St. Anthony Falls Hydraulic Laboratory
University of Minnesota

Project Report No. 166

HYDRAULIC STUDY FOR STORM SEWER OUTLET

by

Edward Silberman

Prepared for

MINNESOTA DEPARTMENT OF TRANSPORTATION

March 1977
Minneapolis, Minnesota

HYDRAULIC STUDY FOR STORM SEWER OUTLET

Final Report, Minn. D.O.T.

Contract No. 58417

A. SUMMARY AND RECOMMENDATION

A storm sewer is planned to discharge into the Mississippi River from its west bank just below St. Anthony Falls. The sewer pipe is 12 ft in diameter and its centerline will make an angle of about 60° with the river bank at the point of discharge. At that point the river is used by barges approaching a lock at the Falls. The water level is closely controlled at 750 ft, MSL.

It is required that sewer discharges should not produce local mean cross-flow velocities in the river in excess of 4 fps at a sewer design discharge of 1700 cfs nor in excess of 2 fps at the more likely discharge of 500 cfs. The river bed is sandstone at elevation 739.3 ft MSL and the sewer invert has been designed to intersect the river bank at that elevation. Because of the rock, it is desired to diffuse or spread the flow horizontally at the sewer outlet to achieve the required velocities rather than to dissipate energy in a conventional stilling basin. The sewer pipe is laid on a straight tangent of at least 600 ft length at a uniform slope of 0.001 before reaching the river.

This study first involved a literature search and this was followed by a model study. Figure A-1 shows the final model configuration with prototype dimensions recommended for the final design.

The model was operated by the Froude model law at a length scale of 1:14.4. At the design discharge of 1700 cfs, a hydraulic jump formed just upstream of the blocks shown in Fig. A-1. This diffused the flow very well so that at the traverse line marked B-B in the figure (30 ft along the centerline from the shore) the local mean velocity did not exceed the 4 fps requirement. At a traverse along A-A (15 ft into the river), local mean velocities up to about 5 fps occurred. (Velocities were all measured with the axis of the meter parallel to the centerline of the outlet structure, but the meter is relatively insensitive to off-axis directions up to 10 or 15 deg.) If is is desired to meet the 4 fps test along A-A, the entire structure should be moved 15 ft shoreward, retaining the angles between guide walls at 7 deg and thus increasing

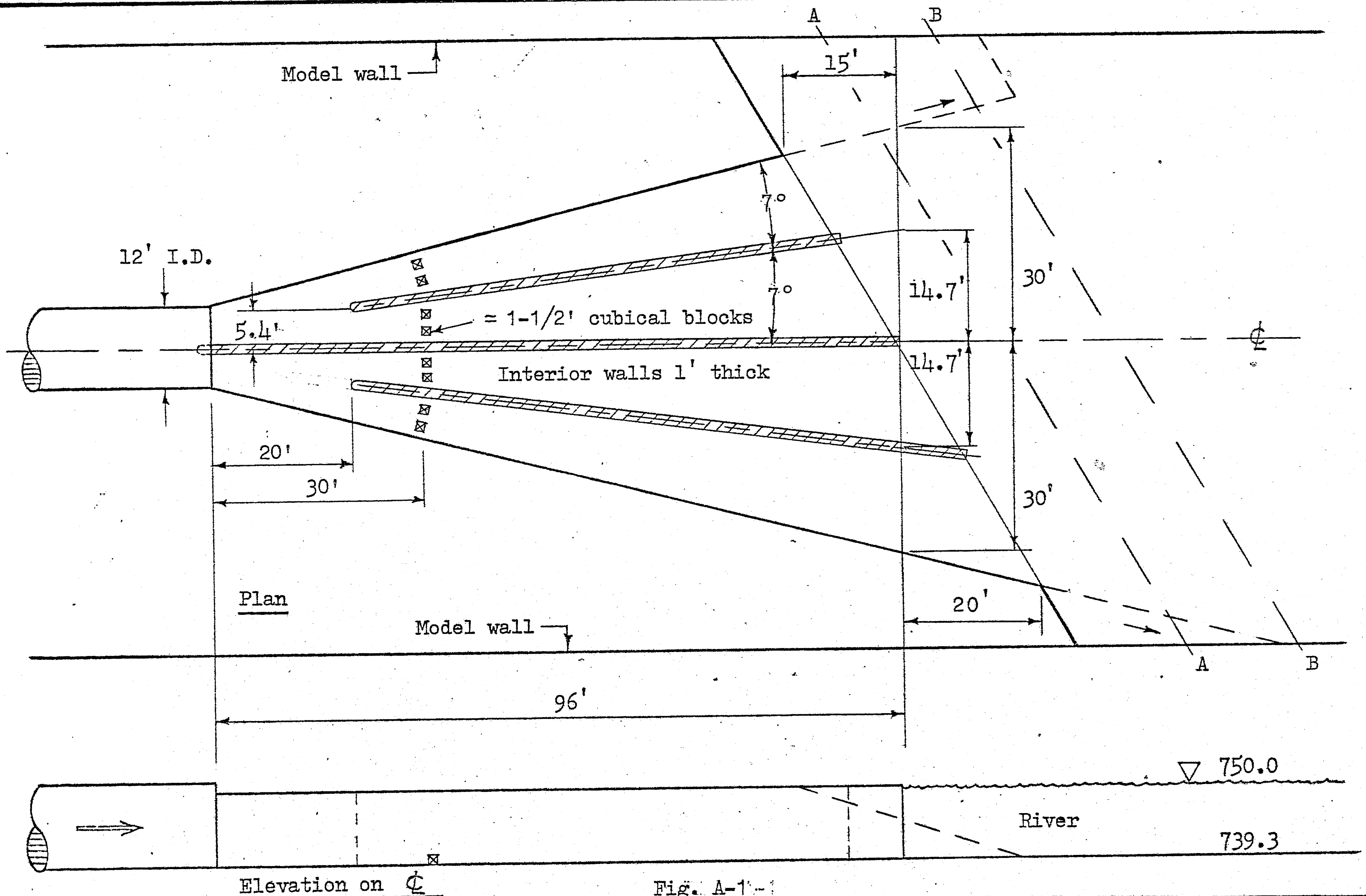


Fig. A-1
STORM SEWER OUTLET STRUCTURE
Model Study
Minnesota D.O.T.

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN DH	CHECKED	APPROVED ES
SCALE 1"=20'	DATE 2/16/77	NO. 263A-2314-2

the area at outlet. A hydraulic jump occurred at discharges down to about 1000 cfs and the outlet velocity at these discharges would be correspondingly lower than at 1700 cfs.

At the discharge of 500 cfs, no jump occurred and most of the flow passed through the two central channels in Fig. A-1. However, local mean velocities were well below the 2 fps maximum along the traverse B-B. The flow could be spread better at the lower discharges by using a transition structure at the pipe outlet. A typical structure is illustrated in the U.S. Bureau of Reclamation Design of Small Dams, 2nd Ed, 1973, p. 476. No transition structure was used in the model.

B. SEQUENCE OF WORK

Work under the contract involved several phases. The first phase was a review of the literature to determine whether the outlet could be designed from available knowledge. This began on December 3, 1976 and was completed with a report submitted under date of December 20, 1976. That report, which included design suggestions is reproduced in the next part of this report (Part C).

On the basis of the literature study, a model study was authorized of one of the proposed schemes (Fig. C-4 of Part C). Authorization was received on December 28, 1976 and the model was constructed and placed in operation on January 17, 1977. After some experimental revision of the model, a formal drawing was submitted on February 16, 1977 showing the desirable overall features of the outlet to accomplish the objectives. Further experiments on details of design were carried out for one more week and all model experiments were completed on February 23, 1977. Figure A-1 is the final recommendation of the model study and is to replace the drawing submitted on February 16, 1977, from which it differs only in details near the pipe exit.

During the course of the model studies several groups of D.O.T. personnel, as well as accompanying persons from other agencies, observed the model in operation as background for eventual design for the outlet.

The model study is described in detail and the velocity profile measurements are displayed in Part D of this report.

C. REVIEW OF LITERATURE

Literature Search

More than 50 sources were investigated seeking design information and about 30 of these were sufficiently useful to make some contribution to this design problem. Only a few of these were used directly in the design, however, and they will be cited subsequently where specific designs are suggested.

The literature abounds with papers dealing with head loss for various kinds of expansions. But there is very little on the corresponding velocity distribution following the expansion. Most of the data available are for uniform entrance velocity profiles to the expansion, and the authors of the papers warn that the kind of entrance profile is very important in determining downstream profiles and losses. Yet, very few papers deal with fully developed pipe entrance profiles of the kind prevailing in this problem. Most of the data are for axially symmetric or plane-wall diffusers in pipes. There is a little data for plane-walled channels. Practically no data are available for the transition from round pipe to open channel without a hydraulic jump. The most important parameters in expansion design are the area ratio between the expanded flow and the entrance flow and the length over which the expansion occurs. Most of the data are for area expansions of four or less whereas this problem involves an area expansion of at least five, as will be seen.

