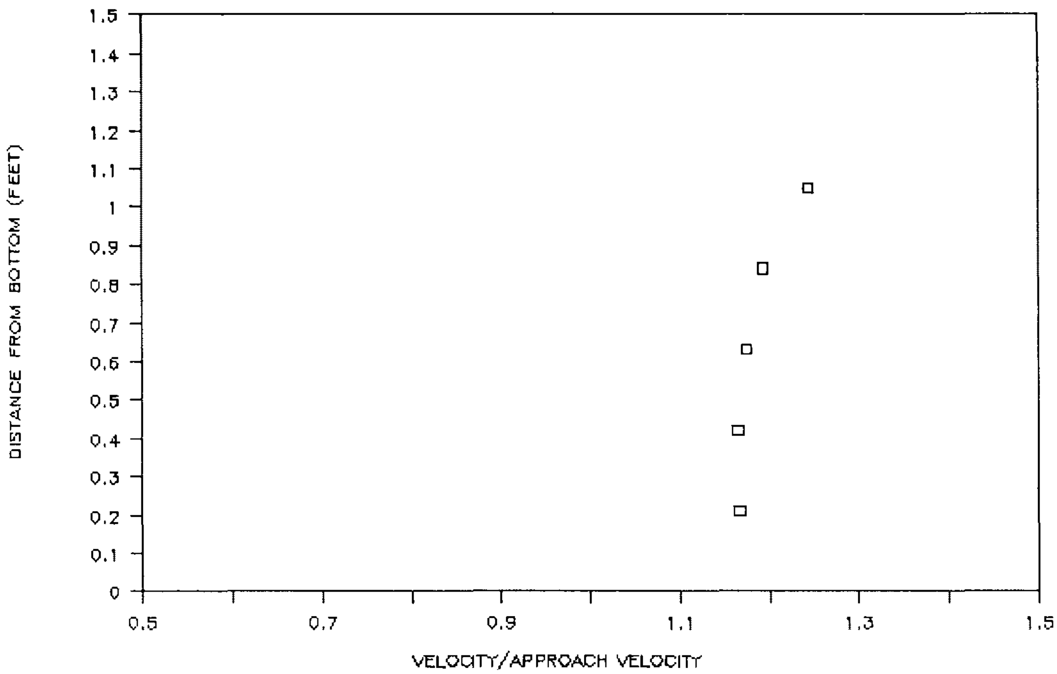


Figure 31. Velocity at center of railroad piers 7 and 8 (run 2).

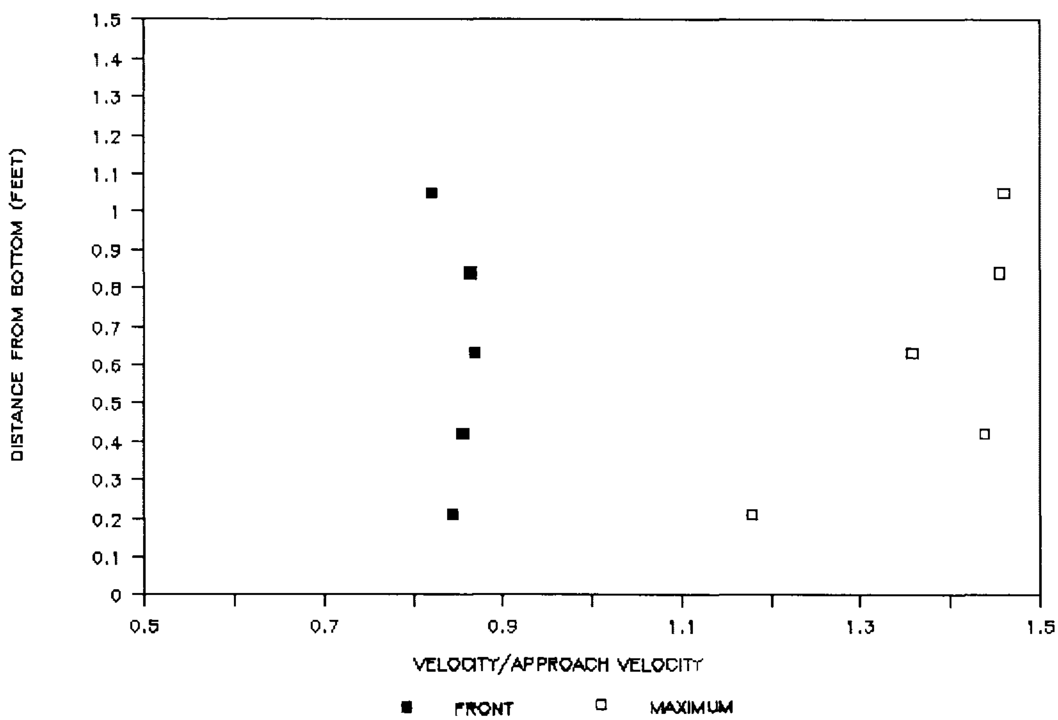


Figure 32. Velocity at center of railroad pier 7 (run 2).

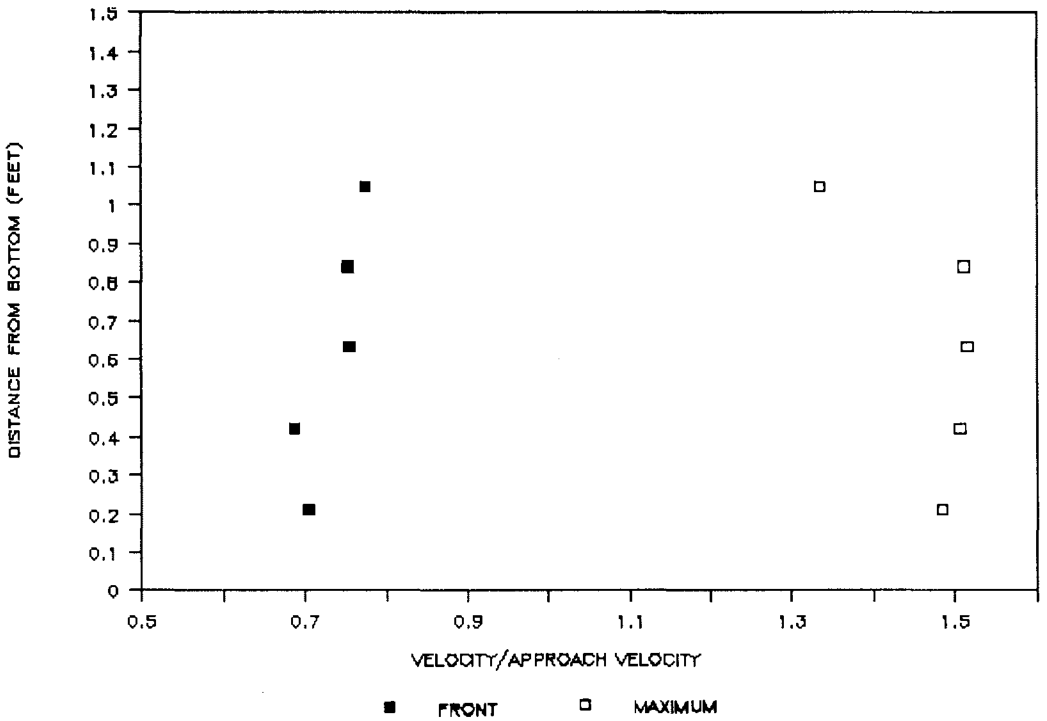


Figure 33. Velocity at railroad pier 8 (run 2b).

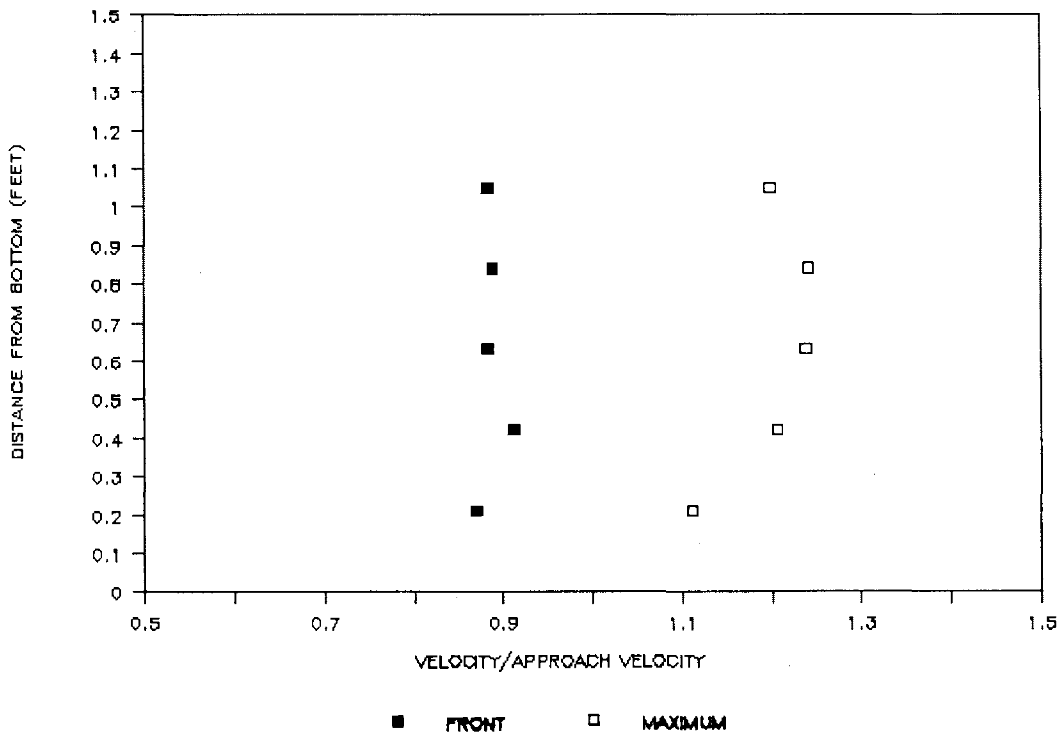


Figure 34. Velocity at highway pier 5 (run 3).

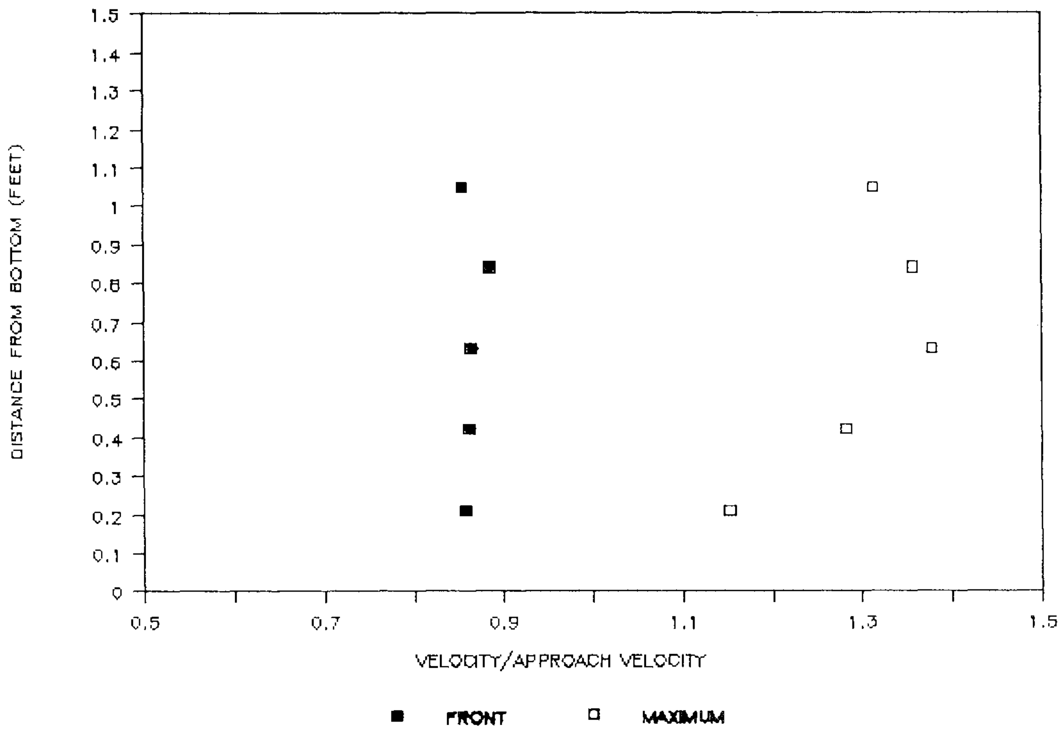


Figure 35. Velocity at highway pier 6 (run 3).

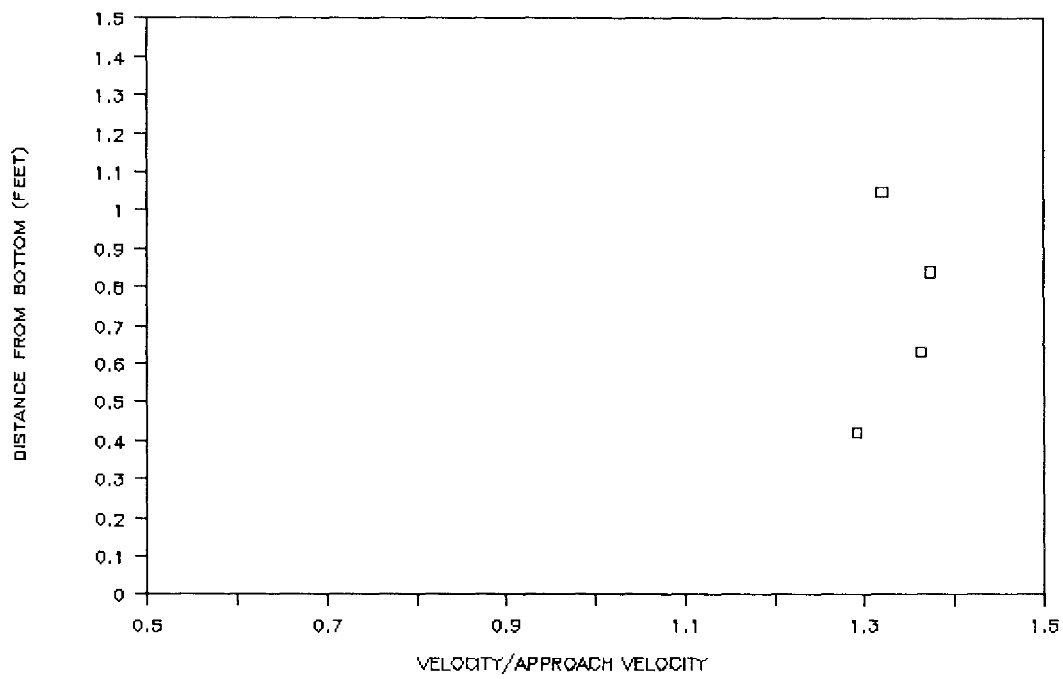


Figure 36. Velocity at railroad pier 7 (run 3).

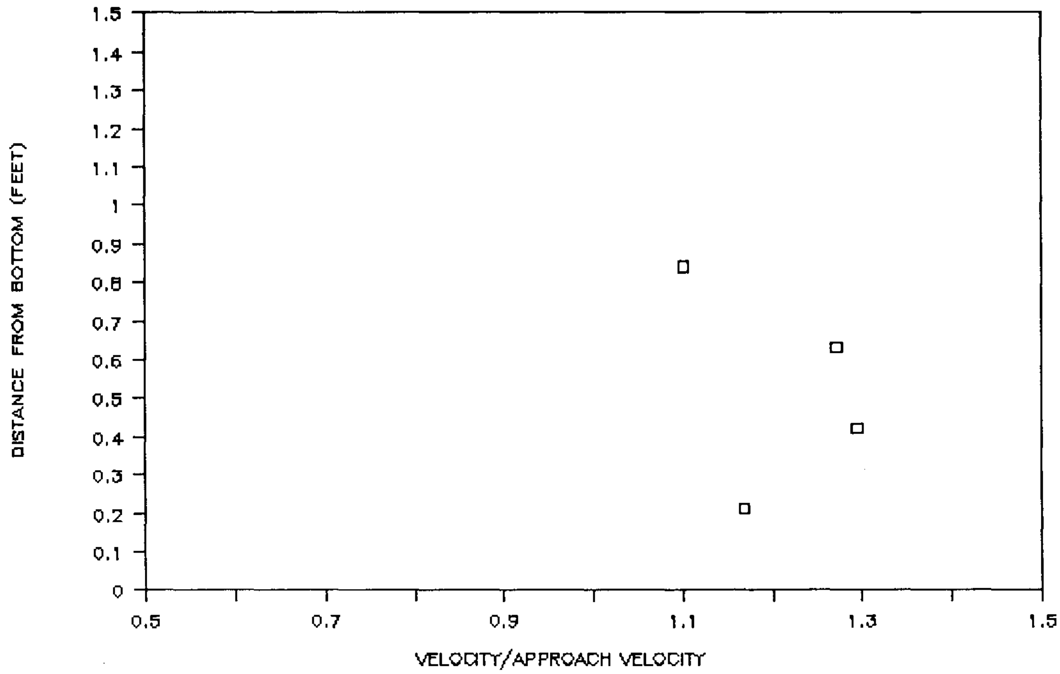


Figure 37. Velocity at railroad pier 8 (run 3).

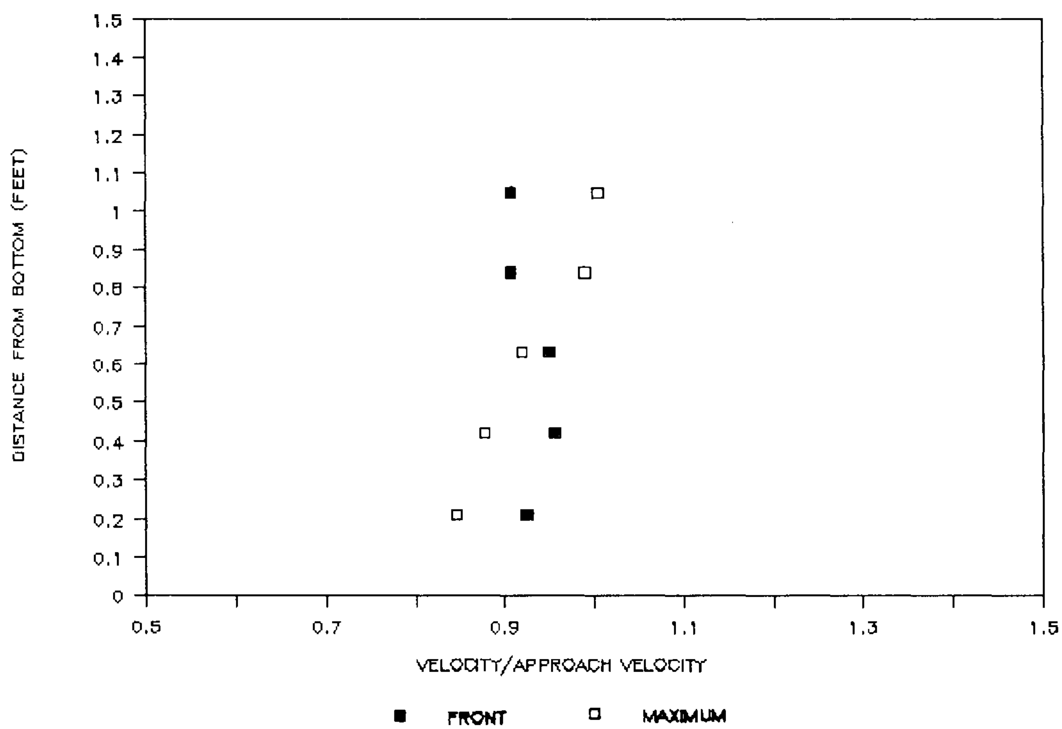


Figure 38. Velocity at highway pier 5 (run 3b).