Despite these deficiencies, much useful design data was found. An attempt has been made to trust data only when it could be corroborated by at least two sources or, at least, by a favorable discussion of the paper reporting the data. On this basis, it was possible to suggest several design ideas. These are presented in the following section.

Design Suggestions

The critical design parameters are the maximum discharge of 1700 cfs from a 12 ft diameter pipe to reach the river with no more than 4 fps maximum velocity. At lesser discharges, the criteria are less severe. To obtain 4 fps maximum velocity at the river, the average velocity at the river must be less than 4 fps. Some possibilities are displayed in Table 1.

Table 1 - Exit Areas at the River for 1700 cfs from 12 ft Diameter Pipe.

$\bar{V}_p$ = Avg. pipe velocity = 15 fps.

A_p = Pipe area = 113.1 ft^2

$\bar{V}$ = Avg. velocity at exit to river

A_R = cross sectional area at exit to river

V_m = Maximum velocity at exit to river = 4 fps

$V_m/\bar{V}$	$\bar{V}$ fps	$\frac{A_R}{A_p} = \frac{\bar{V}_p}{\bar{V}}$	Nominal Exit cross section	Open Channel Hydraulic Dia., ft	$\bar{V}^2/2g$, ft
1.2	3.33	4.5	10 ft x 50 ft	29	0.17
1.33	3.0	5	10 ft x 60 ft	30	0.14
1.6	2.5	6	10 ft x 70 ft	31	0.10
2.0	2.0	7.5	10 ft x 85 ft	32-1/3	0.06

The first line in Table 1 refers to an average value for fully developed, uniform flow in a pipe or channel at high Reynolds numbers. It is usually believed that 50 hydraulic diameters of the downstream pipe or channel are required to come close to reestablishing this uniform flow pattern following an abrupt change such as a sharp inlet, an expansion, or a bend. If this outlet were designed as an abrupt expansion to an open channel of 10 ft by 50 ft cross section, about 1450 feet would be required for the transition on this basis. If the expansion was to a pressurized channel, 850 ft would be required on a similar basis.

The second and following lines in Table 1 are more likely to be achievable in a reasonable distance following flow expansion. For design purposes, $V_m/\bar{V}$ = 1.33 has been chosen because it is believed from the literature search that this can be obtained with the designs contemplated. Also, as already noted, data in the literature are very scarce for area ratios larger than four so that even for the velocity ratio chosen, the available data has to be extrapolated a little. Thus, the channel entering the river is to have a nominal depth of 10 ft (corresponding to the river depth) and a width of 60 ft (57 ft would really be adequate).

Four possible schemes have been sketched in Figs. C-1 through C-4 to accomplish the transition. Figure C-1 shows another abrupt expansion, but with the less severe requirement for velocity profile just outlined. Various authors give widely varying requirements for the distance L . Chaturvedi [1] would require 120 ft; Ball [2] would require 150 ft; and Idel'chik [3] 300 ft (10 hydraulic diameters). If this scheme were adopted without test, nothing less than 300 ft should be used for L . The head loss between the pipe and the river will be about 2.2 ft. Ball [2] has measured the fluctuating head near the pipe outlet and it appears this will be about $\pm$ 2.1 ft. If this design is used, there should probably be a pressurized roof from the pipe outlet to at least half way to the river. However, as soon as the discharge decreases below about 1000 cfs (depending on the roughness coefficient--n = 0.012 has been used herein), open channel flow conditions would prevail. Even with open channel flow, the depth will be subcritical at exit from the pipe, although it could be close to critical there. Further downstream, as the velocity profile develops, it will be more safely subcritical. It would perhaps be useful to provide air ventilation pipes to the expanded section just downstream of the pipe entrance. A refinement that is possible with this design is to place cusped cavities at the head end of the enlargement as shown by dotted lines in Fig. C-1 to encourage establishment of two stationary vortexes. This is supposed to stabilize the flow expansion, but no good data are available to prove it [4]. It has also been suggested that short, pyramidal dividers be placed on the certerline at the floor and ceiling just downstream of the pipe [5]; again, pertinent data are not to be found.

Figure C-2 is a sketch of a gradual expansion. The design is largely based on Miller [6]. Miller recommends a minimum of 4 hydraulic diameters of downstream pipe or channel beyond the diffuser to rectify the velocity profile. This would be a minimum of 120 ft for L ; probably 150 ft would be better. Thus, the total length of diffuser and straight section would be longer than required for an abrupt expansion. The compensating feature is that the head loss would be less--not over 1 ft, but head loss is not a factor in this design anyway. The head loss would be so small that the pipe would not flow full at the outlet. Hence, there would be no point in building a pressurized roof.

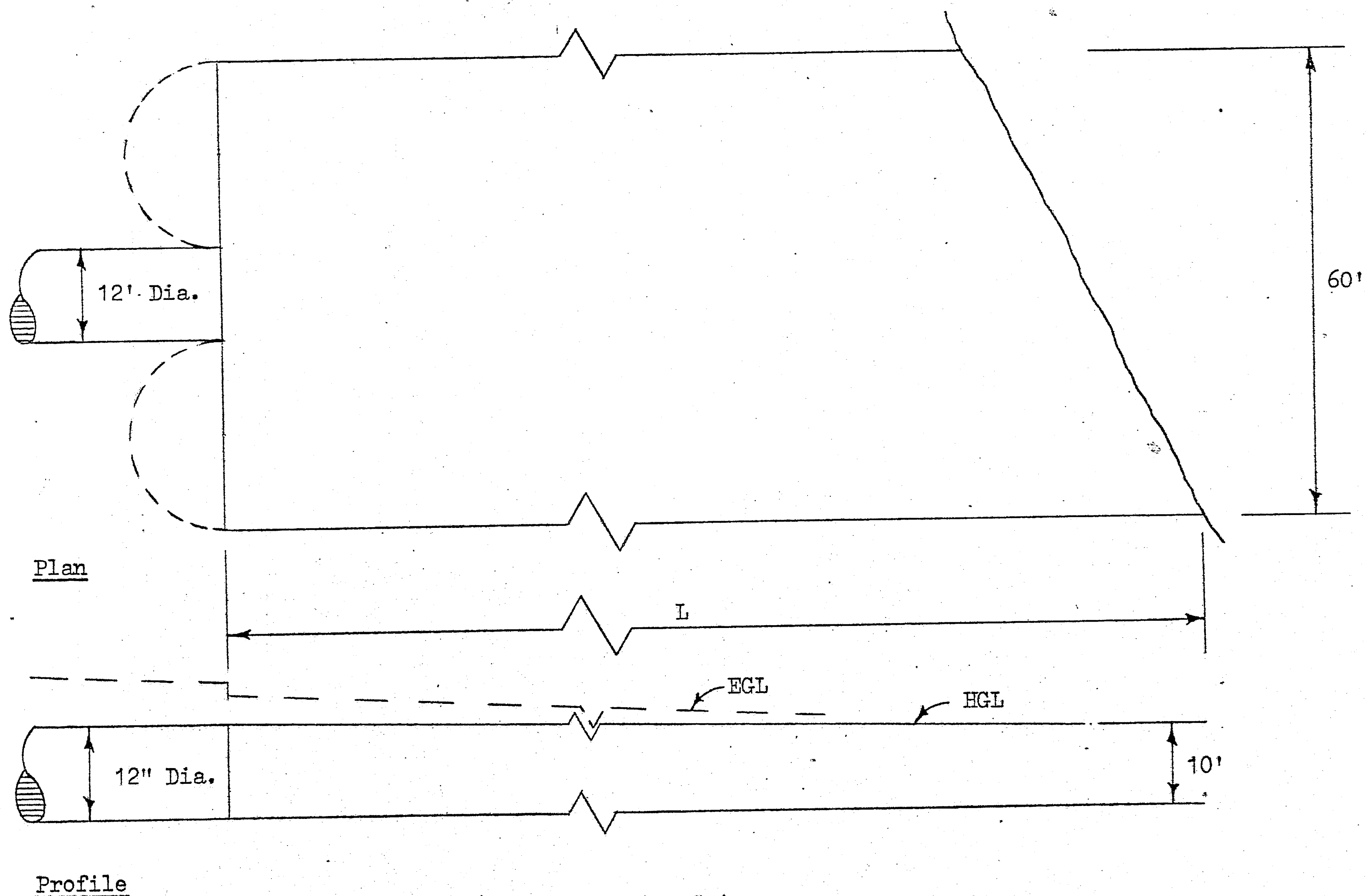


Fig. C-1

ABRUPT ENLARGEMENT

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN	CHECKED	APPROVED ES
SCALE 1"=20'	DATE	NO.

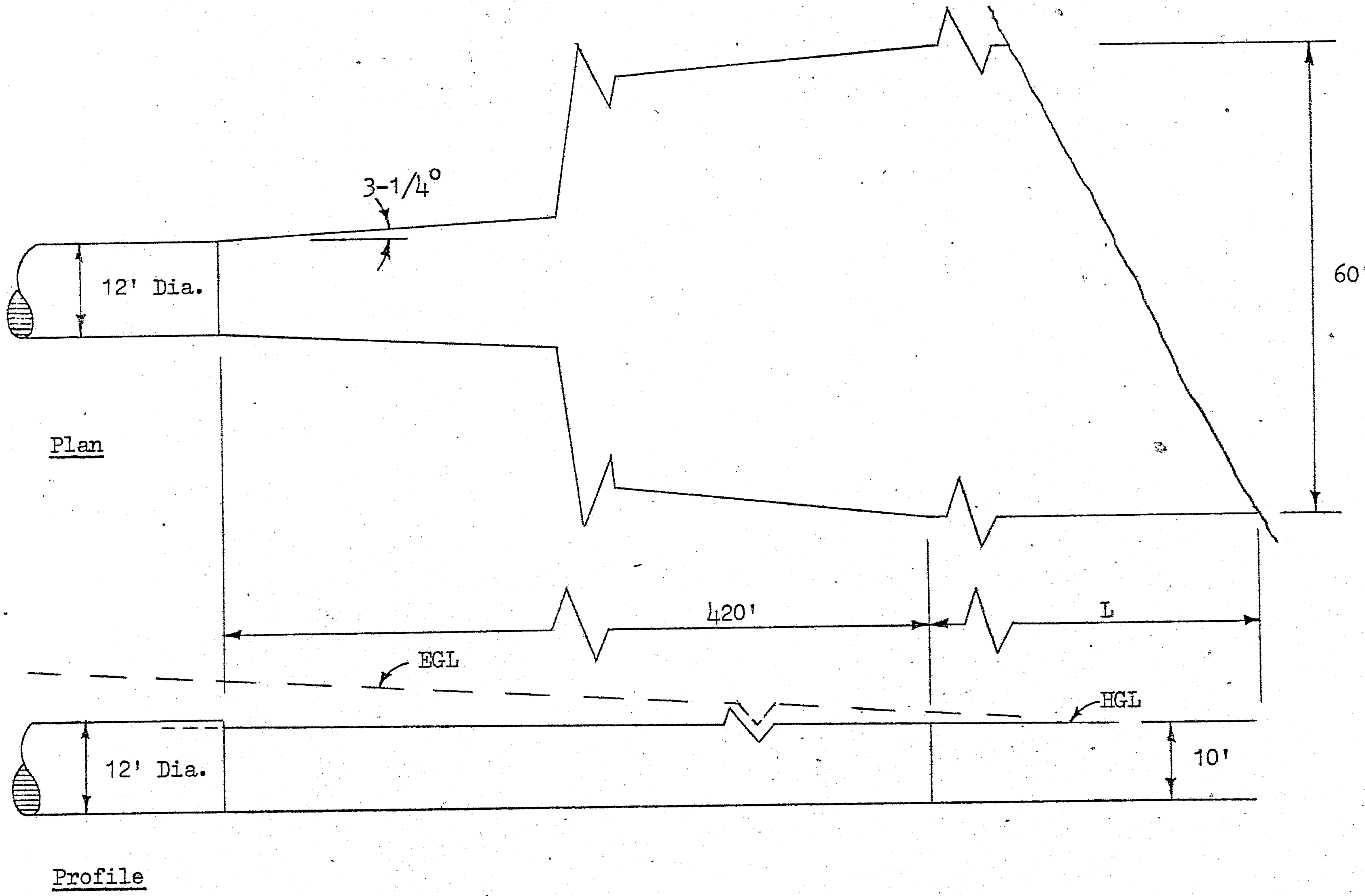


Fig. C-2

GRADUAL ENLARGEMENT

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN	CHECKED	APPROVED ES
SCALE 1"=20'	DATE	NO.

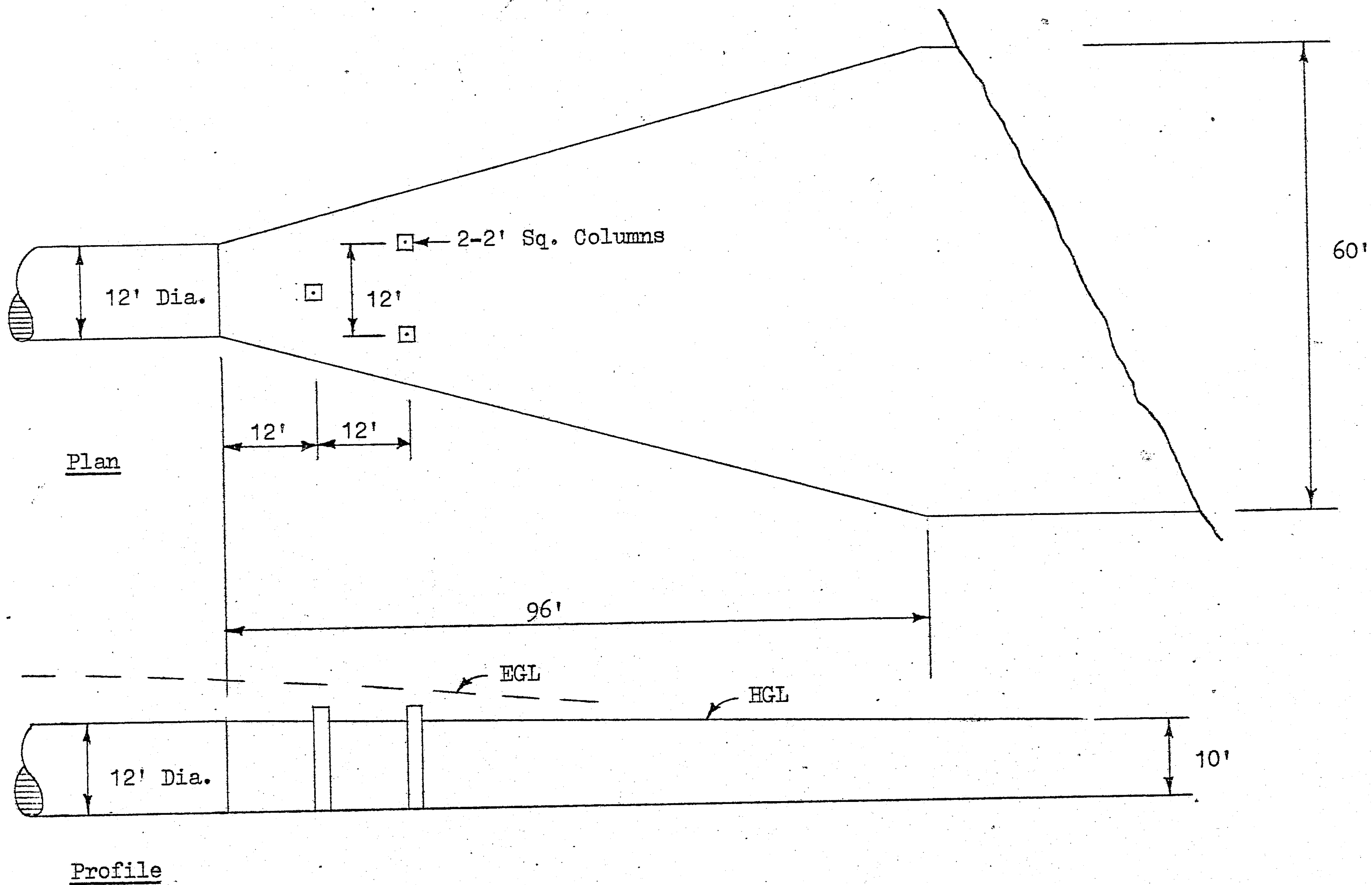


Fig. C-3

HEAD LOSS BAFFLE
ENLARGEMENT

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED ES
SCALE 1"=20'	DATE	NO.