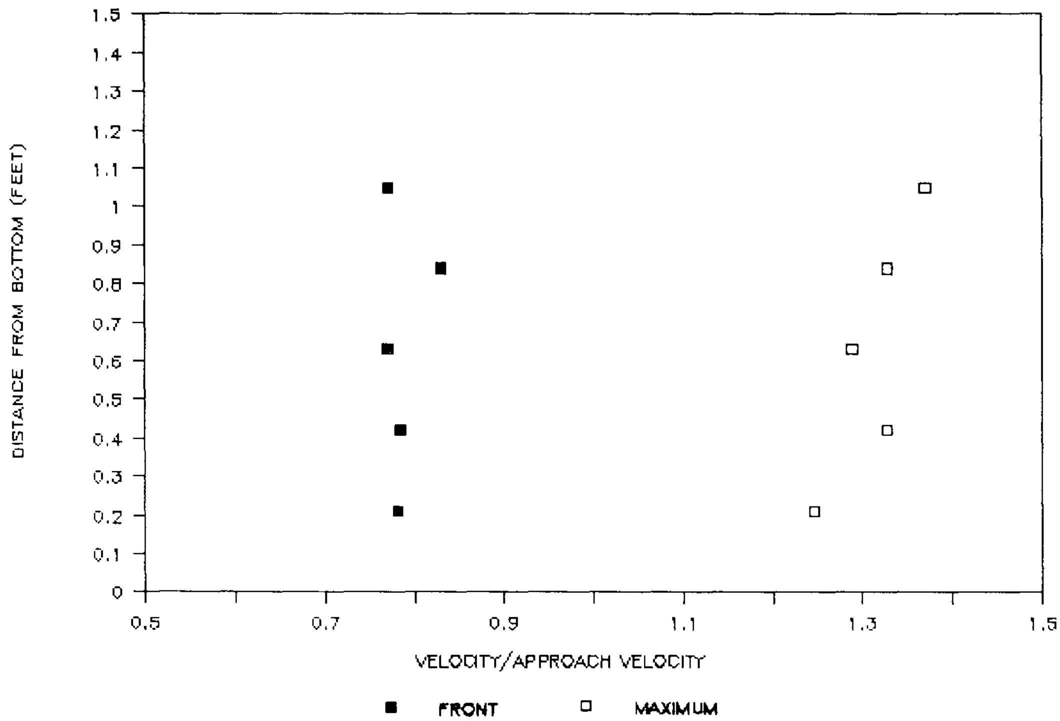


Figure 39. Velocity at highway pier 6 (run 3b).

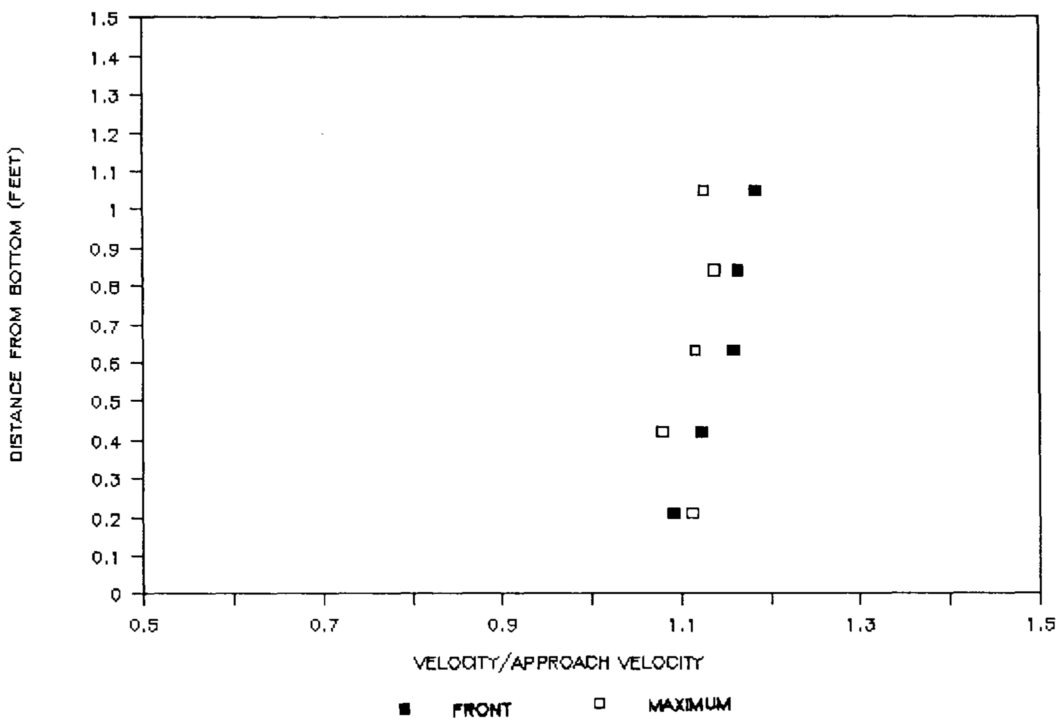


Figure 40. Velocity at railroad pier 7 (run 3b).

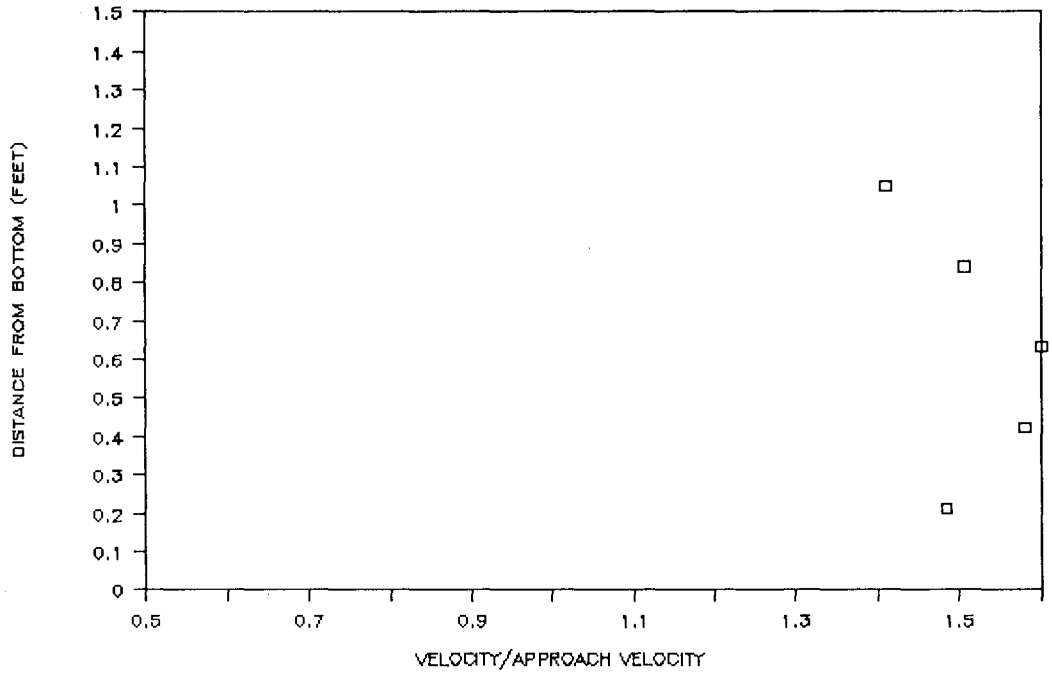


Figure 41. Velocity at railroad pier 8 (run 3b).

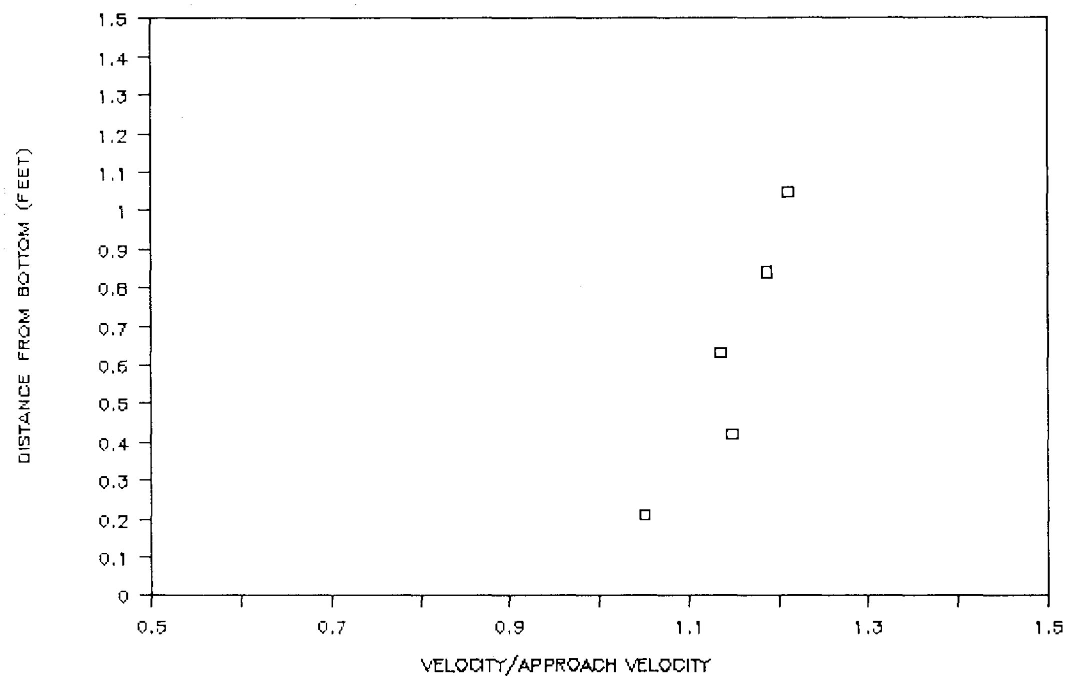


Figure 42. Velocity at center of highway piers 5 and 6 (run 3b).

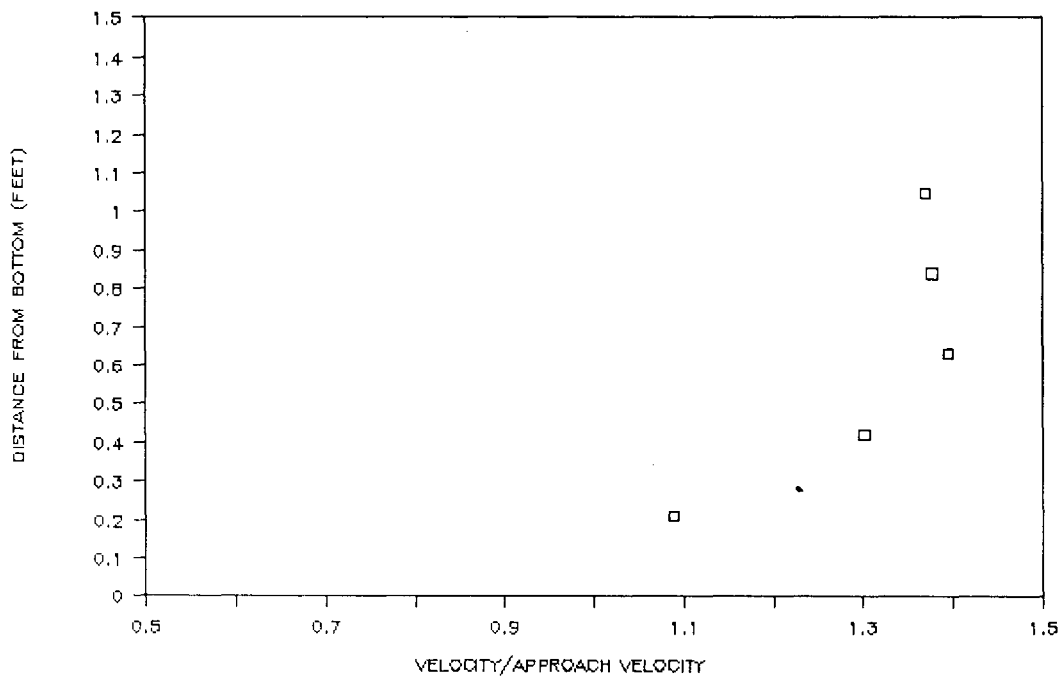


Figure 43. Velocity at center of railroad piers 7 and 8 (run 3b).

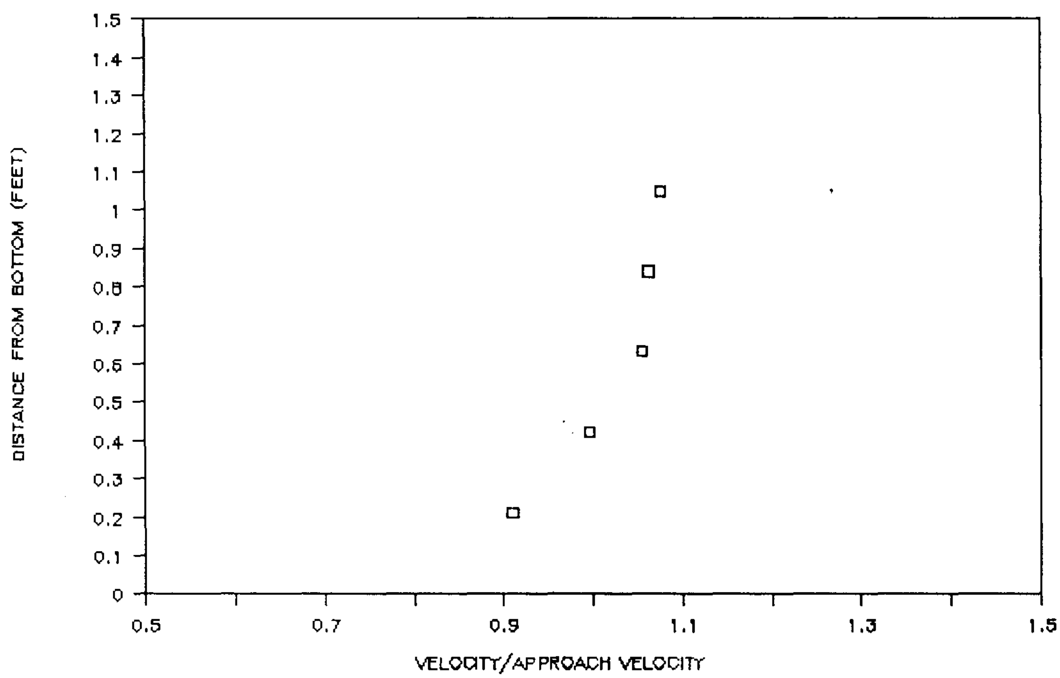


Figure 44. Velocity at highway pier 5 (run 3c)

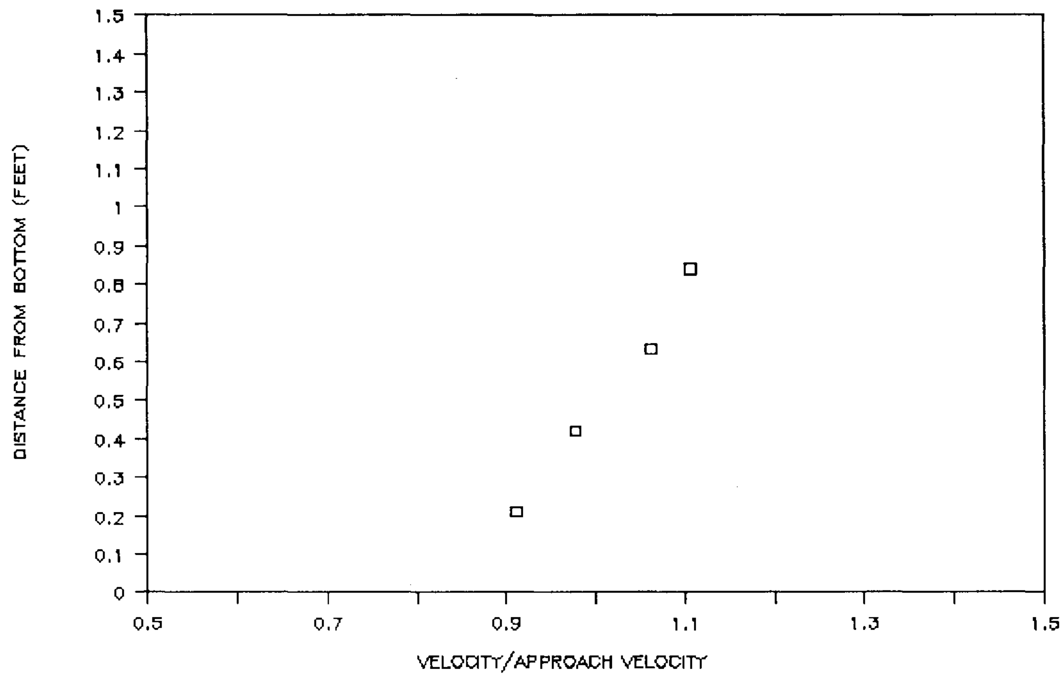


Figure 45. Velocity at highway pier 6 (run 3c).

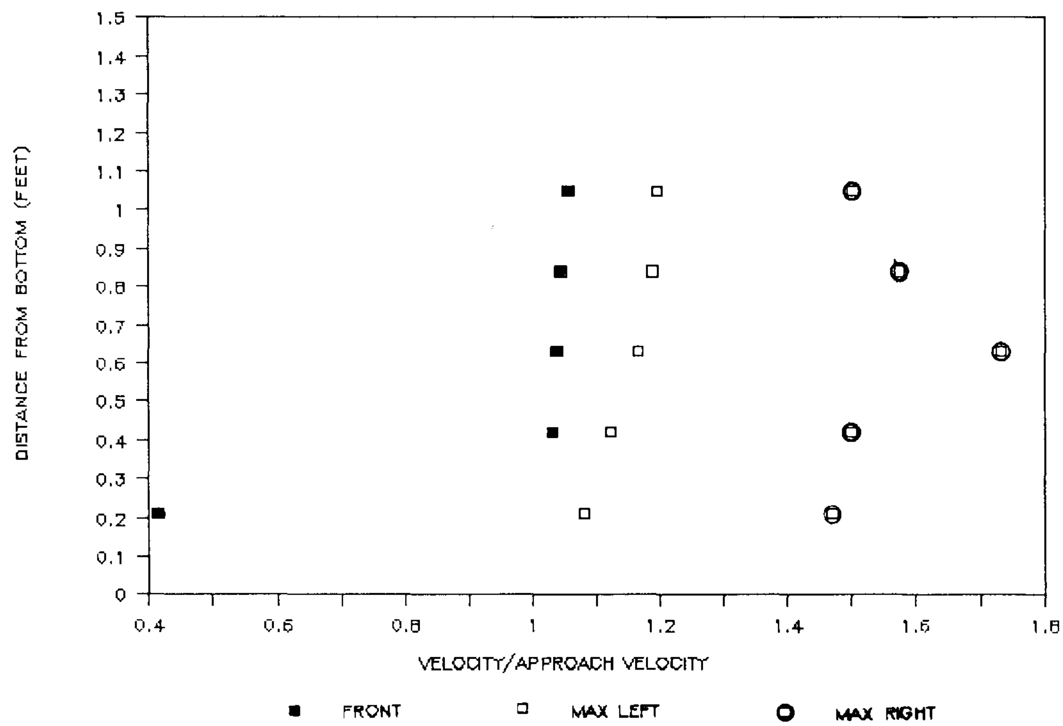


Figure 46. Velocity at railroad pier 7 (run 3c).

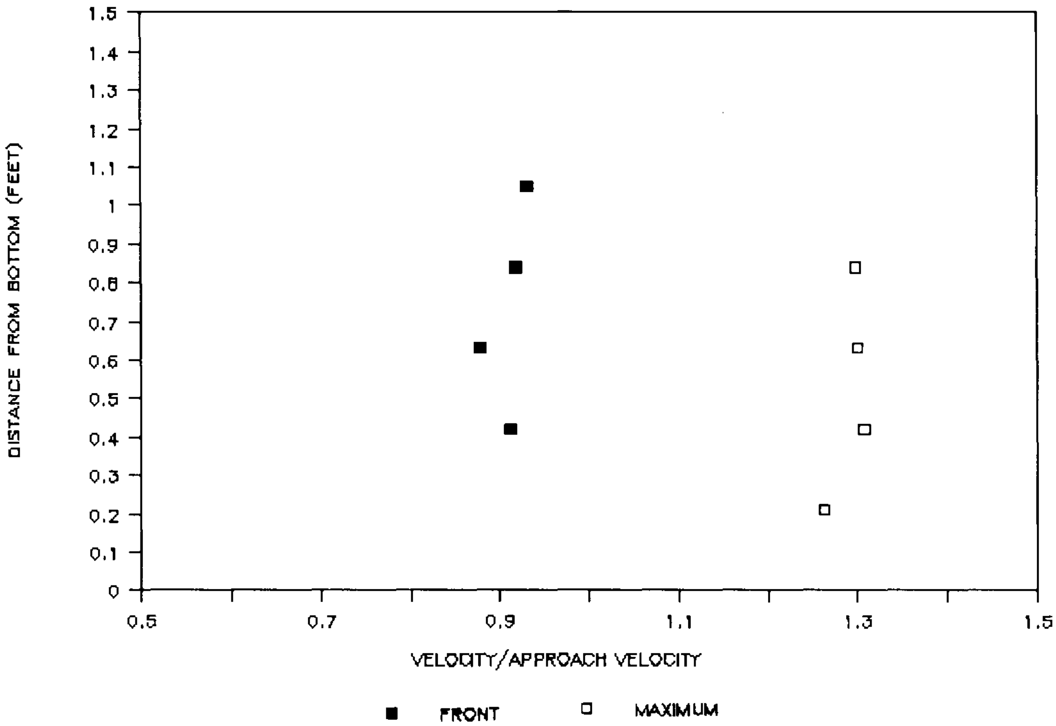


Figure 47. Velocity at railroad pier 8 (run 3c).

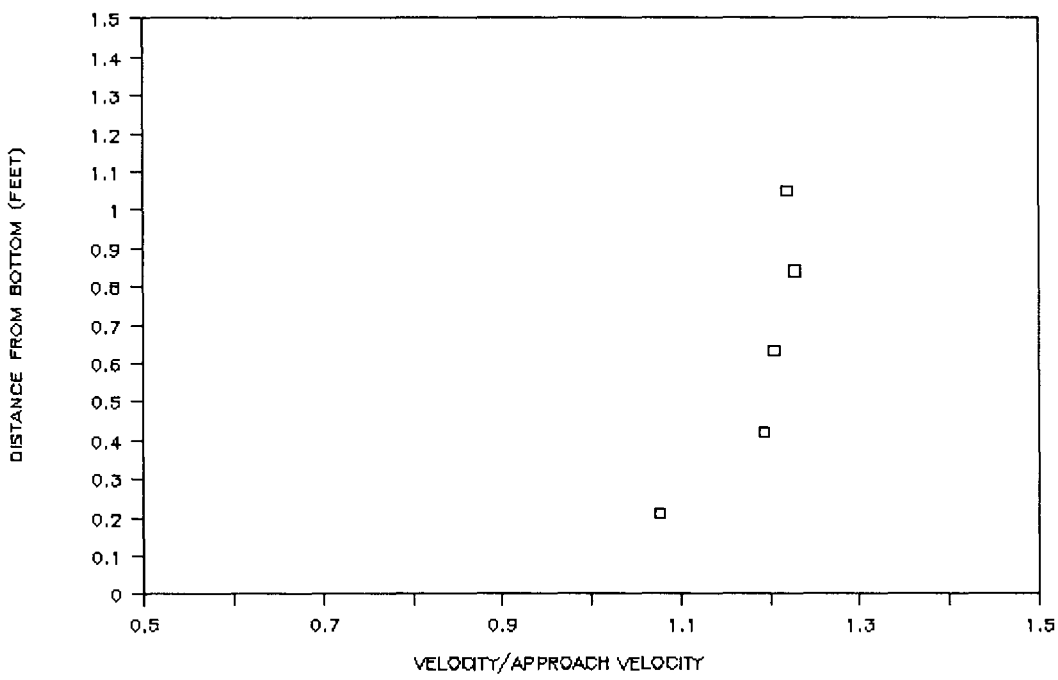


Figure 48. Velocity at center of railroad piers 7 and 8 (run 3c).

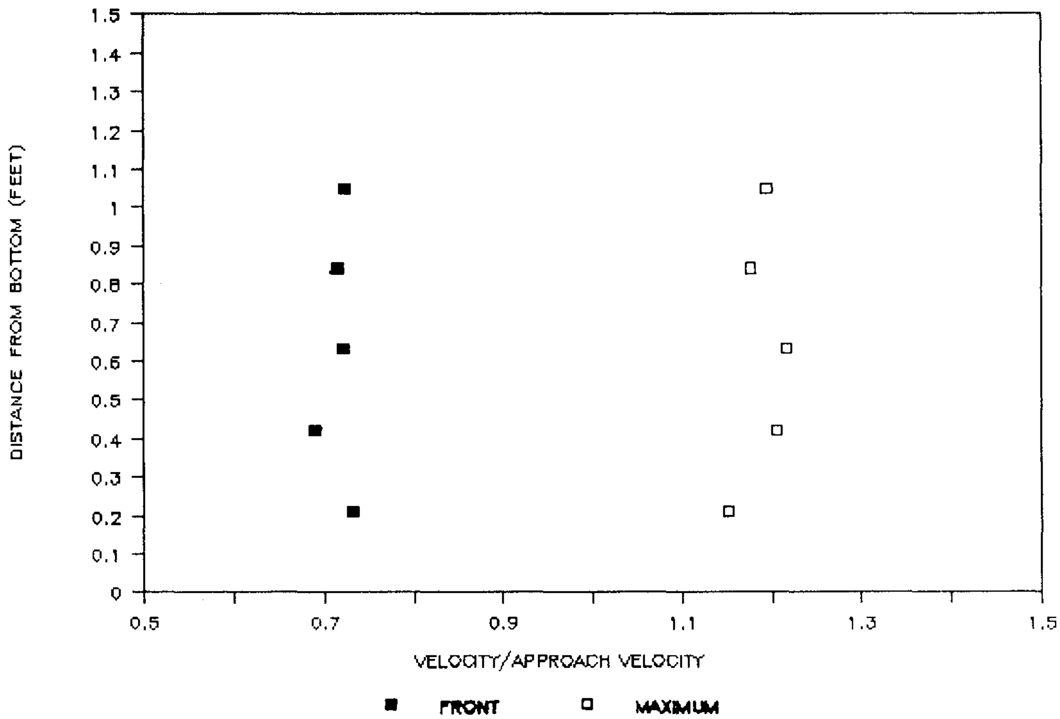


Figure 49. Velocity at highway pier 5 (run 4).

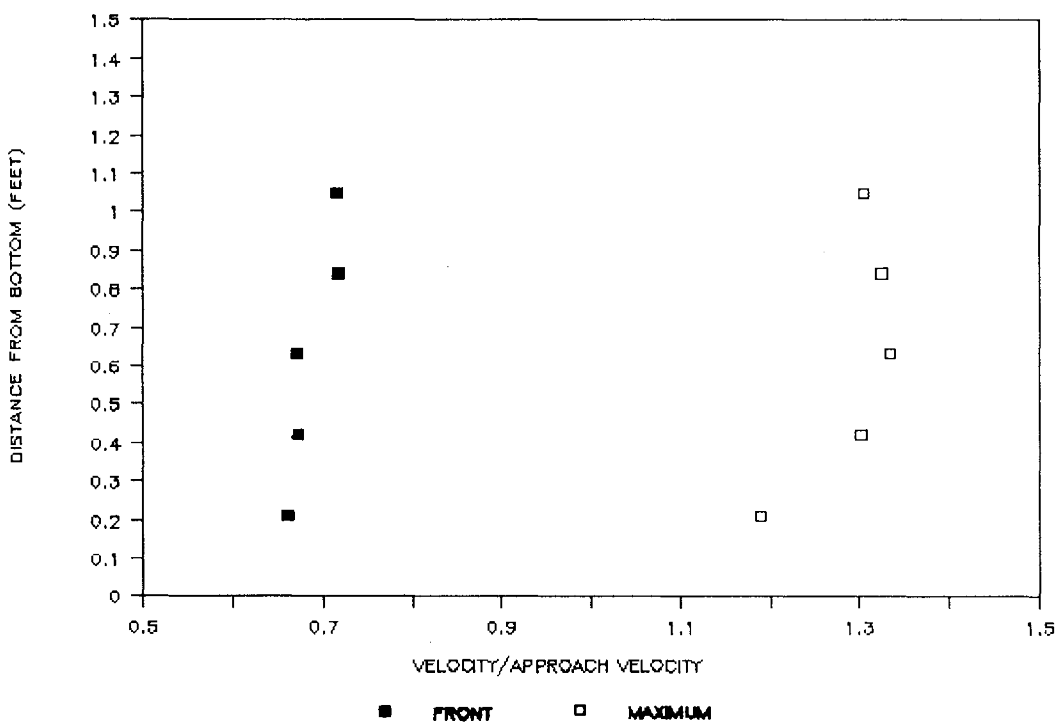


Figure 50. Velocity at highway pier 6 (run 4).

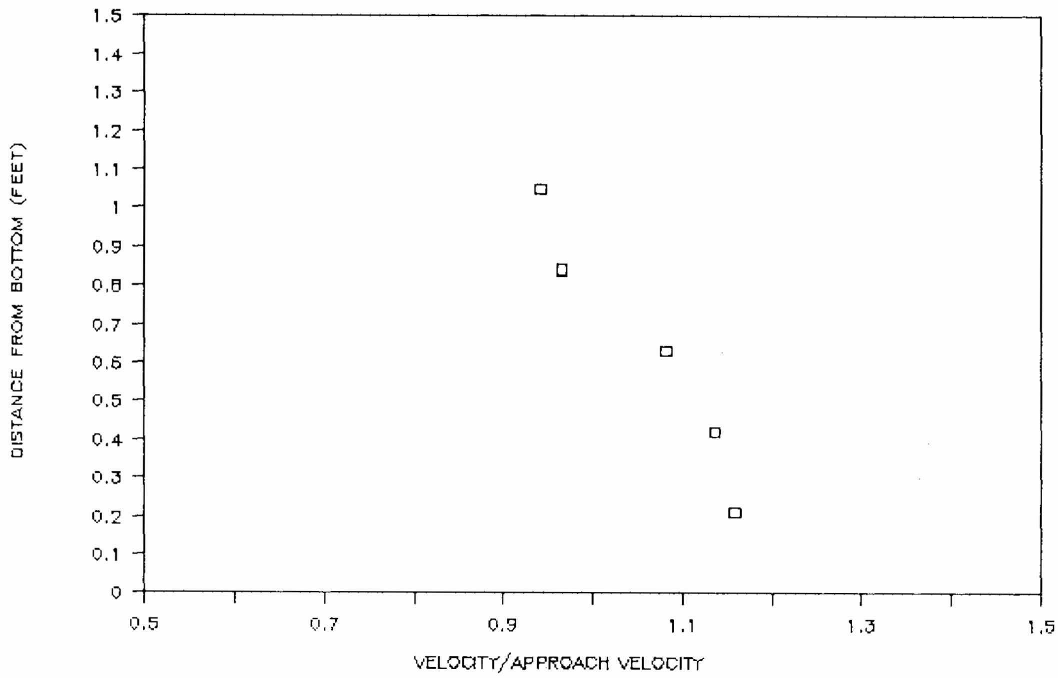


Figure 51. Velocity at railroad pier 7 (run 4).

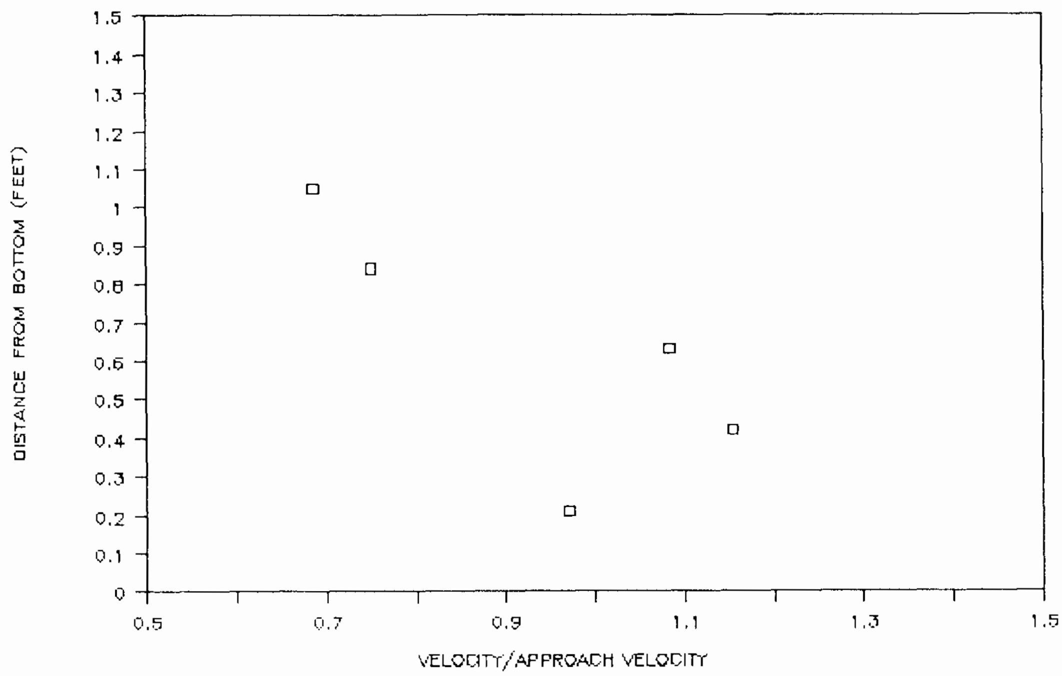


Figure 52. Velocity at railroad pier 8 (run 4).

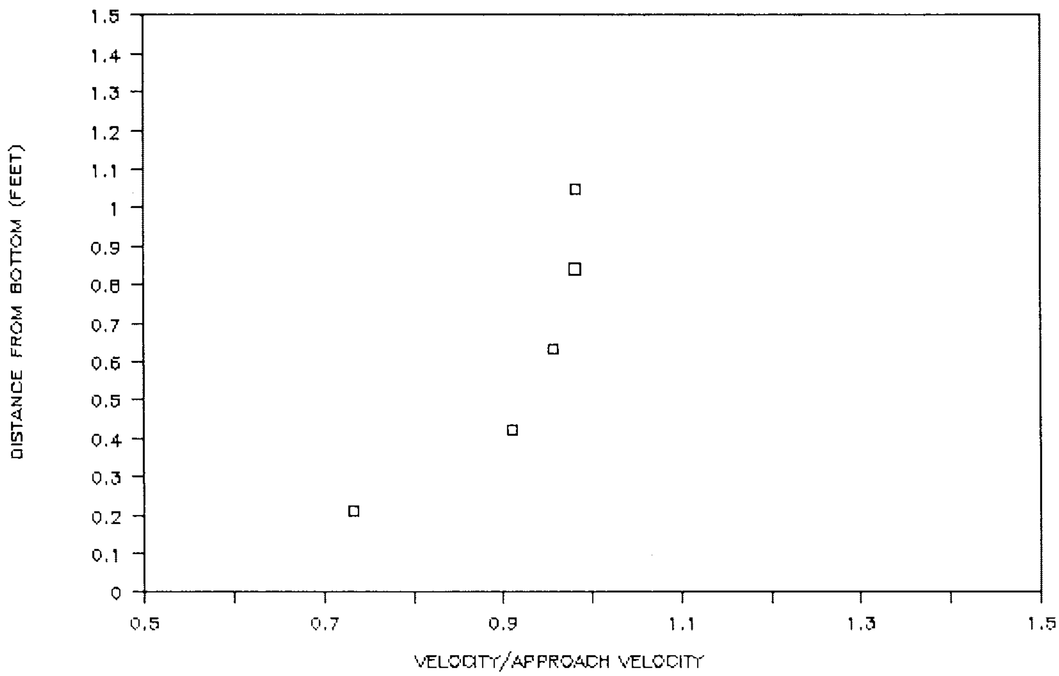


Figure 53. Velocity at center of highway piers 5 and 6 (run 3b).

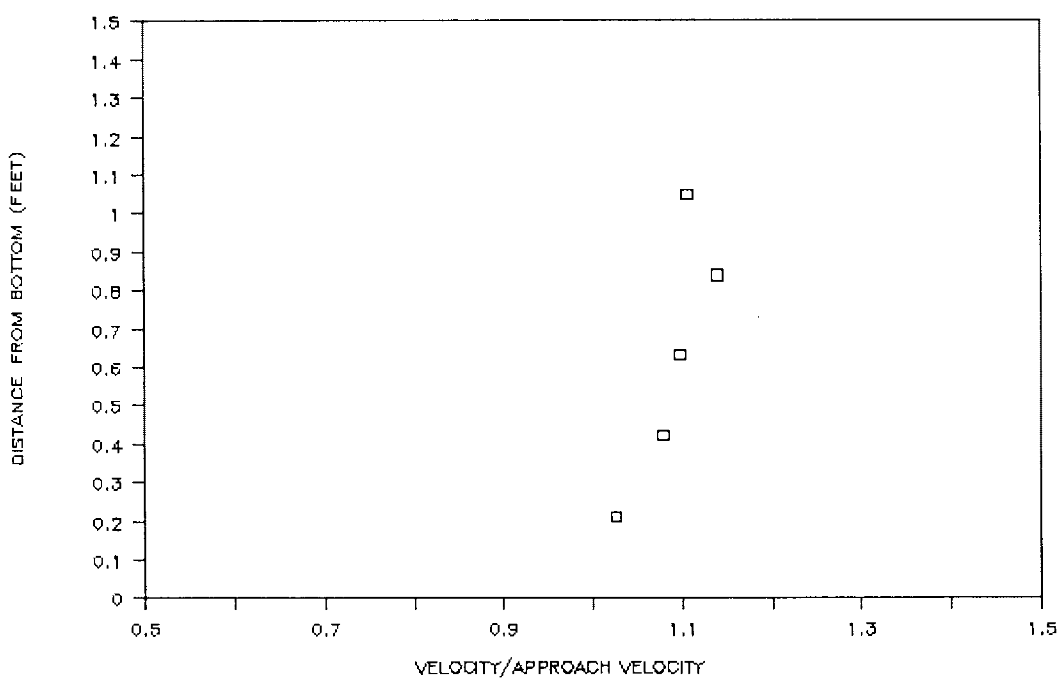


Figure 54. Velocity at center of railroad piers 7 and 8 (run 4).