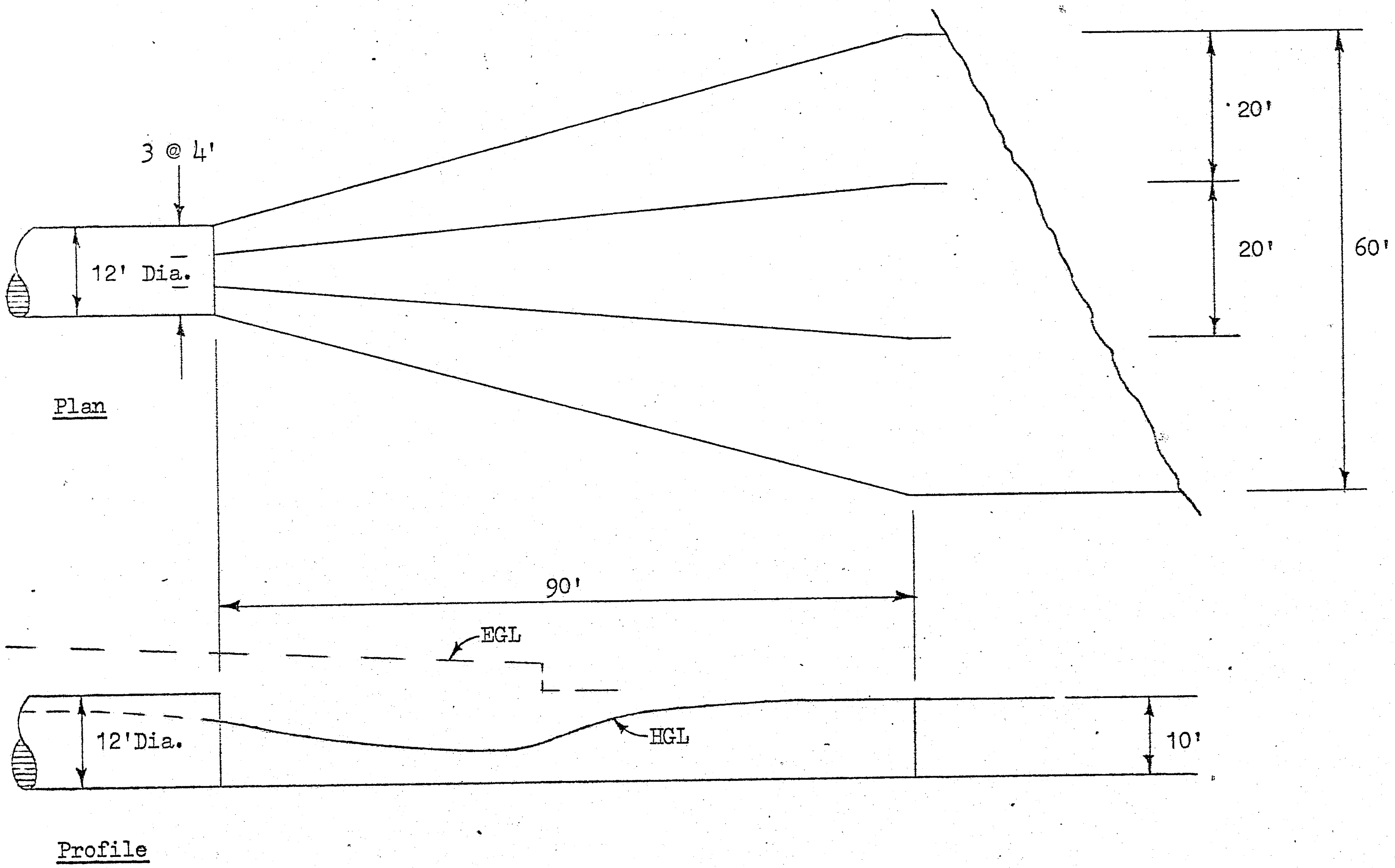


Fig. C-4

SPLIT ENLARGEMENT

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN	CHECKED	APPROVED ES
SCALE 1"=20'	DATE	NO.

An attractive scheme for a shorter expansion using head loss baffles is presented in Fig. C-3. The design is from a small-scale study by Smith and Yu [7] for an open channel transition. The head loss will be about 2 ft so that the pipe will just about be full at exit and a pressurized roof might be used at maximum discharge. However, at lesser discharges, the pipe would not be full at exit and an open channel situation will prevail. The downstream velocity distribution was only indicated schematically in the paper referred to so that this design should be checked by model study if it is to be adopted.

A comparably short diffuser can be obtained using splitters as shown in Fig. C-4. The design is from Idel'chik [3] for rectangular ducts. Head loss is only about 0.5 ft from the pipe to the river so that open channel flow would prevail. A.T. Ippen in his chapter in Rouse [8] recommends a more conservative design with smaller angles and longer transition of about 130 ft. This was based on a model study for a transition from a rectangular to a trapezoidal open channel. Again, this design should not be adopted without a model study to check velocity distribution.

Another possibility for a short expansion is to use short guide vanes placed in the pipe exit. Data may be found in Idel'chik [3]. However, these vanes have sophisticated shapes which would be more difficult to form than the straight walls suggested in Figs. C-1 to C-4 and would be more subject to damage from debris. Such an expansion can be designed, however, if it is desired to go that way.

References for Section C

[1] Chaturvedi, M.C., "Flow Characteristics of Axisymmetric Expansions", Journ. Hydraulics Div., ASCE 89:HY3: 61-92, (May 1963).

[2] Ball, J.W., "Sudden Enlargements in Pipelines", Journ. Power Div., ASCE 88:PO4: 15-27 (Dec. 1962).

[3] Idel'chik, I.E., Handbook of Hydraulics Resistance, 1960, Translated from Russian, AEC-tr-6630, 1966, 515 pp.

[4] Ringleb, F.O., "Two-Dimensional Flow with Standing Vortexes in Ducts and Diffusers", Journ. Basic Engr., ASME 82D: 921-928, (Dec. 1960).

[5] Dake, J.M.K. and Francis, J.R.D., "Outlet Works of Spillway Tunnels", La Houille Blanche, 1965, No. 6, pp 555-563.

[6] Miller, D.S., Internal Flow; A Guide to Losses in Pipe and Duct Systems, BHRA 1971, 329 pp.

[7] Smith, D.D. and Yu, J.N.G., "Use of Baffles in Open Channel Expansions" Journ. Hydraulics Div., ASCE 92:HY2: 1-17, (March 1966).

[8] Rouse, H., Ed. Engineering Hydraulics, Wiley, 1950, pp 523-524.

D. MODEL STUDY

Design of Model

Since the outlet design involves mainly diffusion or spread of the flow, Reynolds number is an important modeling parameter. But once the model Reynolds number is large enough, no purpose is served by making it larger. To operate the model with the same Reynolds numer as in the prototype sewer would require an impracticably large model since adequate model discharges can only be obtained with water as a fluid. Furthermore, the sewer flow in the outlet is mainly free surface with the possibility of hydraulic jumps at the larger discharges. Free surface flow is a gravity or Froude law phenomenon. Hence, it was decided to operate the model in accordance with the Froude law, maintaining a sufficiently high Reynolds number to obtain faithful reproduction of turbulence spreading behavior. For this purpose, a length scale of 1:14.4 was selected.

With the above length scale, the 12 ft sewer pipe was modeled with a 10 in. galvanized iron pipe. The original model scheme is shown in Fig. D-1. A photographic view of the downstream end of the model is contained in Fig. D-2. (Some changes had been made from the original model shown in Fig. D-1 by the time this picture was taken). The 17 ft length of 10 in. pipe is equivalent to 20 pipe diameters at full flow, which should be sufficient to produce fully developed flow in the pipe corresponding to that in the sewer.

By the Froude law, the corresponding velocity and time scales are $1:\sqrt{14.4}$ or 1:3.8 and the discharge scale is $1:(14.4)^{5/2}$ or 1:787. The mean velocity in the sewer for the design discharge of 1700 cfs is 15 fps. The corresponding model velocity is 3.95 fps. The model pipe Reynolds number is thus about 300,000. This is adequate to achieve satisfactory reproduction of all turbulence effects in the model. At the lesser discharge of 500 cfs, the model pipe Reynolds number will be a little less than 100,000, but this should still be adequate.