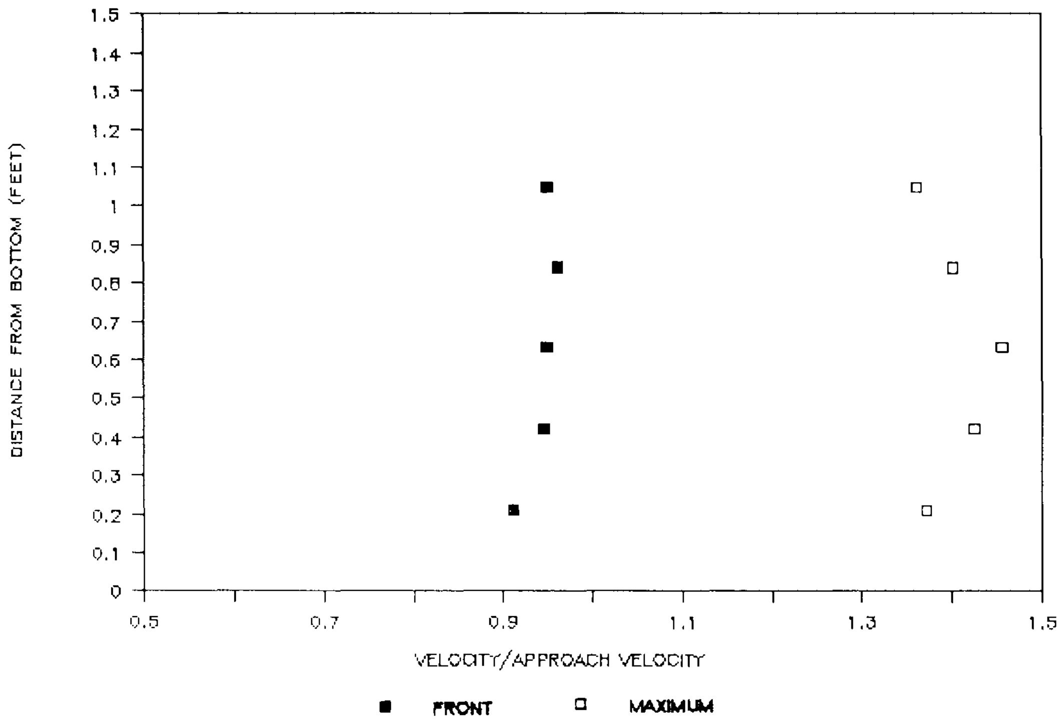


Figure 55. Velocity at highway pier 5 (run 5).

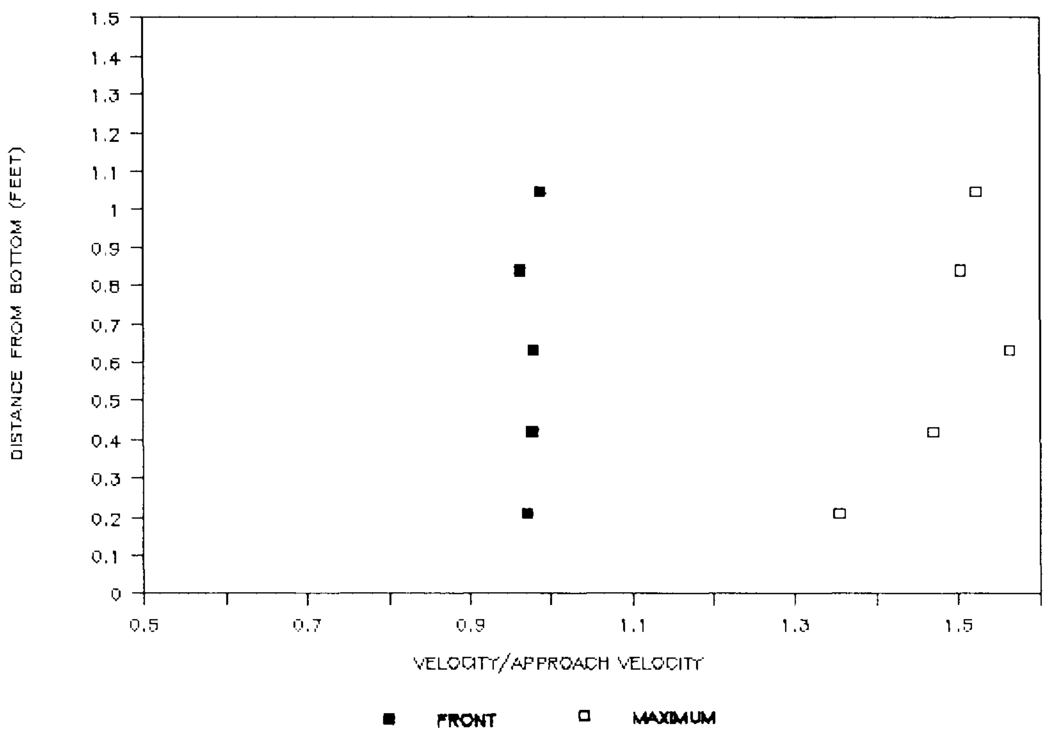


Figure 56. Velocity at highway pier 6 (run 5).

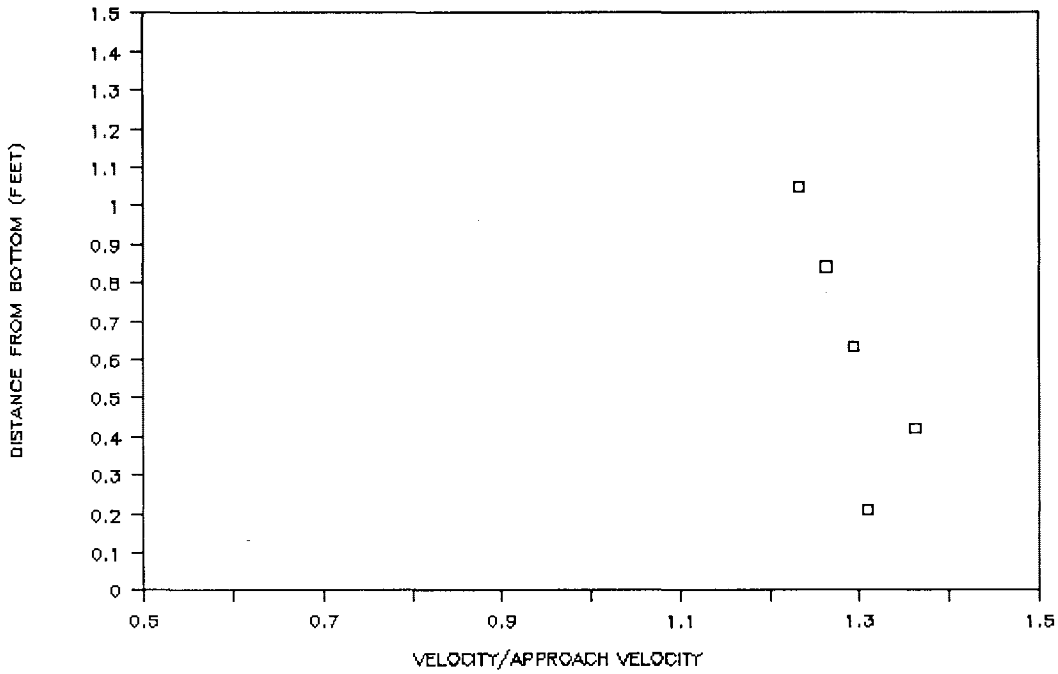


Figure 57. Velocity at railroad pier 7 (run 5).

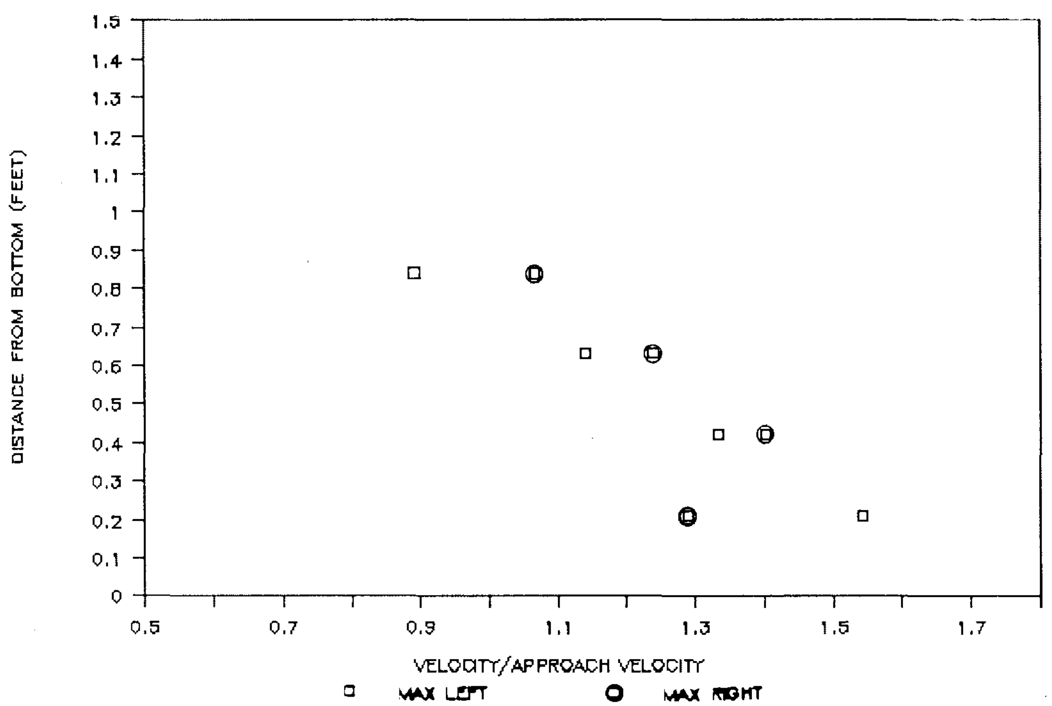


Figure 58. Velocity at railroad pier 8 (run 5).

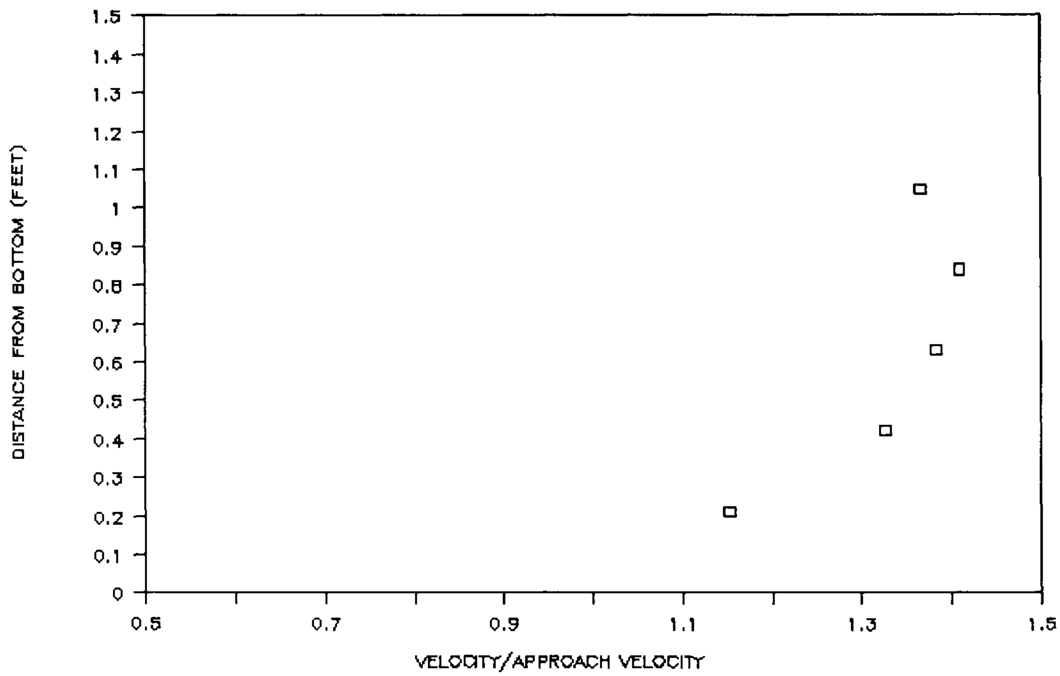


Figure 59. Velocity at center of railroad piers 7 and 8 (run 5).

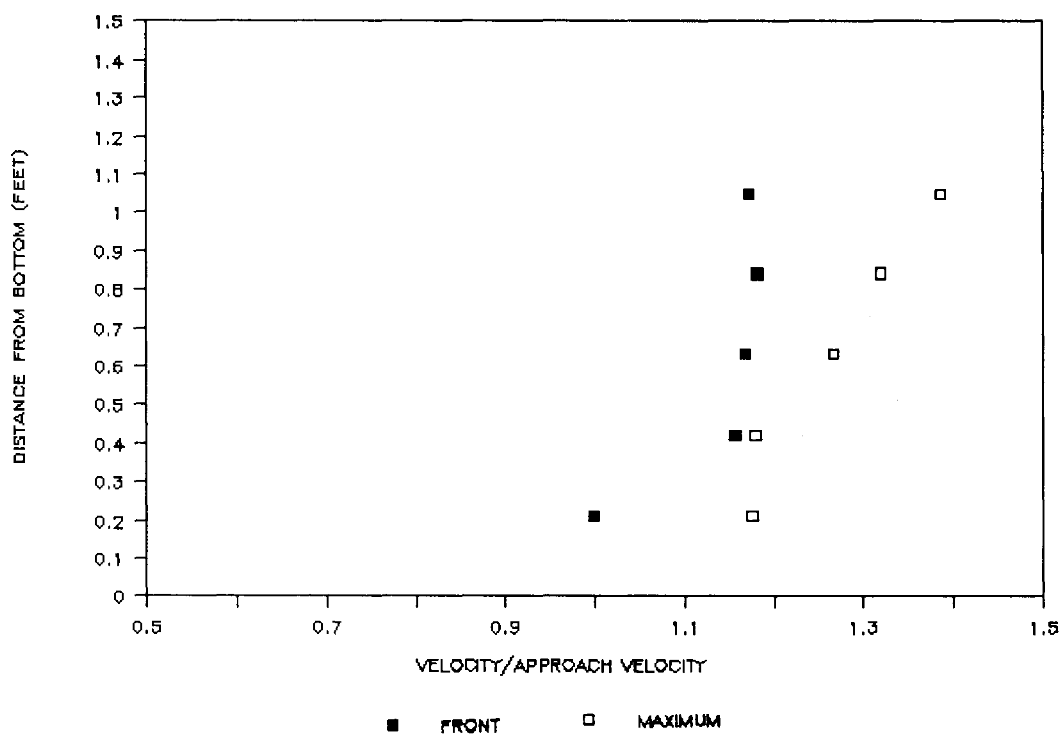


Figure 60. Velocity at railroad pier 8 (run 6).

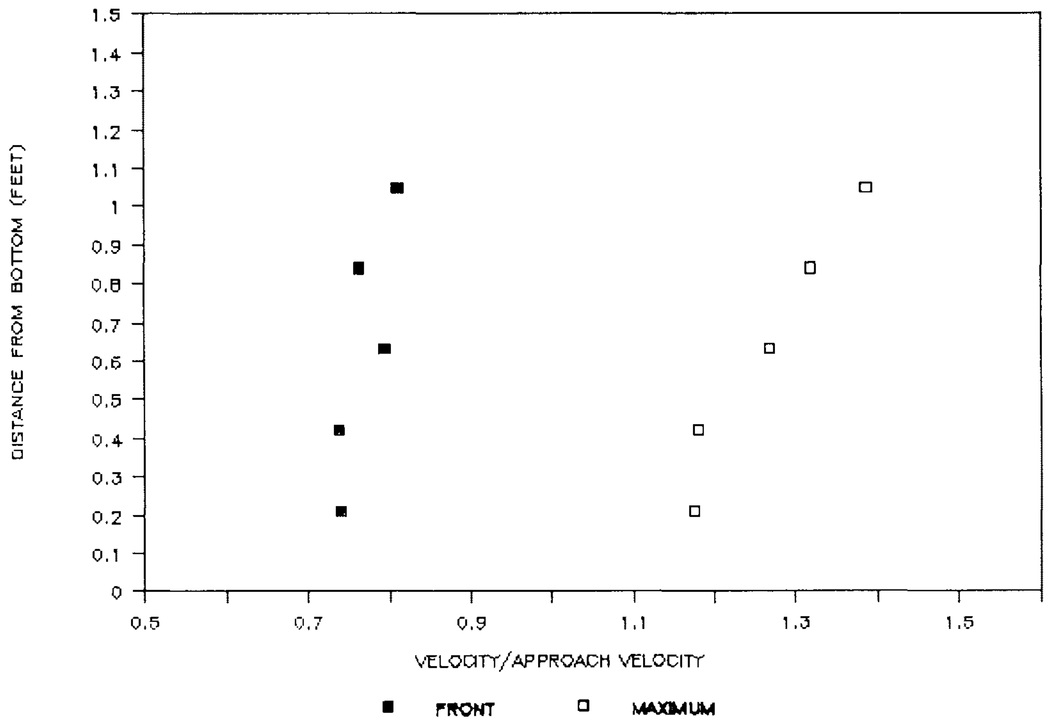


Figure 61. Velocity at railroad pier 8 (run 7).

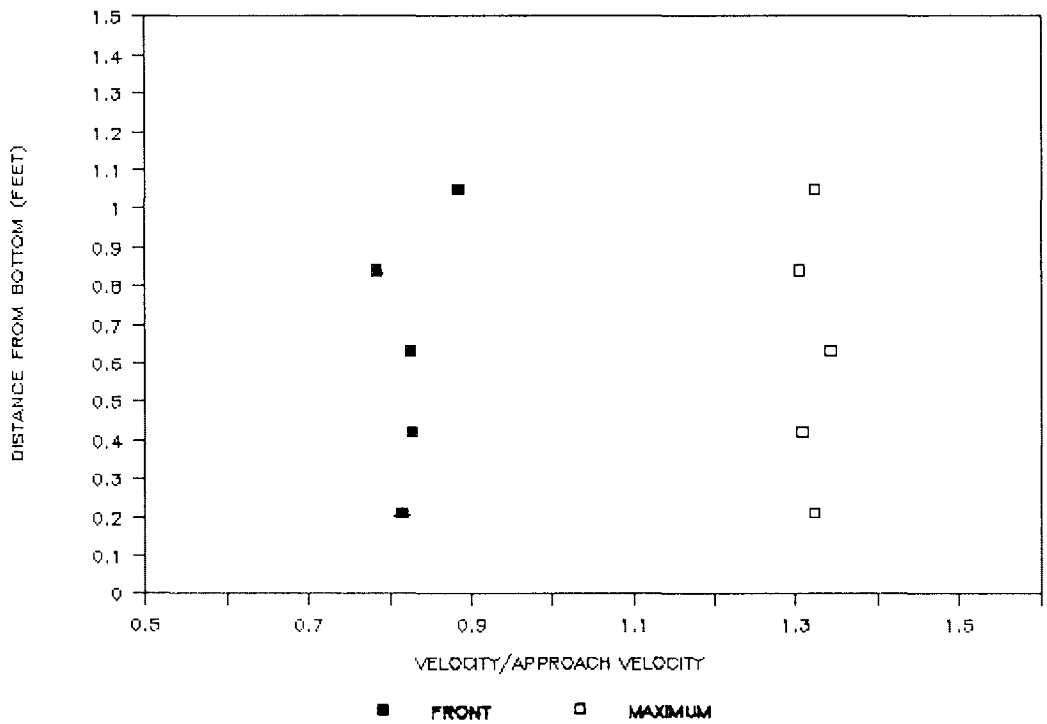


Figure 62. Velocity at railroad pier 8 (run 7b).

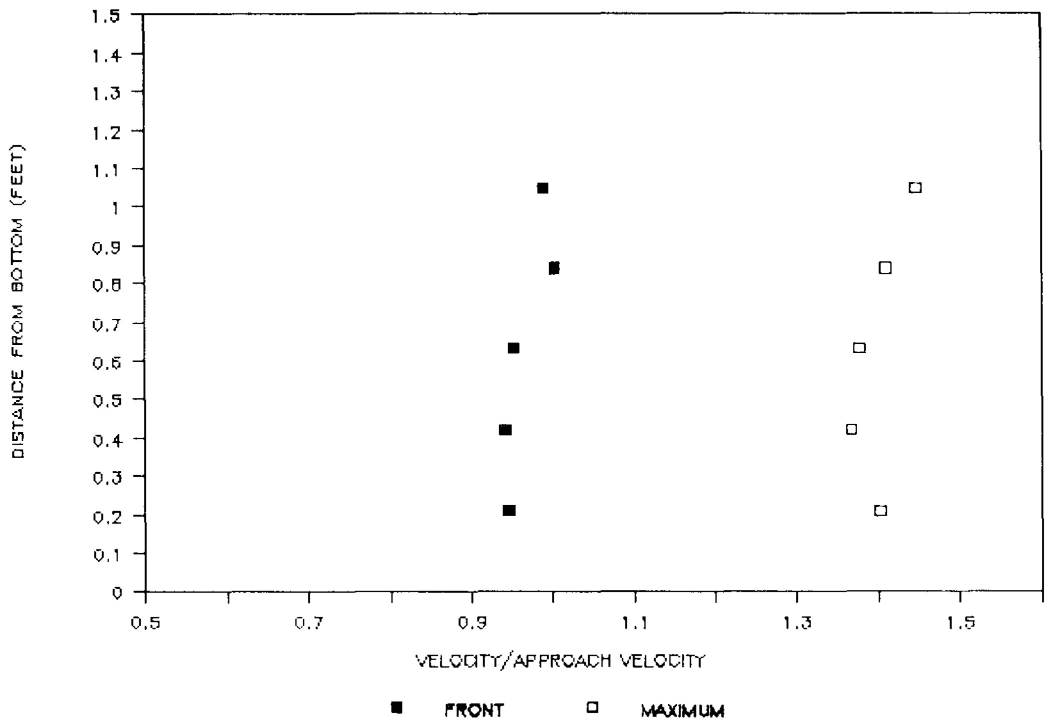


Figure 63. Velocity at railroad pier 8 (run 7c).

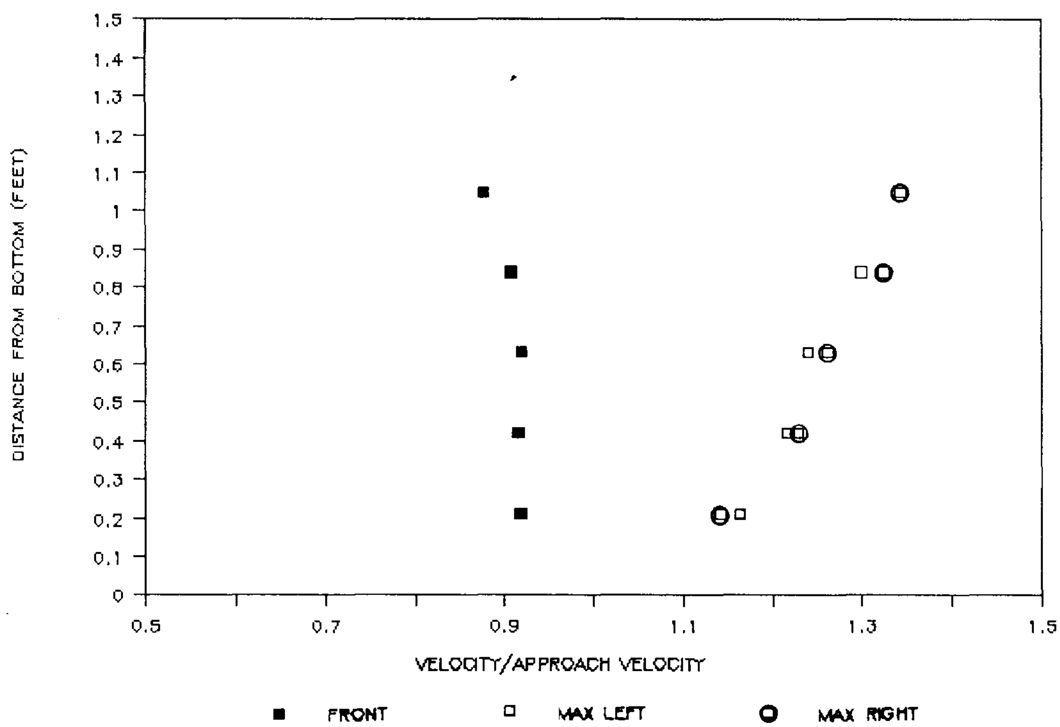


Figure 64. Velocity at proposed steel pier (run 8).

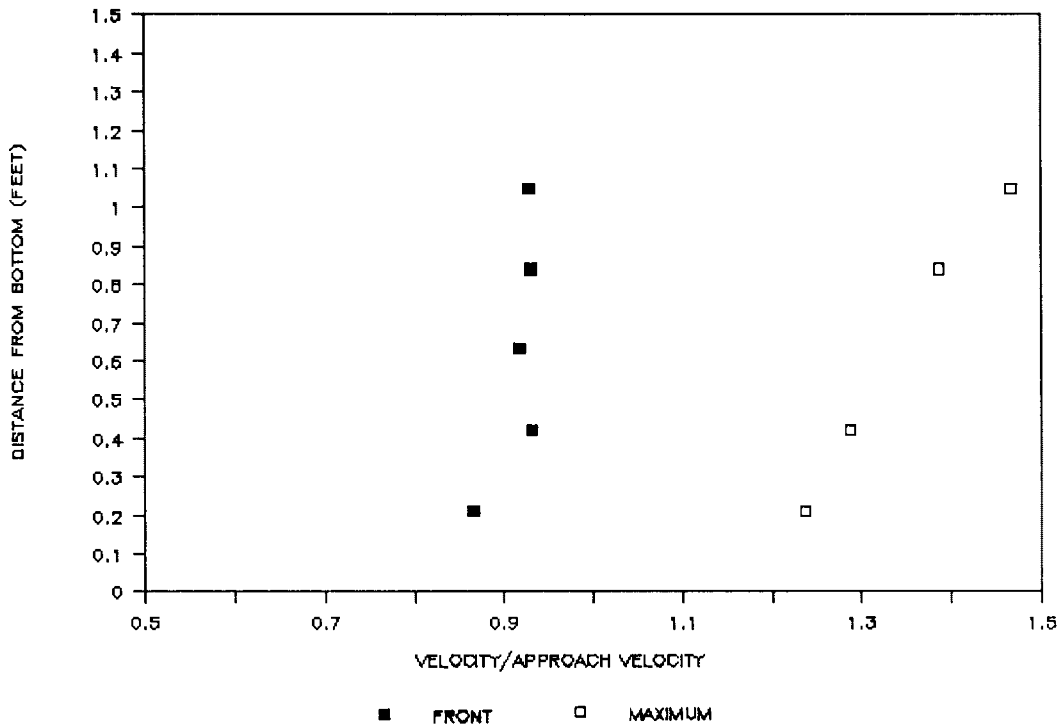


Figure 65. Velocity at proposed steel pier (run 9).

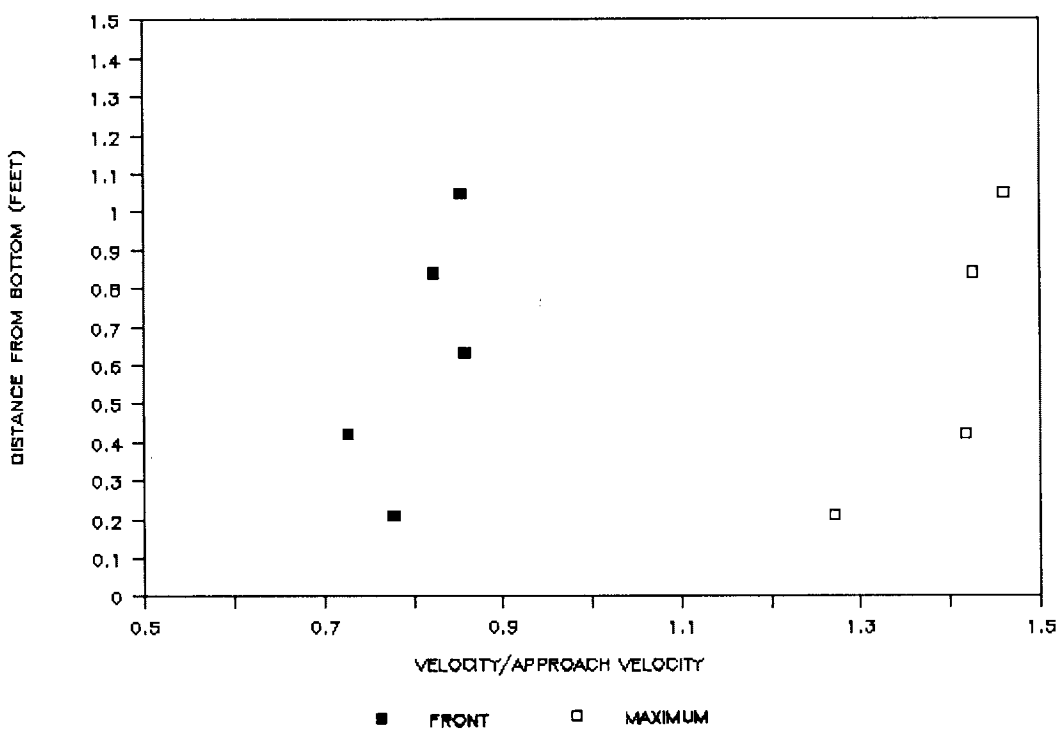


Figure 66. Velocity at proposed steel pier (run 9b).

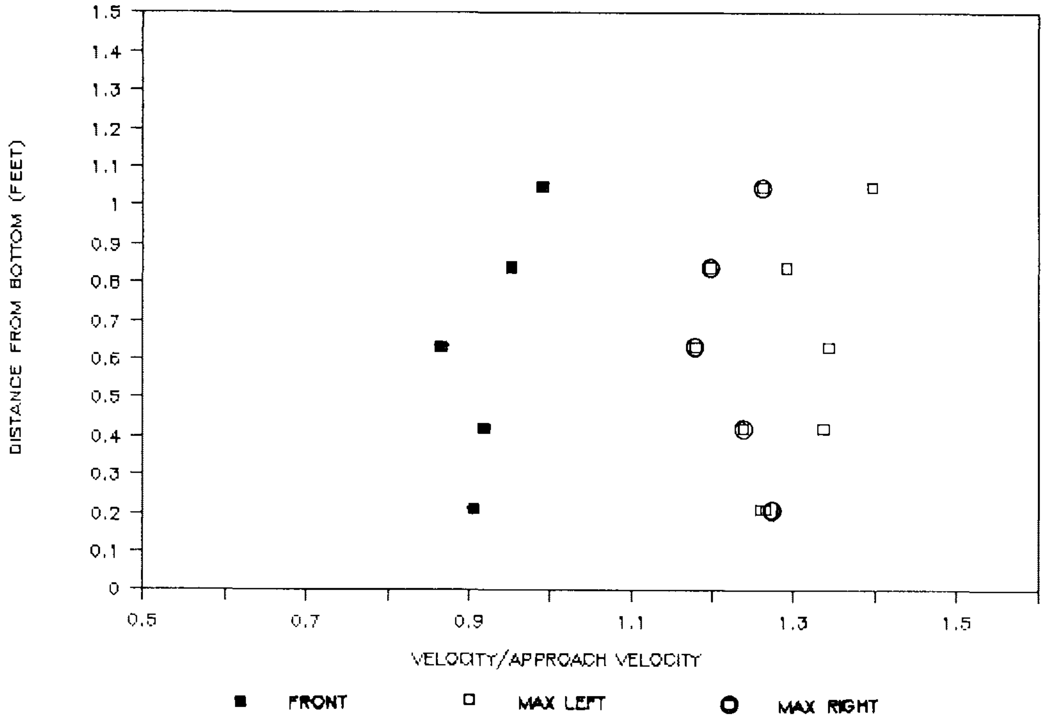


Figure 67. Velocity at railroad pier 8 (run 10).

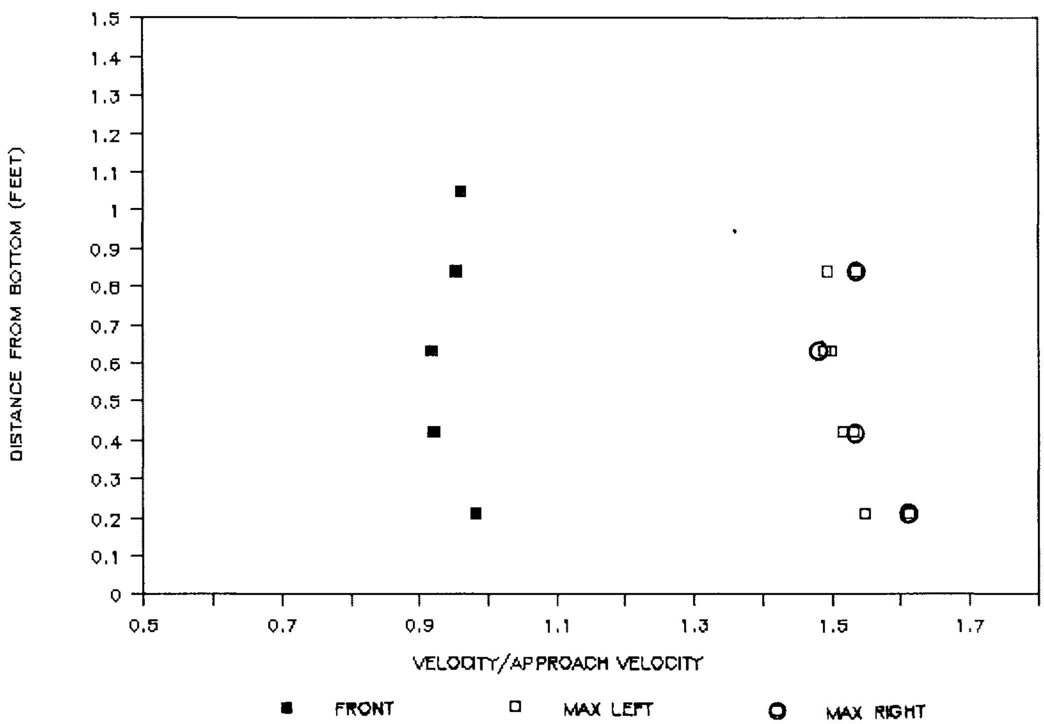


Figure 68. Velocity at railroad pier 8 (run 11).

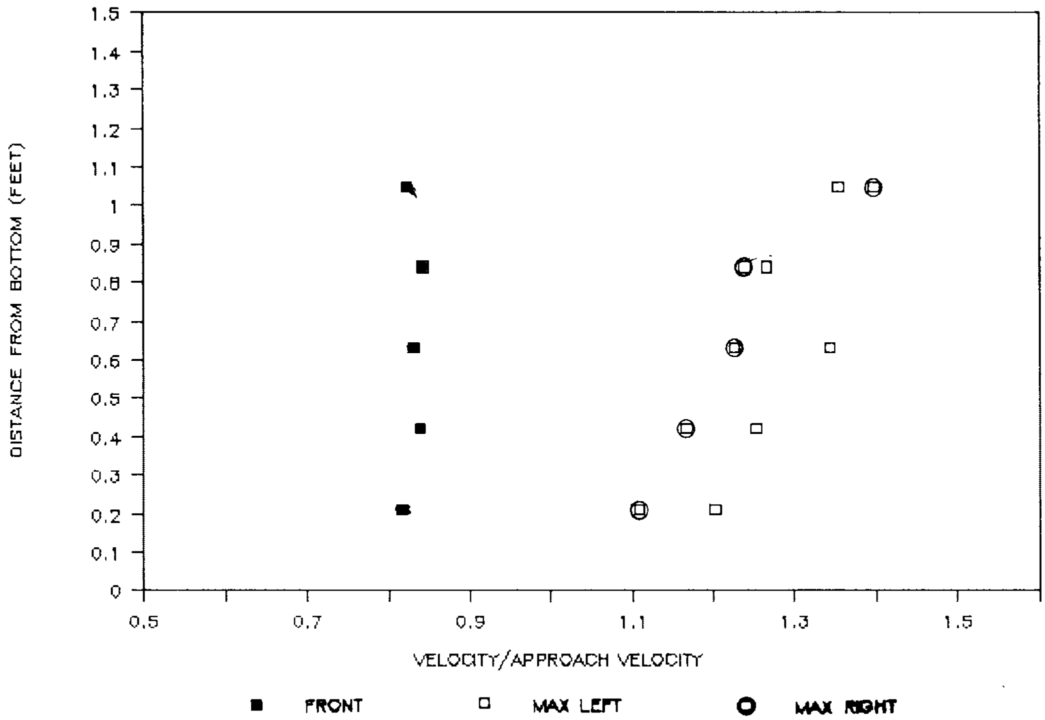


Figure 69. Velocity at railroad pier 7 (run 12a).

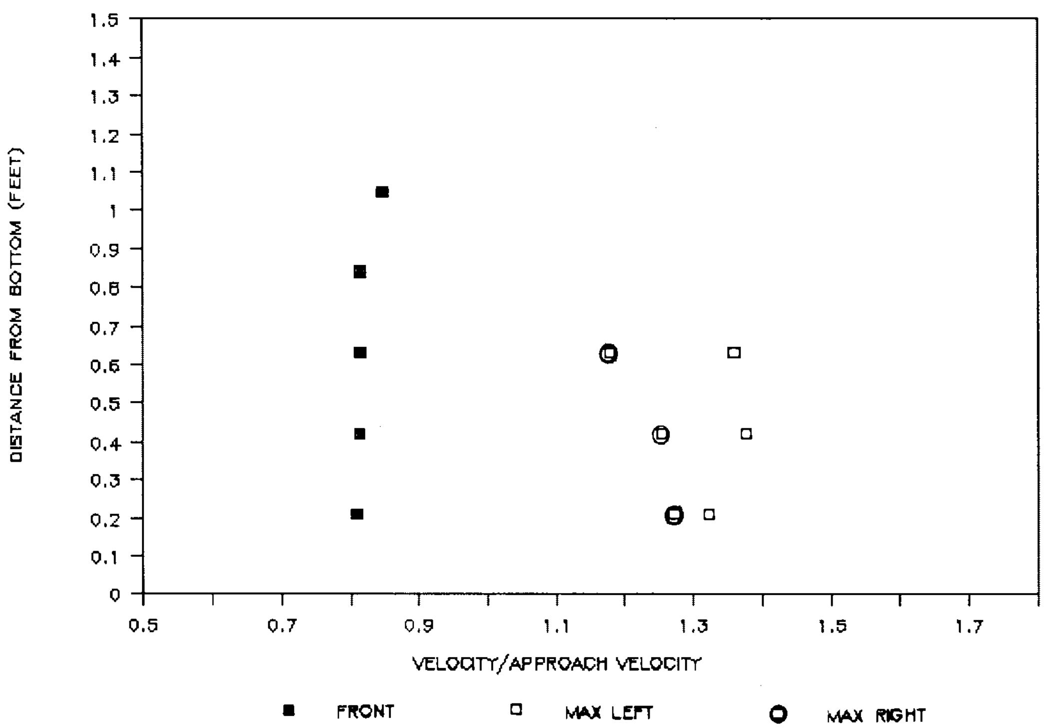


Figure 70. Velocity at railroad pier 9 (run 12a).

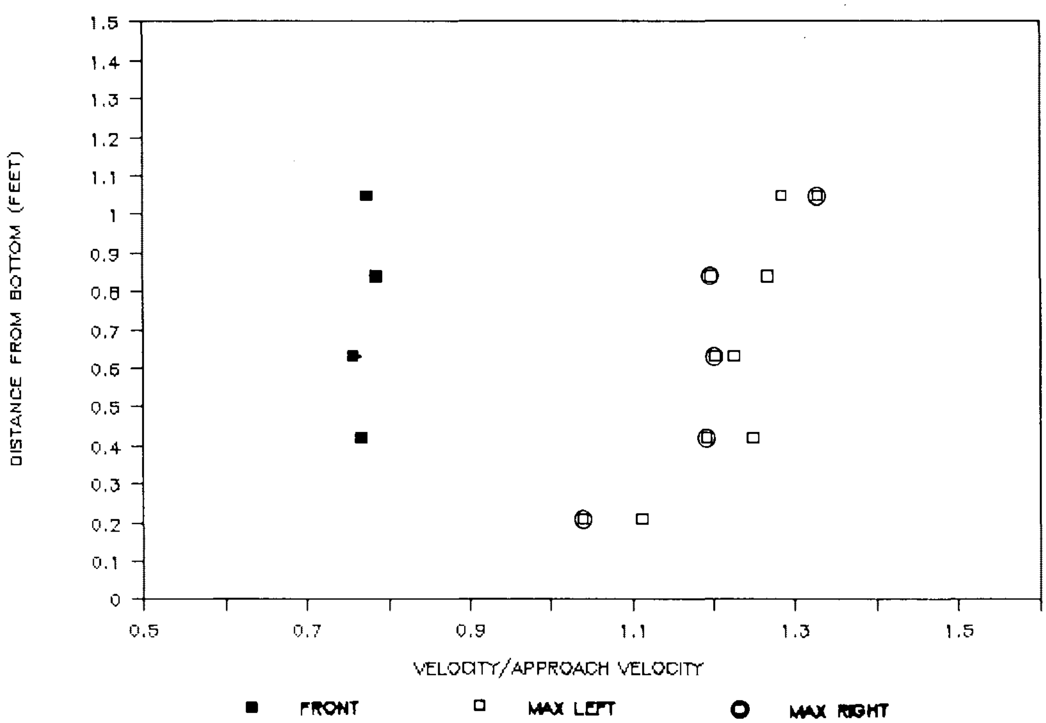


Figure 71. Velocity at railroad pier 7 (run 12b).