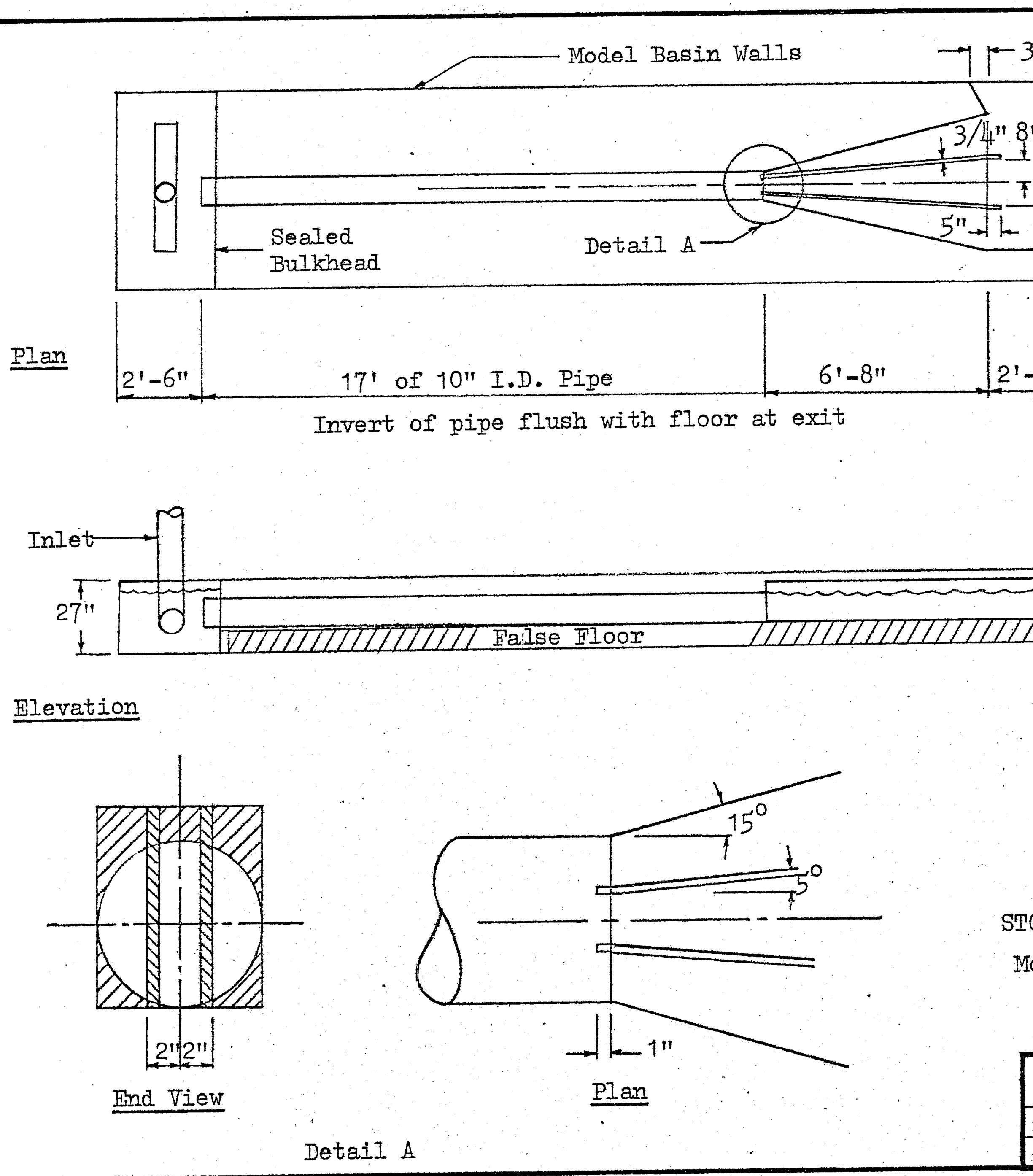


Fig. D-1

Model "A"

STORM SEWER OUTLET CONUIT

Model Scale: 1 to 14.4

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED ES
SCALE 1"=5'	DATE 12/22/76	NO.

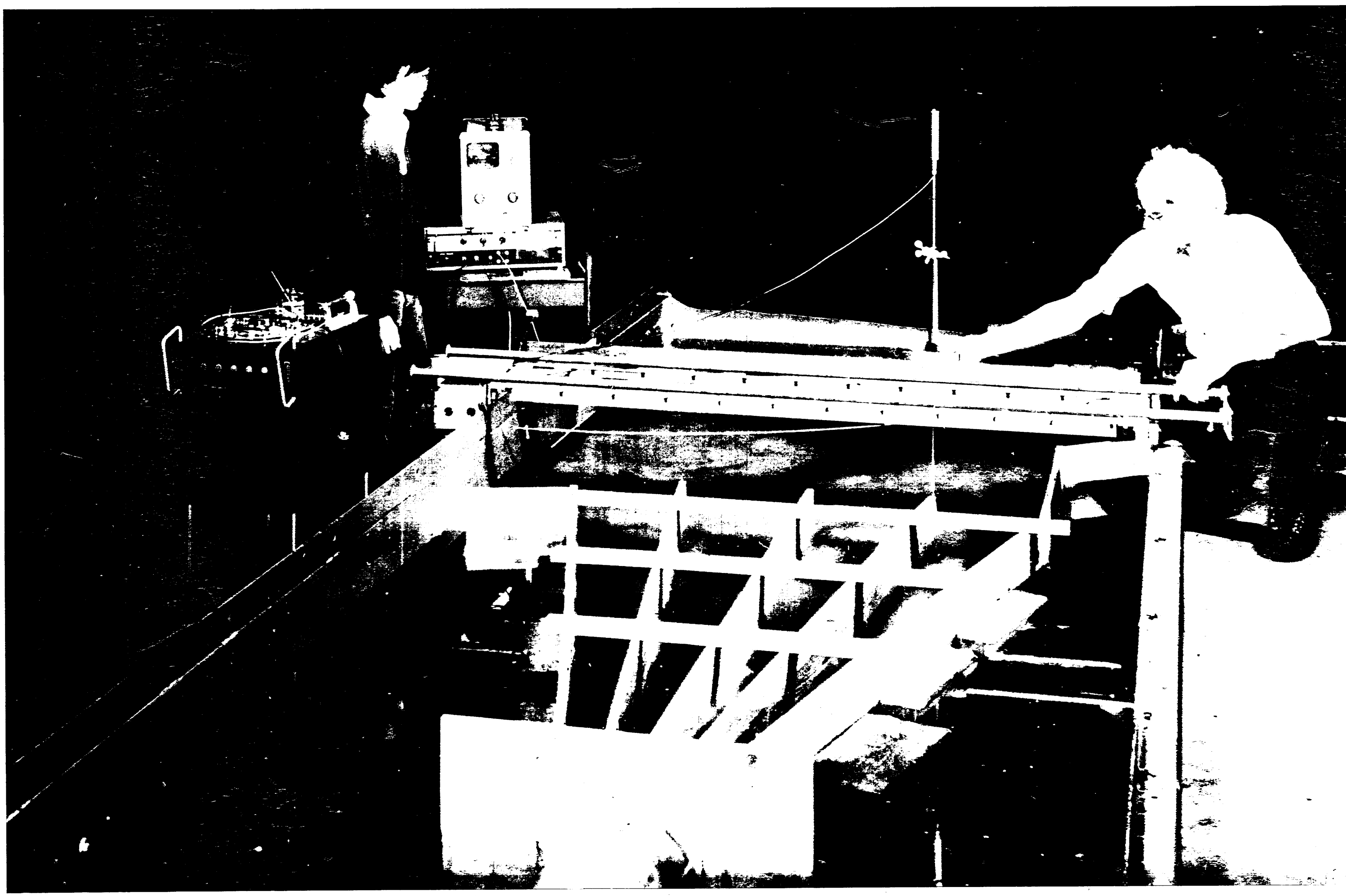


Fig. D-2 - PHOTOGRAPH OF MODEL B-1

Model Operation

Experiments were conducted initially on the model shown in Fig. D-1. This was identified as model "A". It soon became apparent that at large discharges the maximum permissable outlet velocity would be exceeded at 1700 cfs at some points in model "A". Hence, this configuration had to be modified. (Velocity measurements in model "A" are not shown in this report, but they are available in the project records.)

Modification consisted of adding an additional dividing wall, reducing the angle between walls to 7 deg so that the outlet structure looked like the photograph in Fig. D-2. This was called model "B-1". Model "B-1" met the design objective for maximum velocities and the most extensive velocity measurements were made on it. Horizontal velocity profiles taken in the river opposite the outlet at about 1.8 ft below the water surface, at mid-depth, and at about 1.8 ft above the bottom are shown for the critical discharges of 1700 and 500 cfs in Figs. D-3 and D-4, and at about 1.8 ft below the surface (where the highest velocities occur) at discharges of 300, 750, 1000, 1250, 1500, and 2000 cfs in Figs. D-5 to D-10.

The velocities shown in these and subsequent figures are averages taken over a 38 sec. prototype time. They are very repeatable. However, shorter time averages, which will be given later for the final design, show fairly large variations from these averages. The measured velocities are in the direction of the outlet structure center wall and are all in the horizontal plane at each of the various depths.

The dividing walls in model "B-1" terminated at 96 ft from the pipe end as shown in Fig. D-2; this was the distance to where the left wall of the outlet met the water line at the river bank. The structure designers expressed a desire to have all the walls carry to the shore line so as to achieve a better structure for blending with the environment. They also wanted to move the structure riverward so that there would be only 96 ft from the pipe to where the center wall intersected the water line at the river bank. These changes were made in the model so that it took on essentially the configuration shown in Fig. A-1. (The two interior walls shown as beginning at 20 ft from the pipe end in Fig. A-1 were only 10 ft from the pipe end in this model, as they were in models "A" and "B-1", and roughness was provided by a number of nails sticking into the floor instead of using blocks as shown in Fig. A-1). This was model

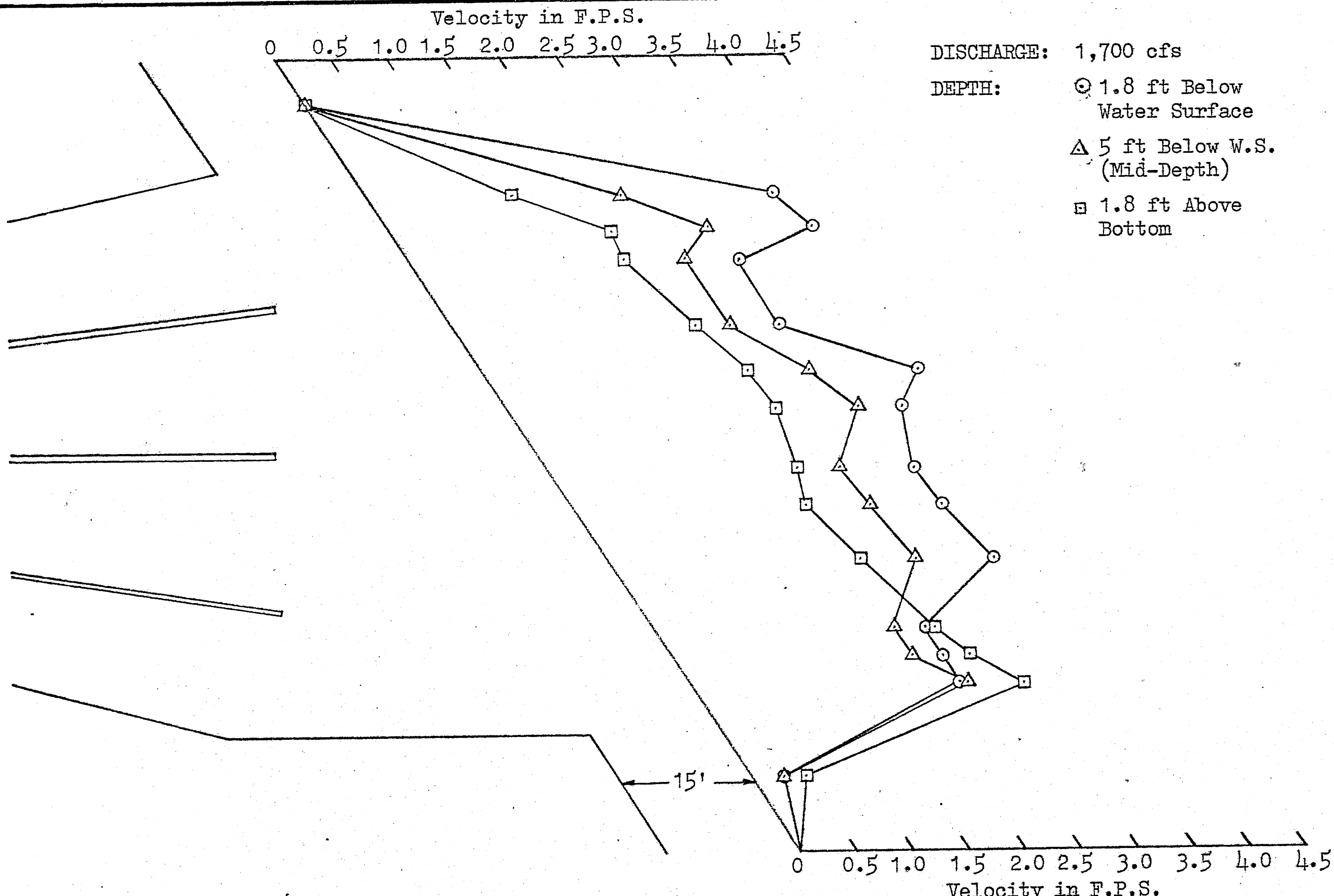


Fig. D-3

VELOCITY PROFILE AT OUTLET

MODEL B-1

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

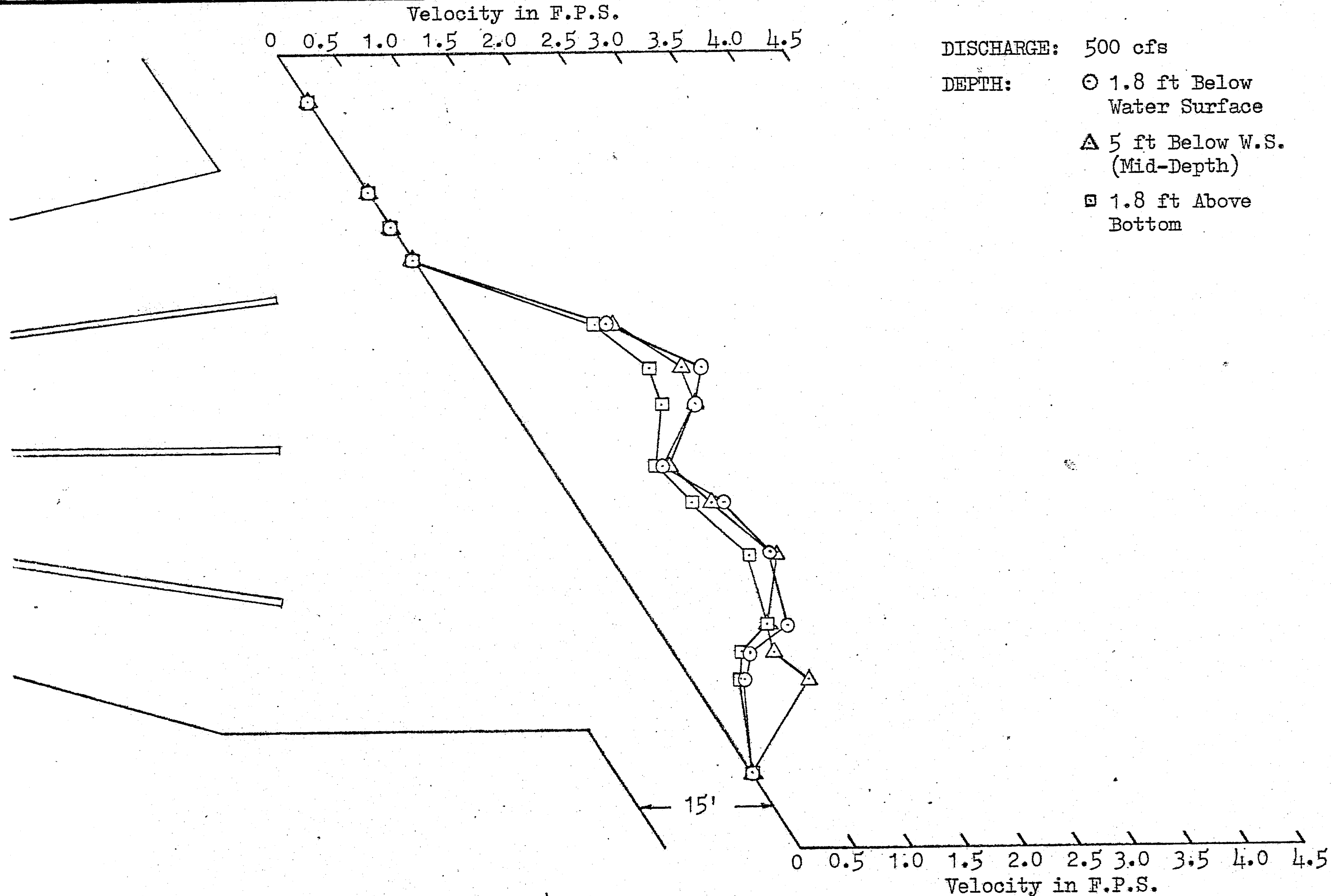


Fig. D-4

VELOCITY PROFILE AT OUTLET

MODEL B-1

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

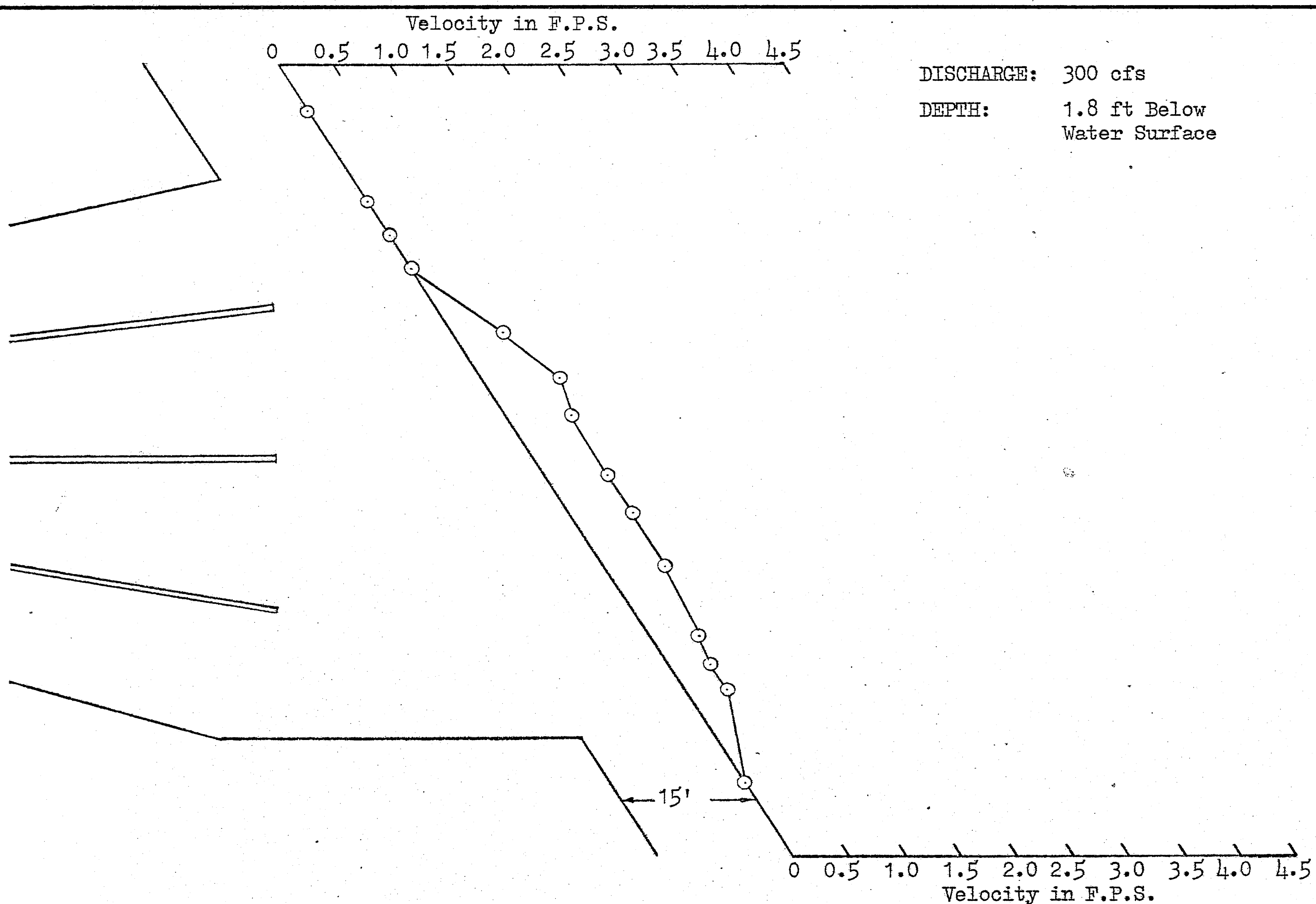


Fig. D-5

VELOCITY PROFILE AT OUTLET