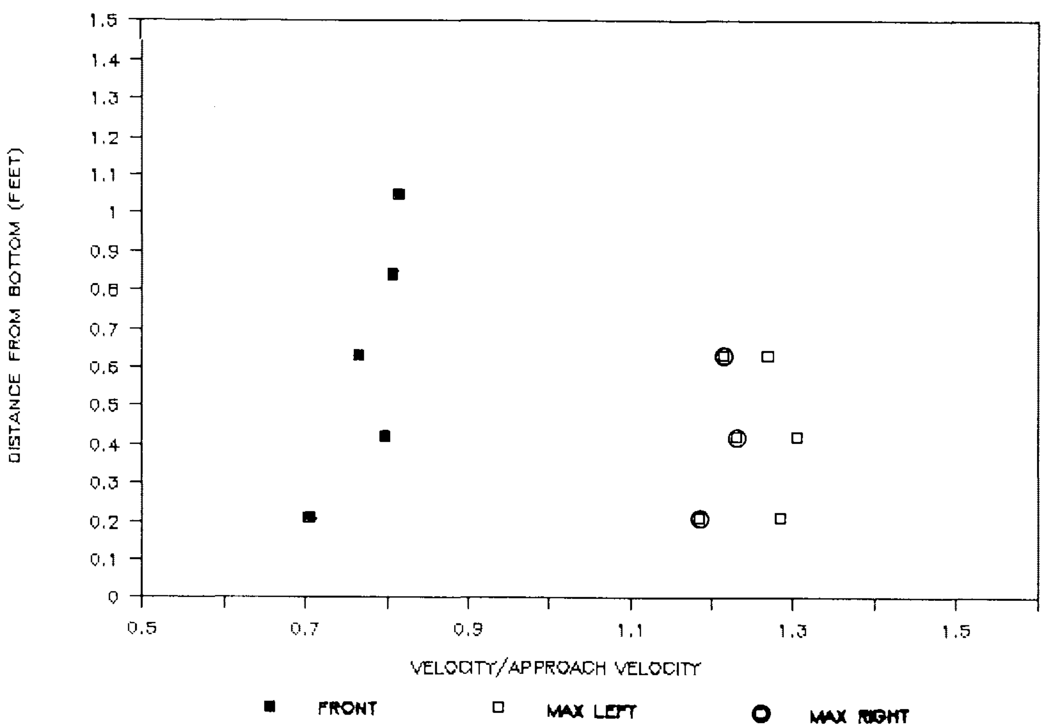


Figure 72. Velocity at railroad pier 9 (run 12b).

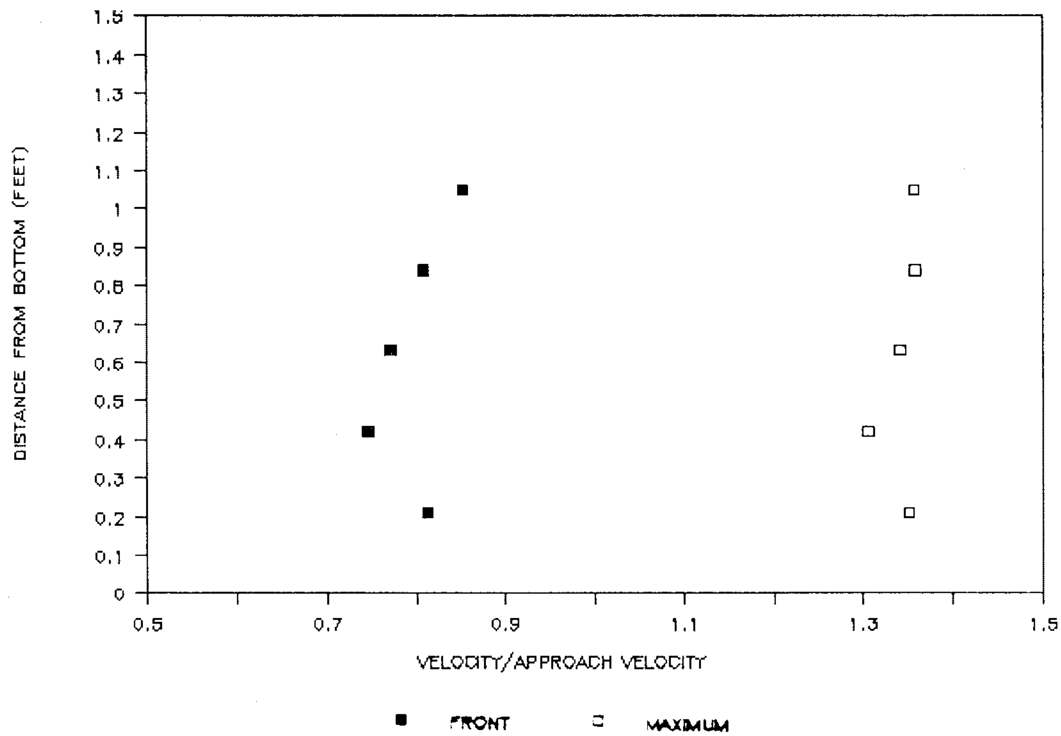


Figure 73. Velocity at railroad pier 9 (run 13).

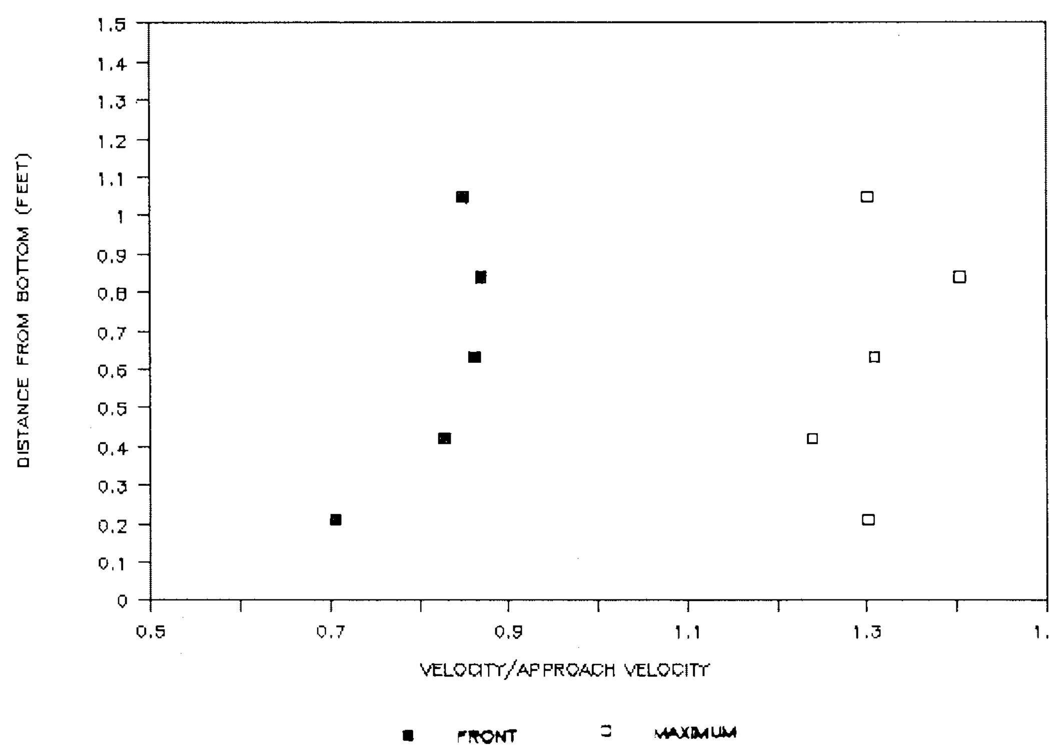


Figure 74. Velocity at railroad pier 9 (run 14).

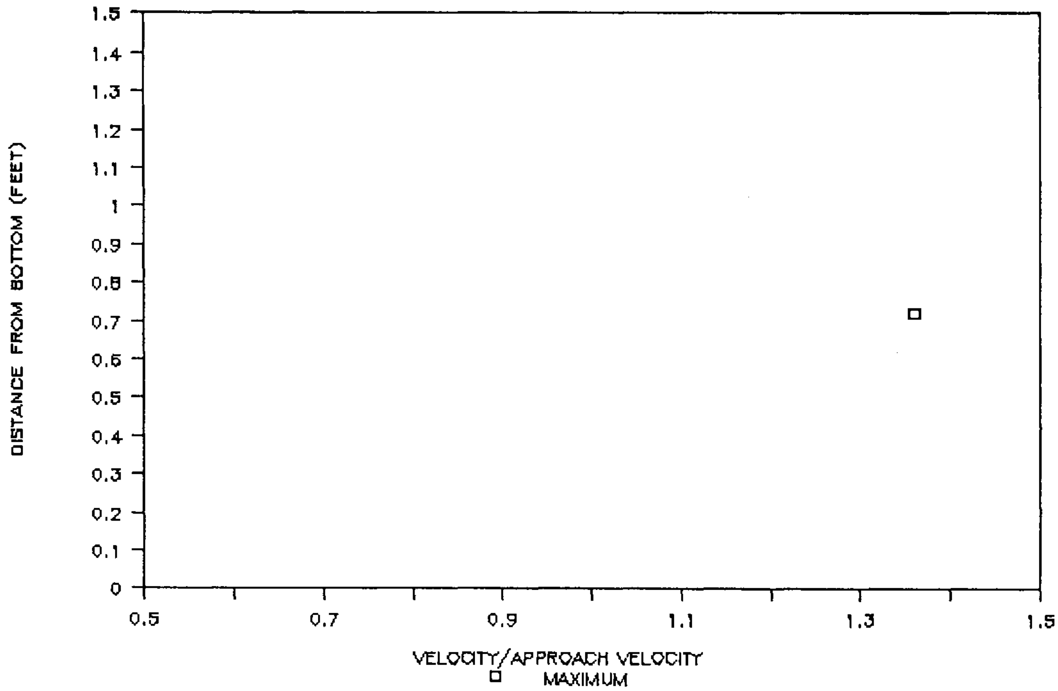


Figure 75. Velocity at railroad pier 9 (run 15a).

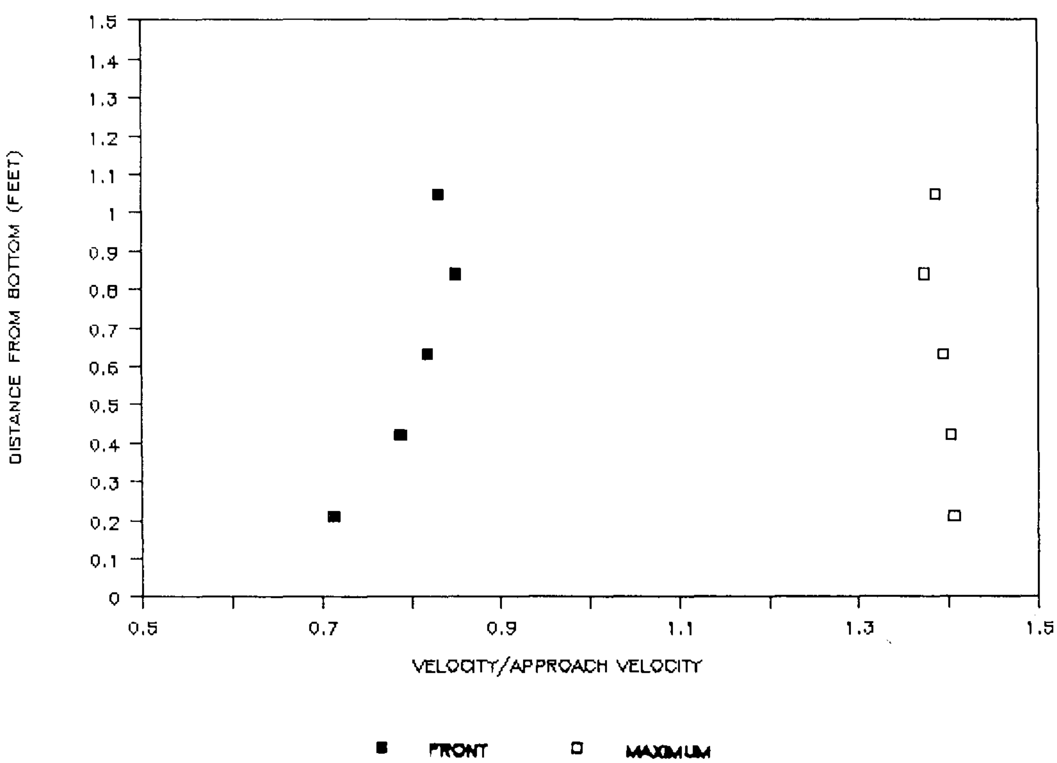


Figure 76. Velocity at railroad pier 9 (run 15b).

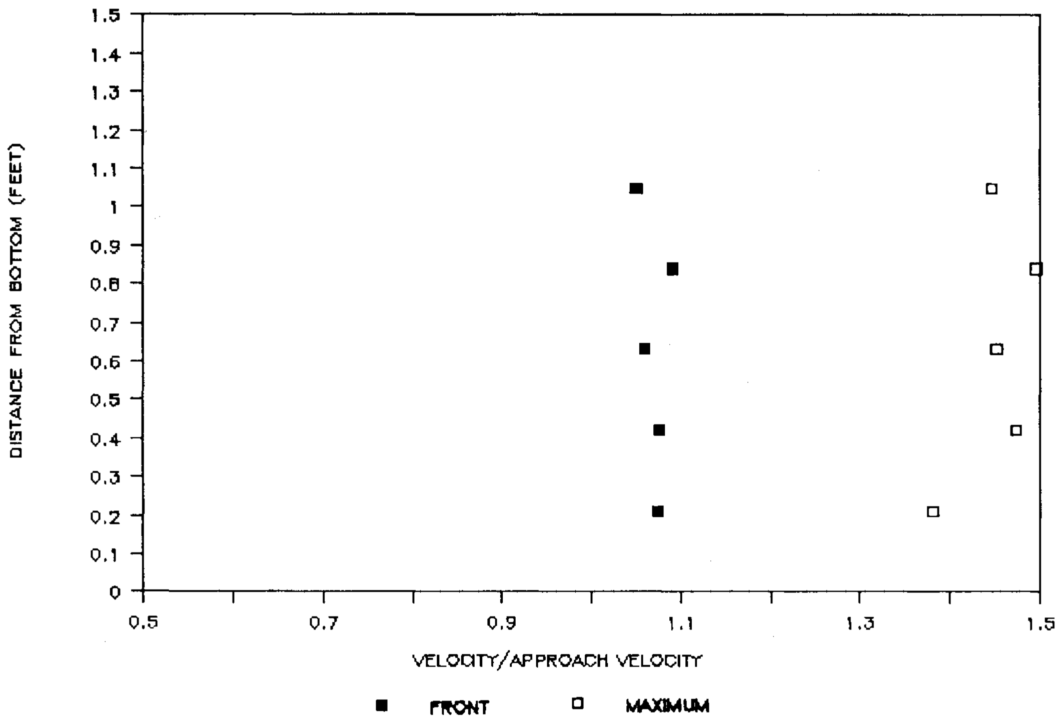


Figure 77. Velocity at highway pier 5 (run 16).

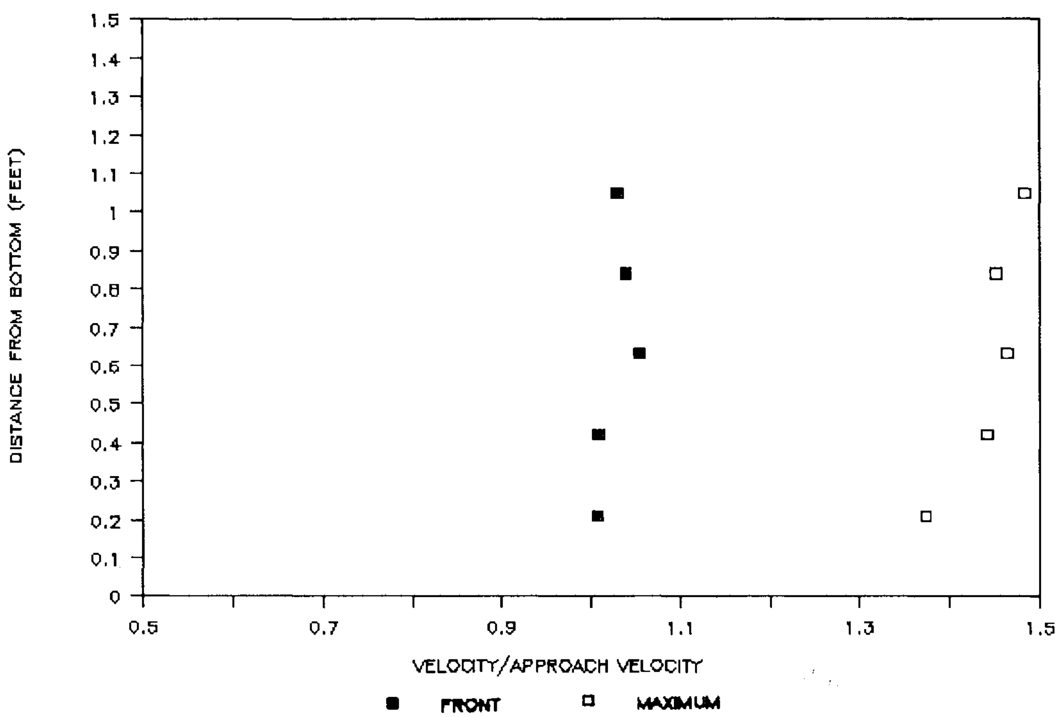


Figure 78. Velocity at highway pier 6 (run 16).

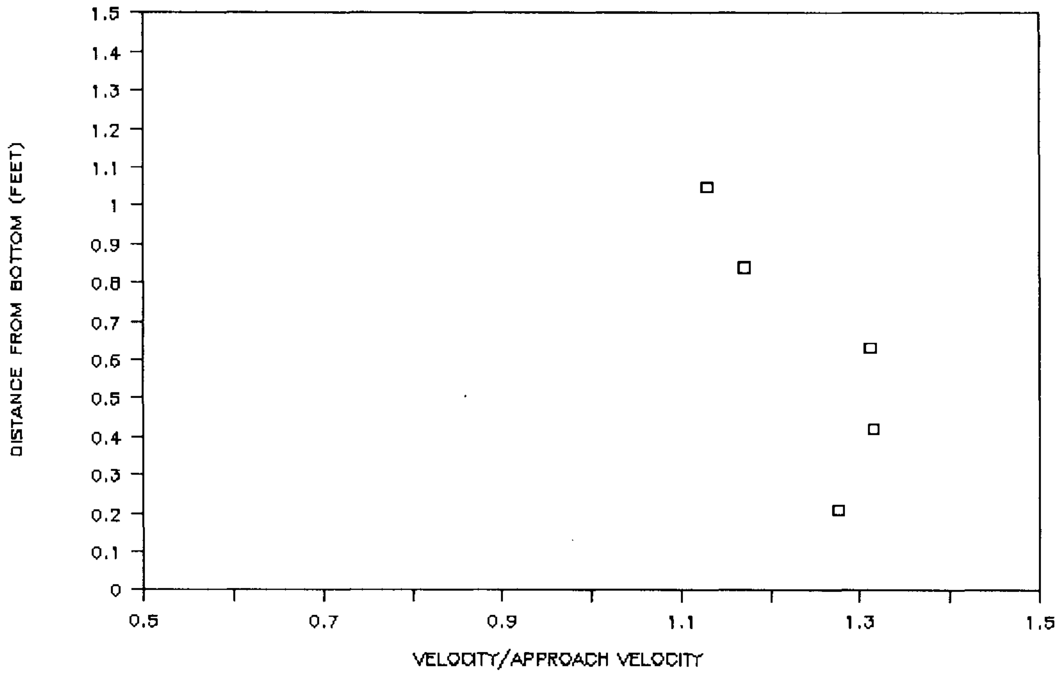


Figure 79. Velocity at railroad pier 7 (run 16)

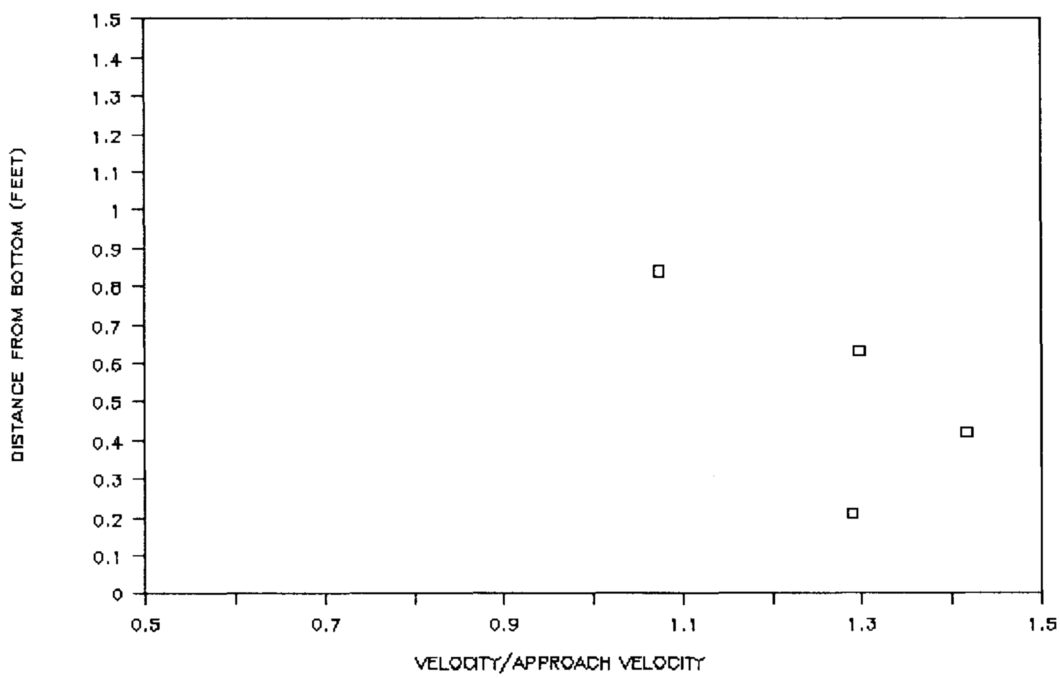


Figure 80. Velocity at railroad pier 8 (run 16).

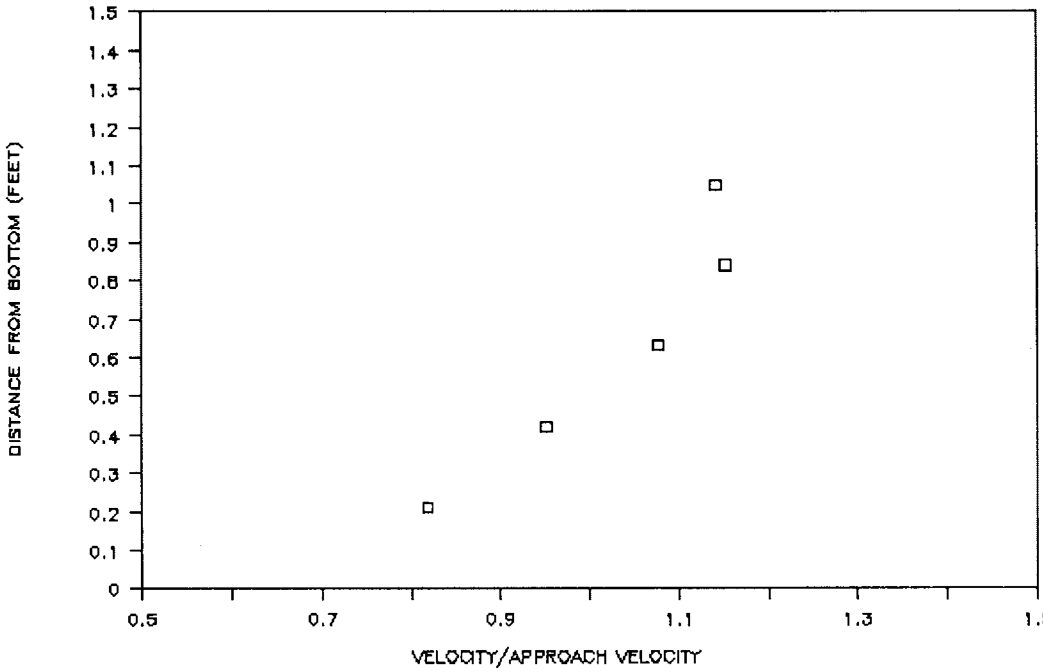


Figure 81. Velocity at center of highway piers 5 and 6 (run 16).

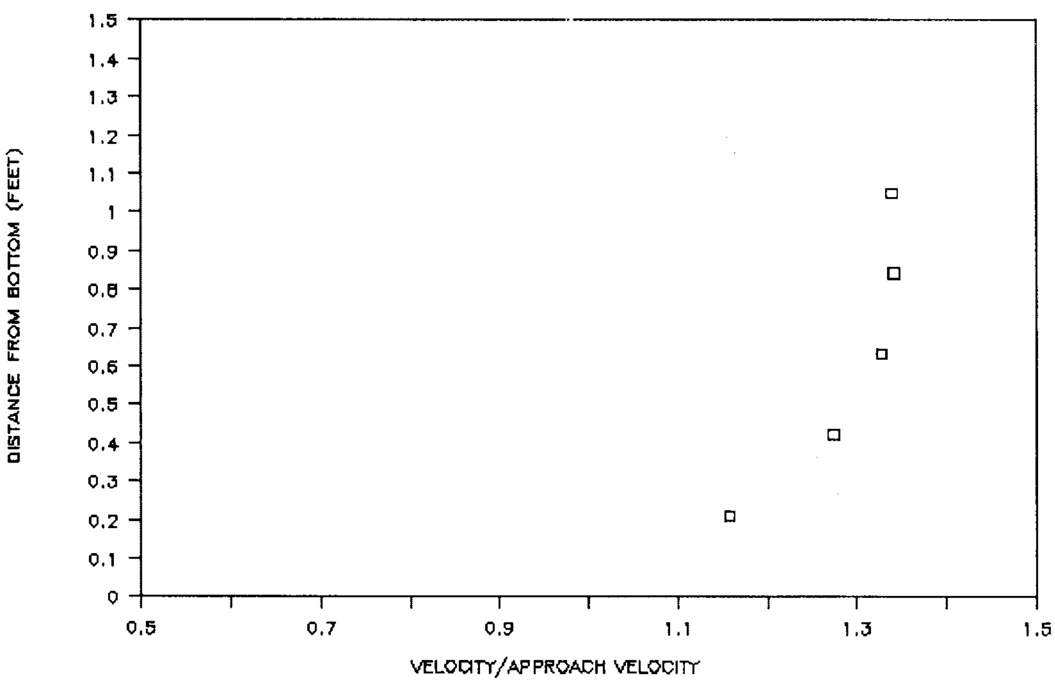


Figure 82. Velocity at center of railroad piers 7 and 8 (run 16).

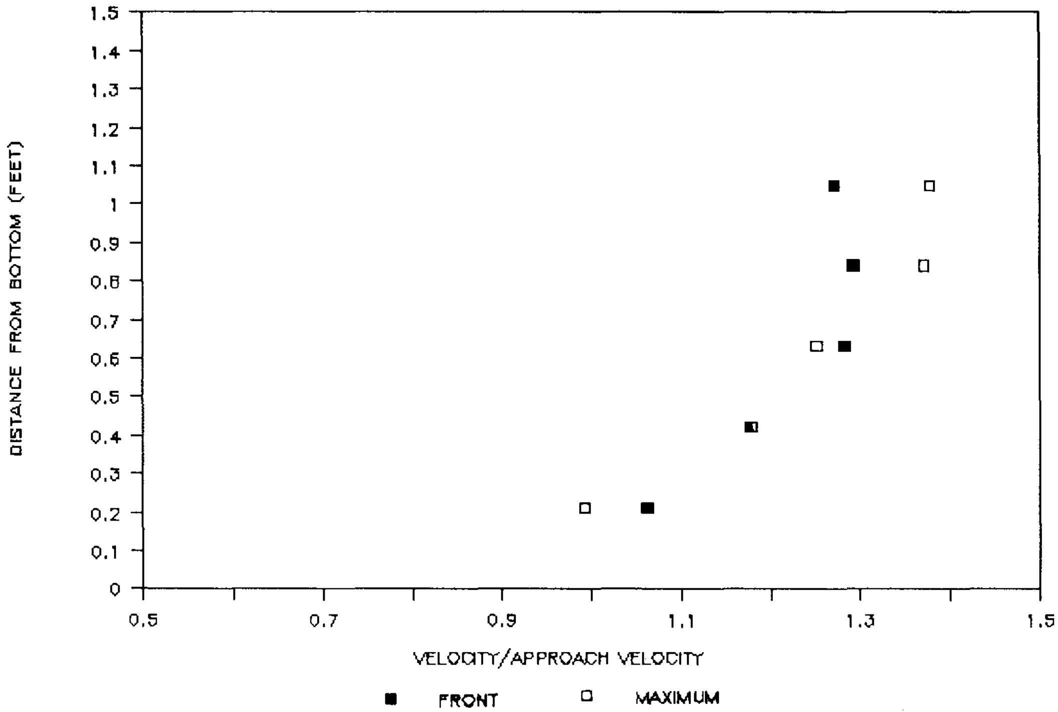


Figure 83. Velocity at railroad pier 7 (run 17).

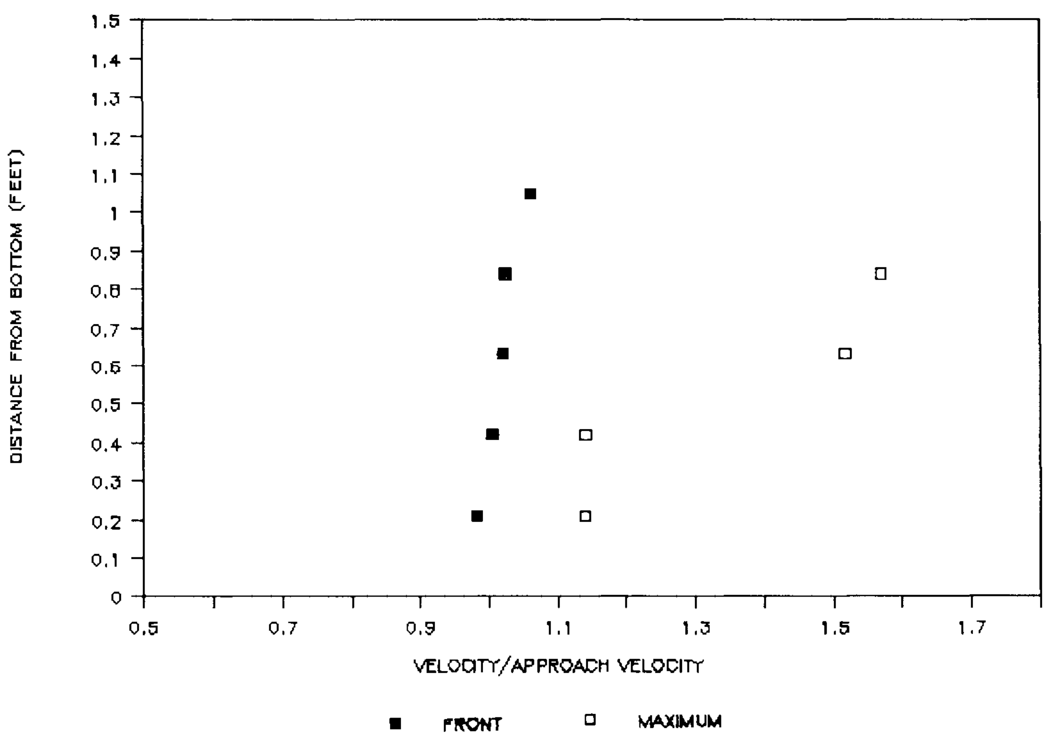


Figure 84. Velocity at railroad pier 8 (run 17).

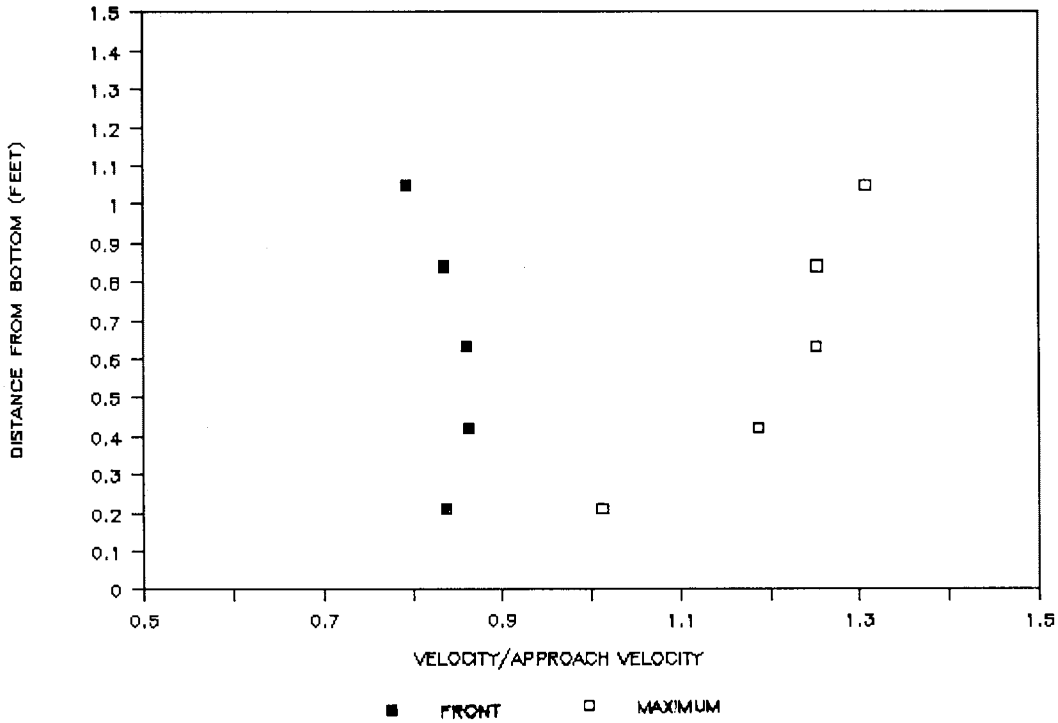


Figure 85. Velocity at concrete pier (run 18).

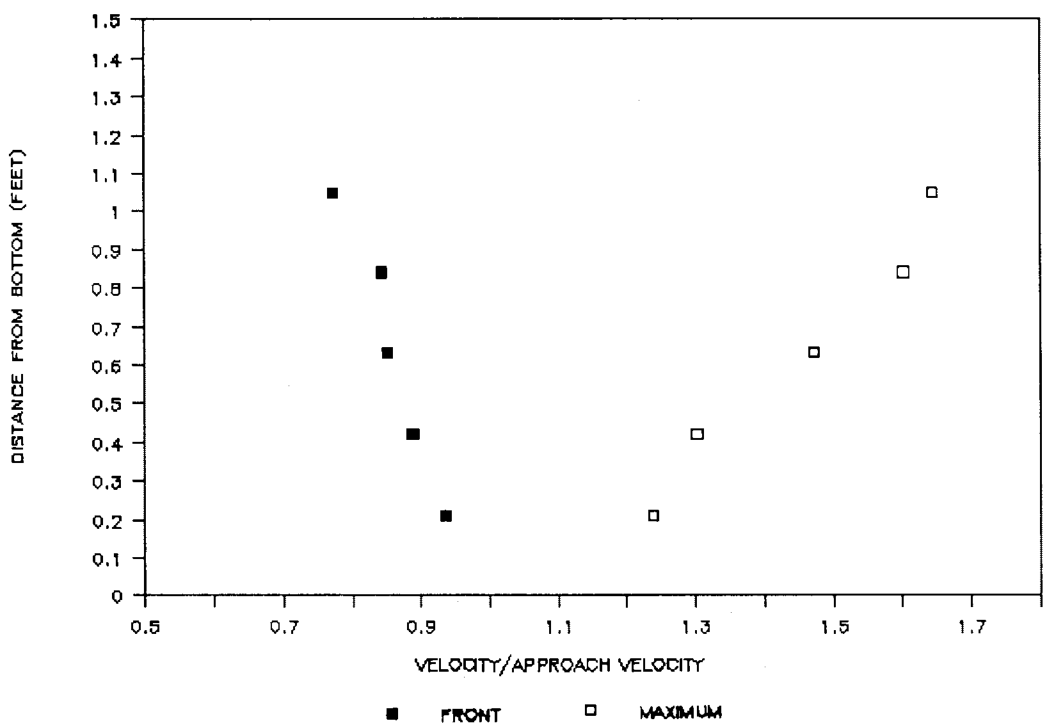


Figure 86. Velocity at concrete pier (run 19).

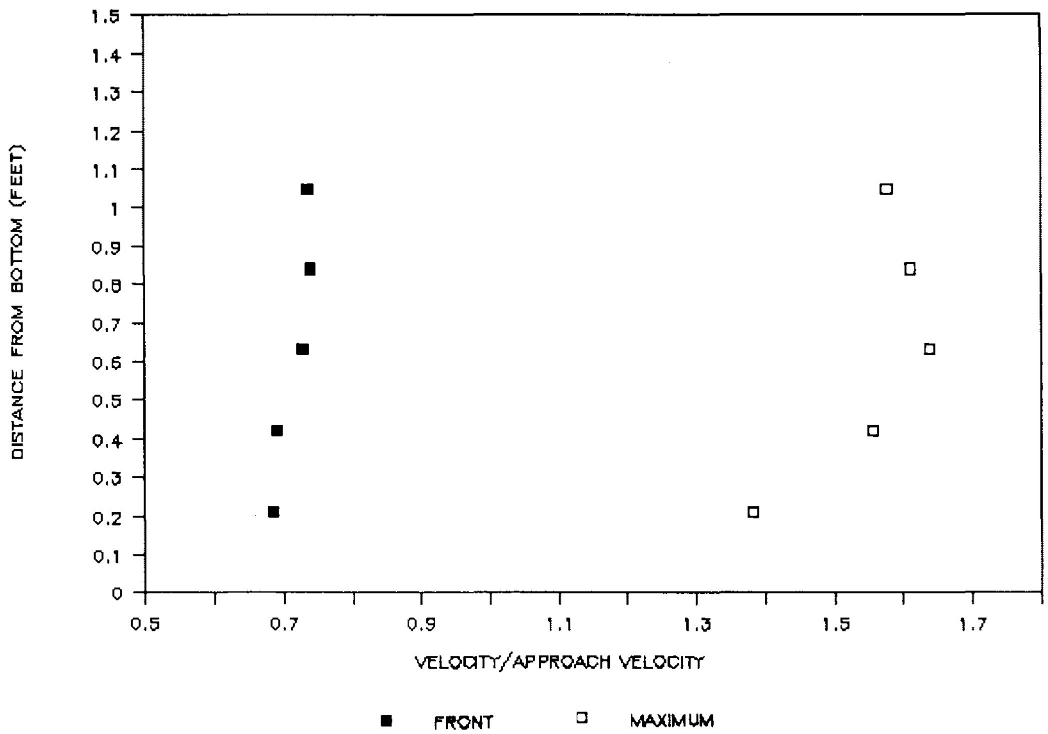


Figure 87. Velocity at concrete pier (run 20a).

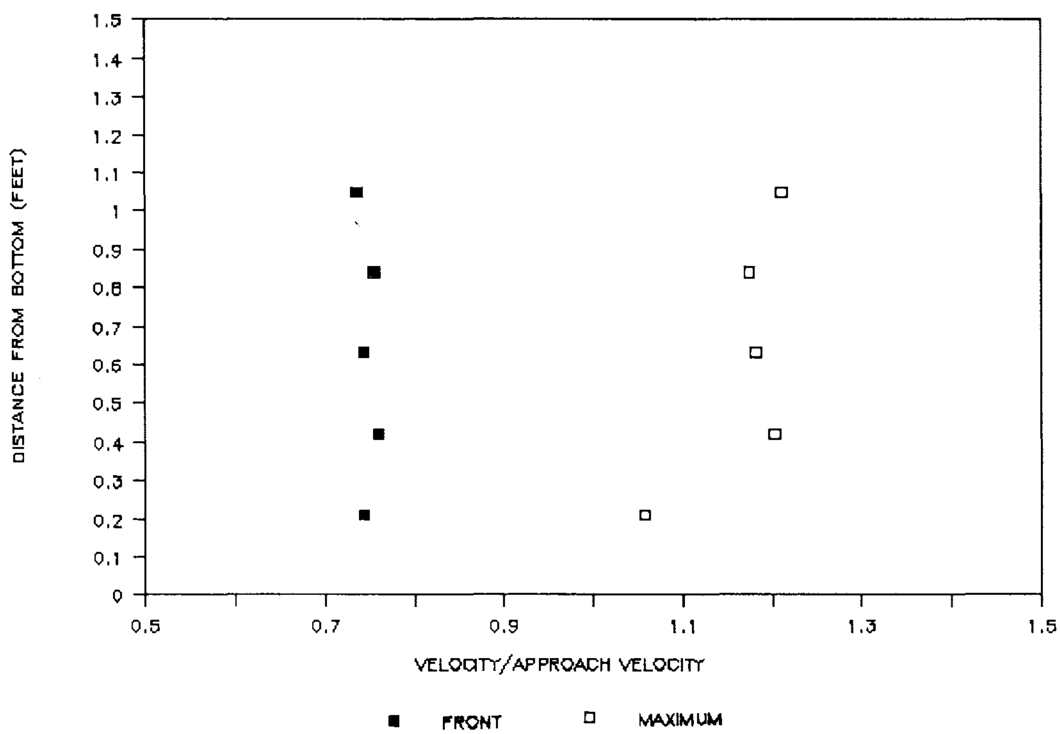


Figure 88. Velocity concrete pier (run 20b).

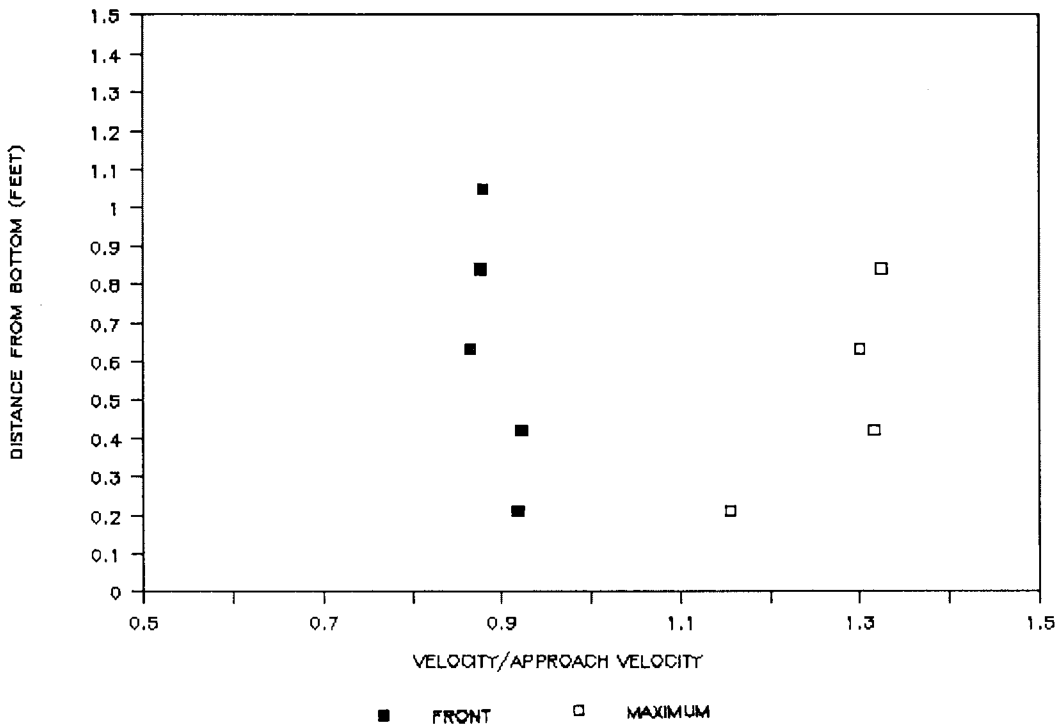


Figure 89. Velocity at modified concrete pier (run 21a).

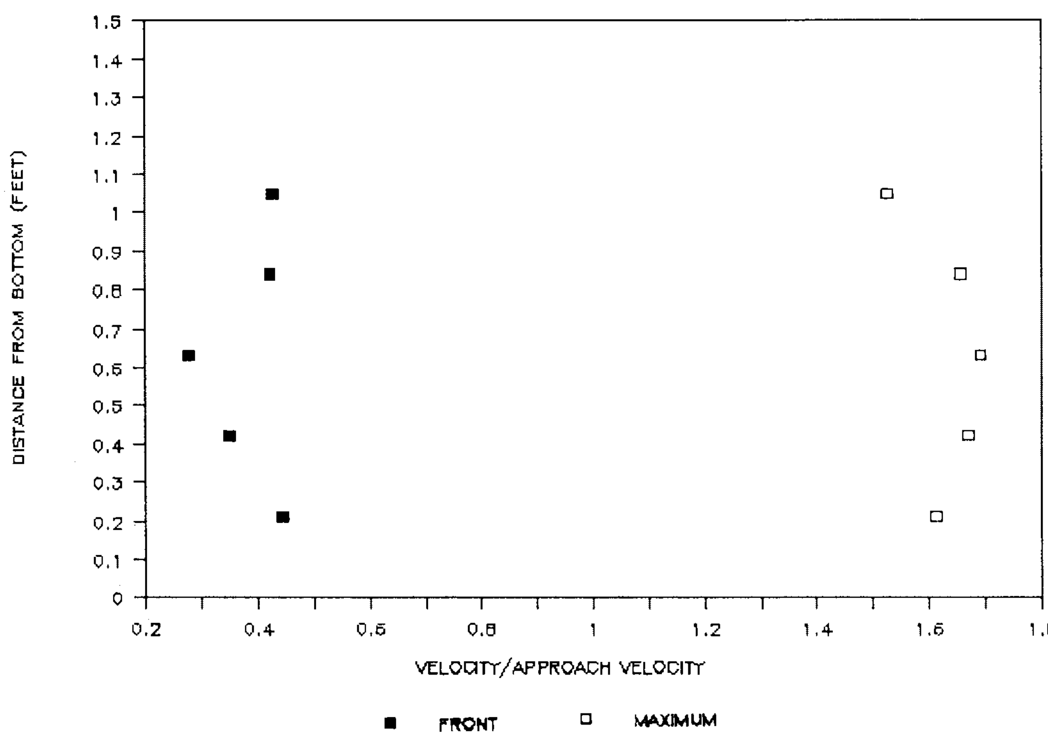


Figure 90. Velocity at equivalent rectangular pier (run 21b).

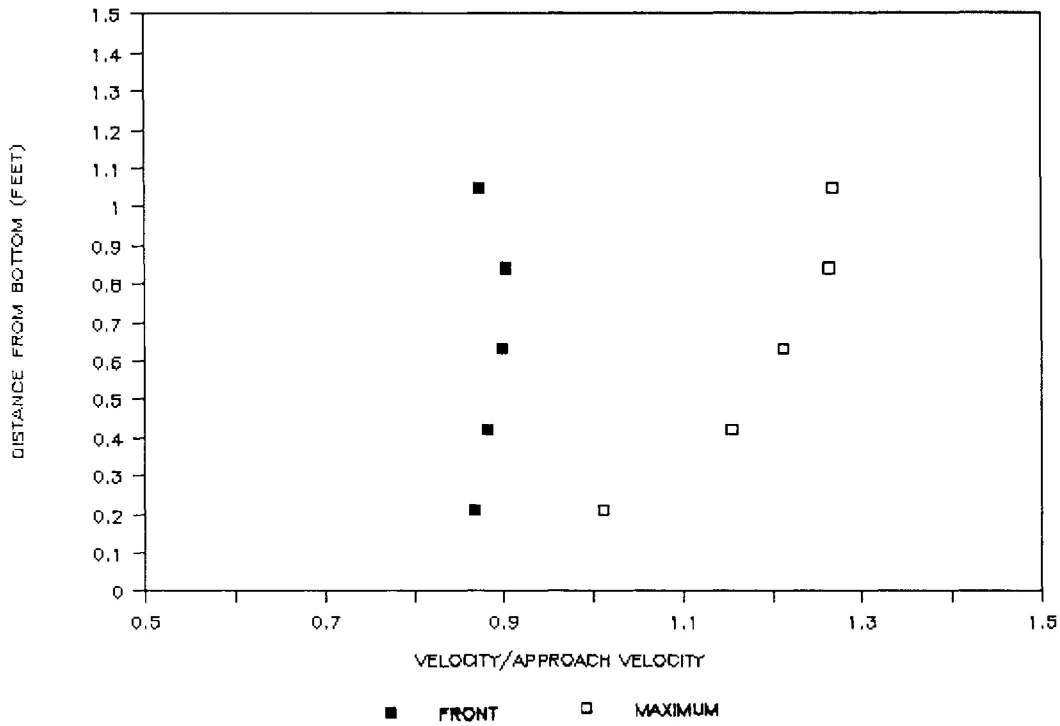


Figure 91. Velocity at concrete pier (run 22a).

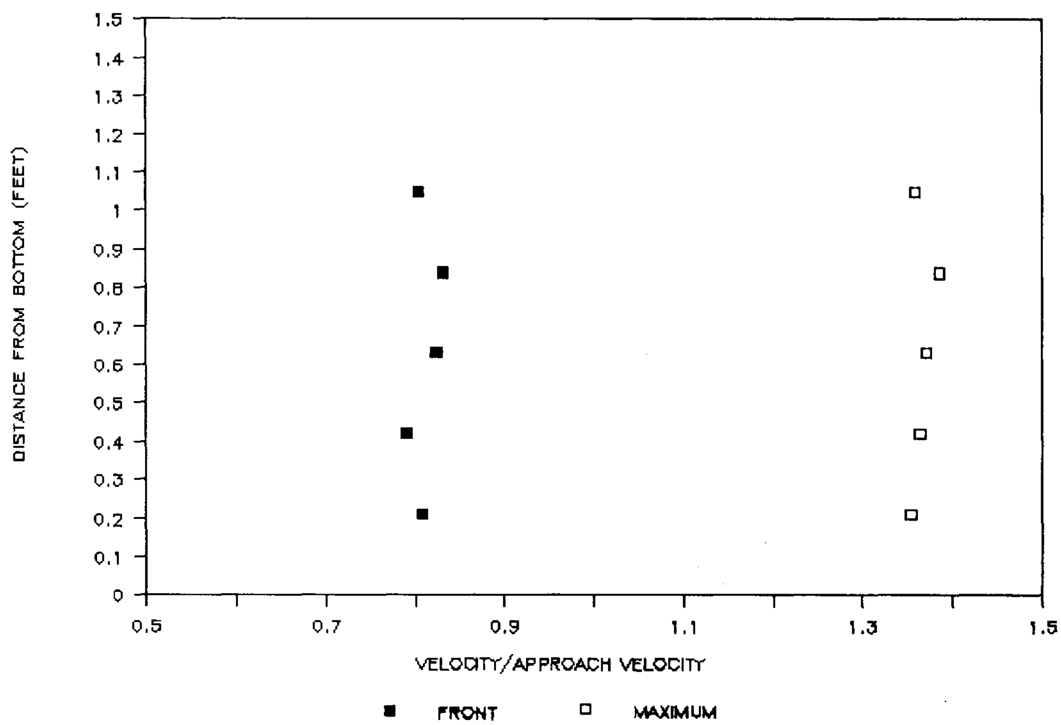


Figure 92. Velocity at equivalent rectangular pier (run 22b).

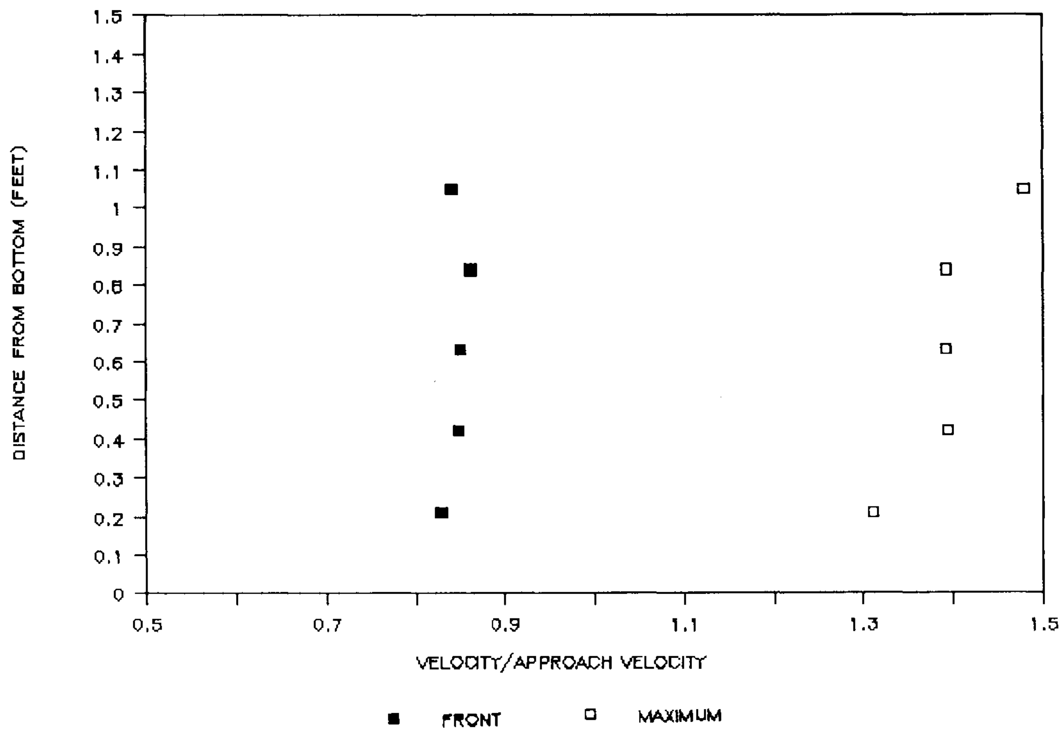


Figure 93. Velocity at modified concrete pier (run 23a).

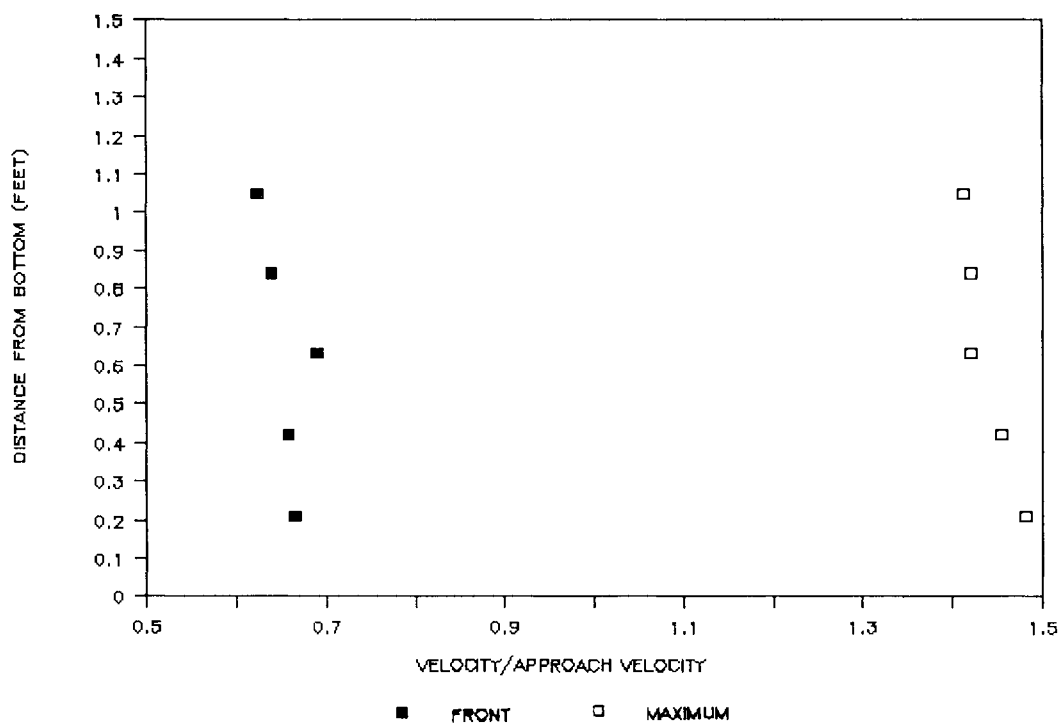


Figure 94. Velocity at equivalent rectangular pier (run 23b).

References

(1) National Oceanic and Atmospheric Administration, Tide Tables 1986; High and Low Water Predictions; East Coast of North and South America Including Greenland. U.S. Department of Commerce, 1986.

(2) National Oceanic and Atmospheric Administration, Tide Current Tables 1986; Atlantic Coast of North America. U.S. Department of Commerce, 1986.

(3) Ishbash, S.V., "Construction of Dams by Depositing Rock in Running Water," Transactions, Second Congress on Large Dams, Vol. 5, Washington, D.C., pp. 123-136, 1936.

(4) American Society of Civil Engineers, "Sedimentation Engineering." Manuals and Reports on Engineering Practice - Number 54, New York, New York, 1975.

(5) Breusers, Nicollet, and Shen, "Local Scour Around Bridge Piers," Journal of Hydraulic Research, Number 3, p. 248, 1977.

(6) Brown, Scott A., and Clyde, Eric S., "Use of Riprap for Bank Protection," Unpublished Report for FHWA Revised HEC-11, 1989.

(7) Maynord, Steven T., "Stable Riprap Size for Open Channel Flows," Ph.D. Dissertation for Colorado State University, 1987.

(8) Neill, R., Guide To Bridge Hydraulics, by Project Committee on Bridge Hydraulics of RTAC, University of Toronto Press, 1975.

(9) U.S. Army Corps of Engineers, Shore Protection Manual, Volume II, pp. 7-249 through 7-253, 1984.

(10) Christiansen, B.A., Appendix C of Acosta Bridge Replacement Preliminary Design Report, Section B, Geotechnical Report for the Steel Alternative, p. 5 of the Appendix, September 17, 1987.