MODEL B-1

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

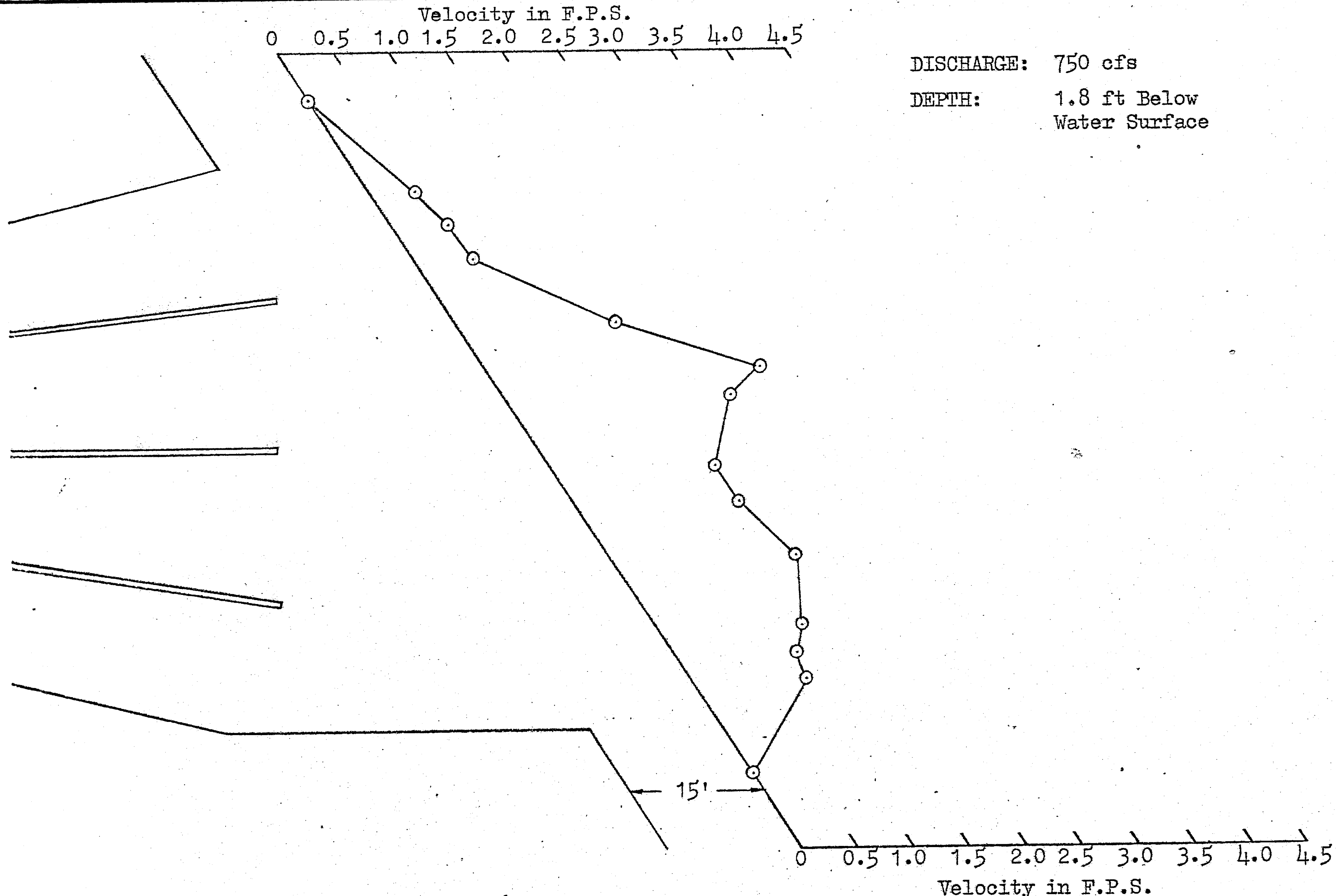


Fig. D-6

VELOCITY PROFILE AT OUTLET

MODEL B-1

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

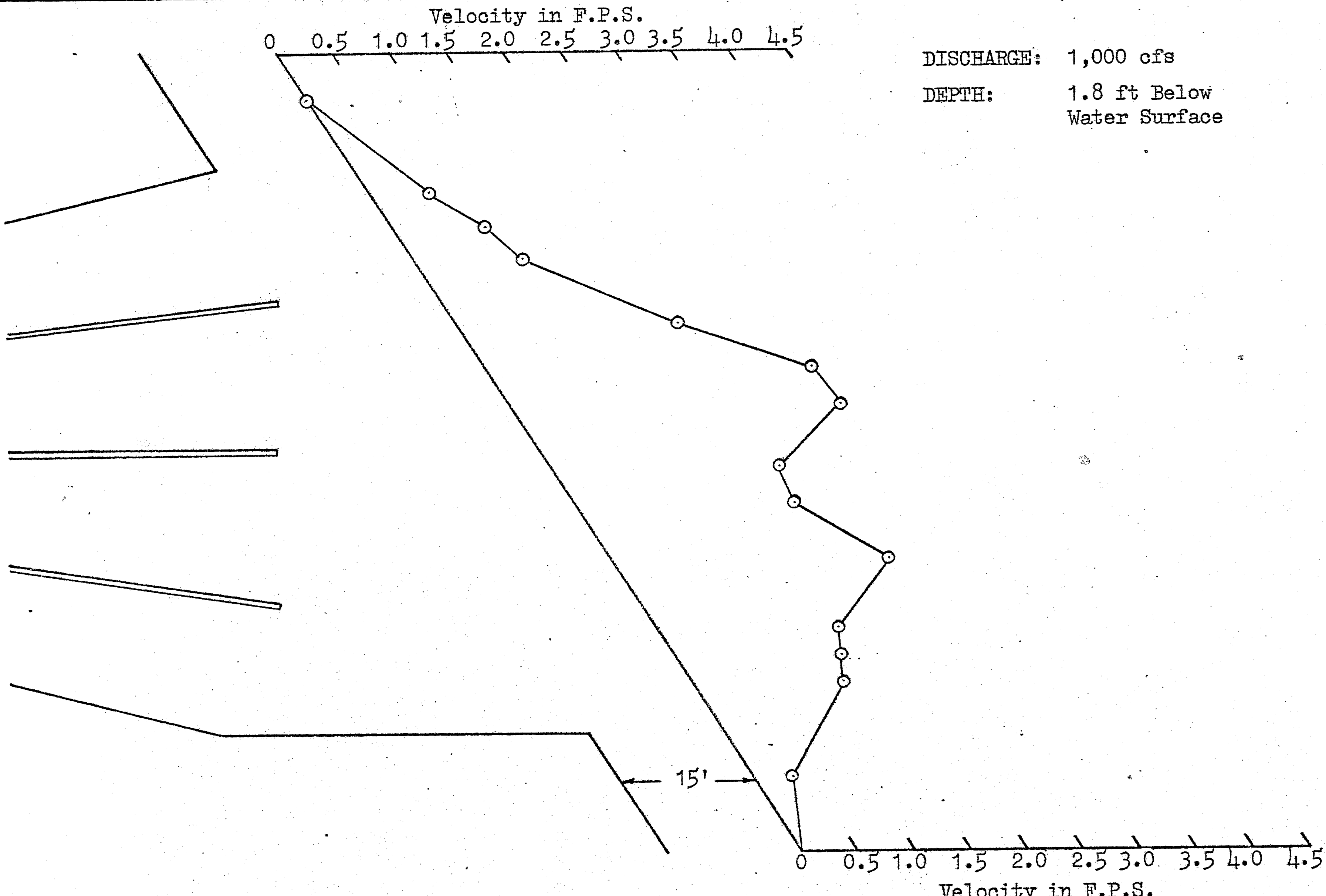


Fig. D-7

VELOCITY PROFILE AT OUTLET

MODEL B-1

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

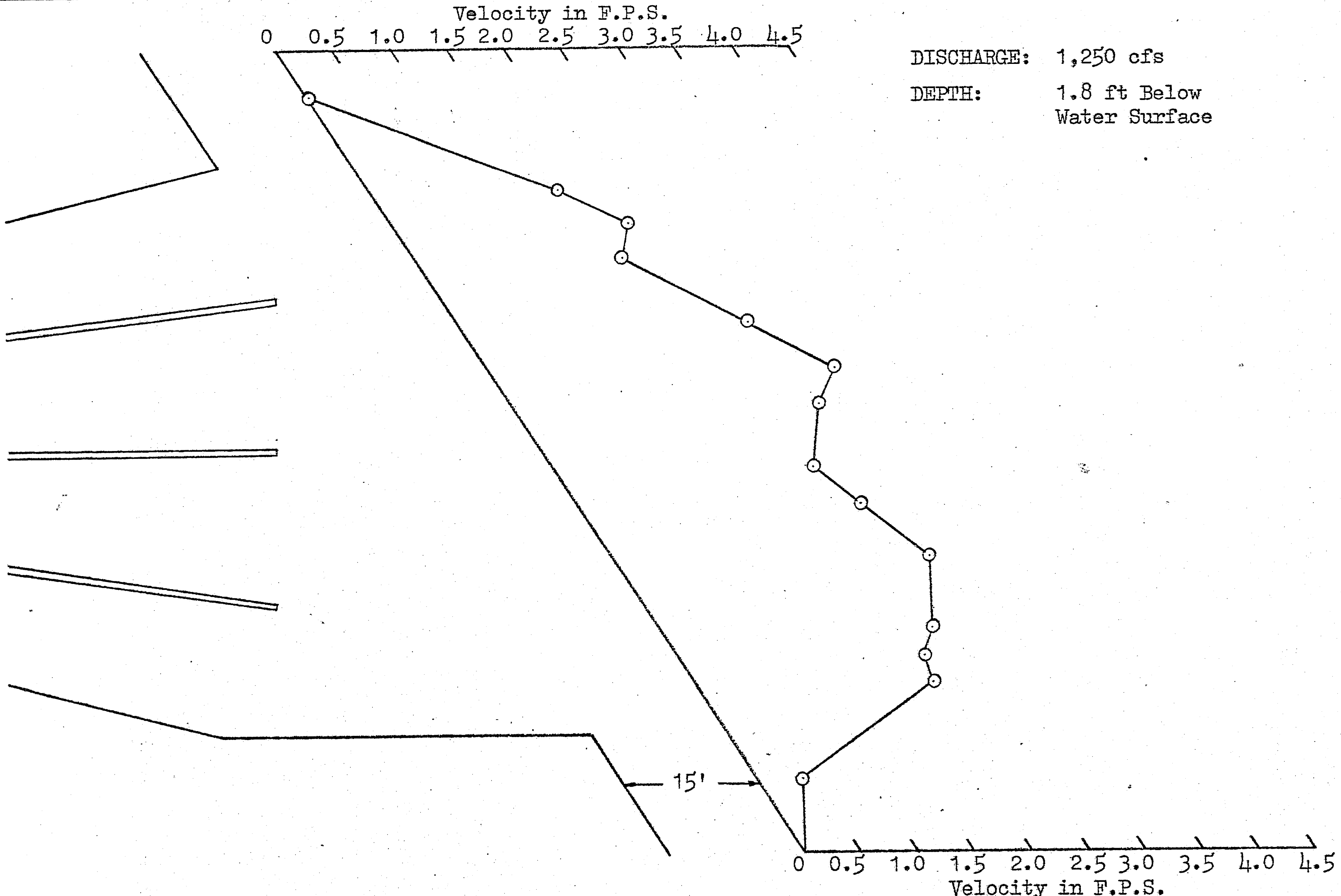


Fig. D-8

VELOCITY PROFILE AT OUTLET

MODEL B-1

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

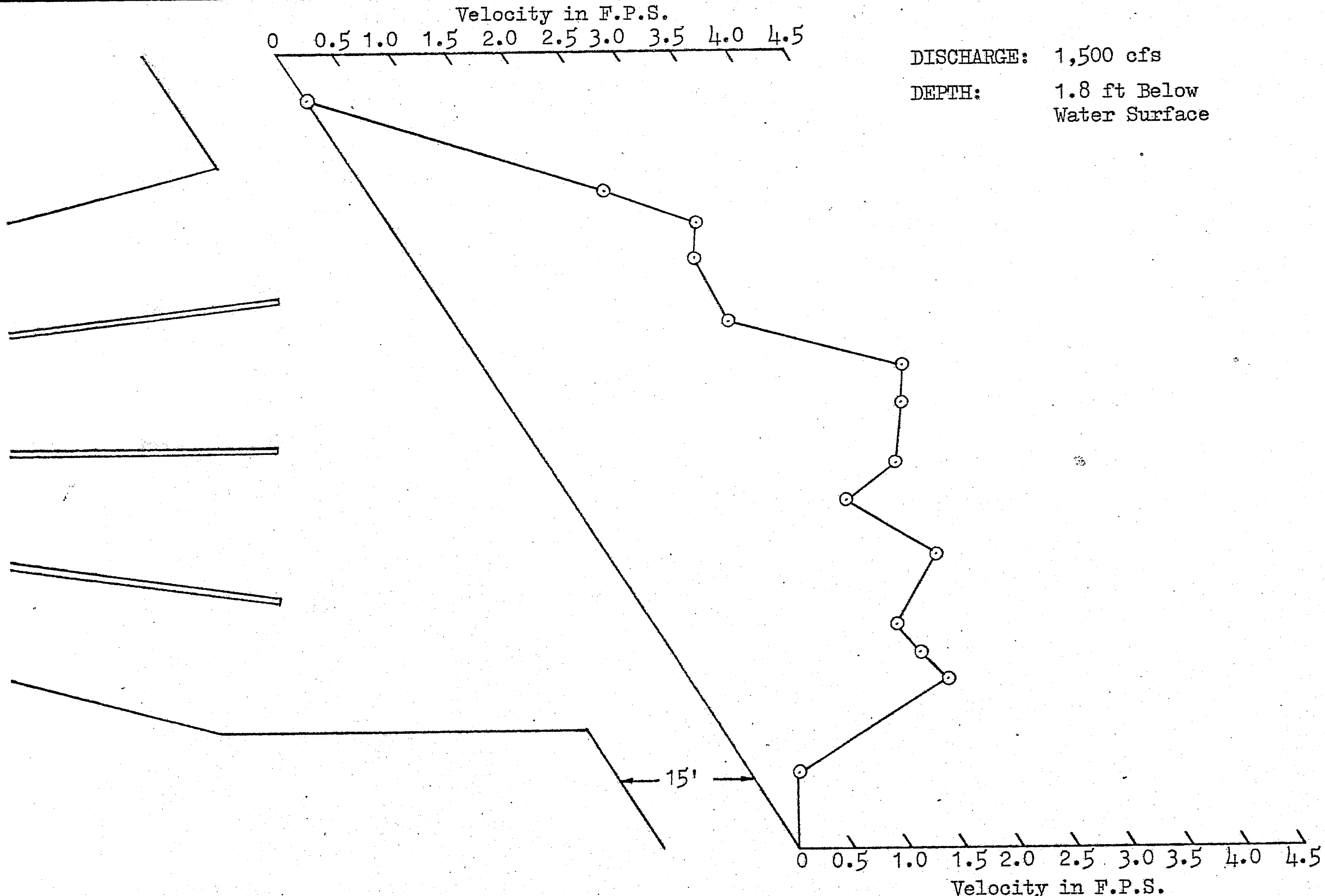


Fig. D-9

VELOCITY PROFILE AT OUTLET

MODEL B-1

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

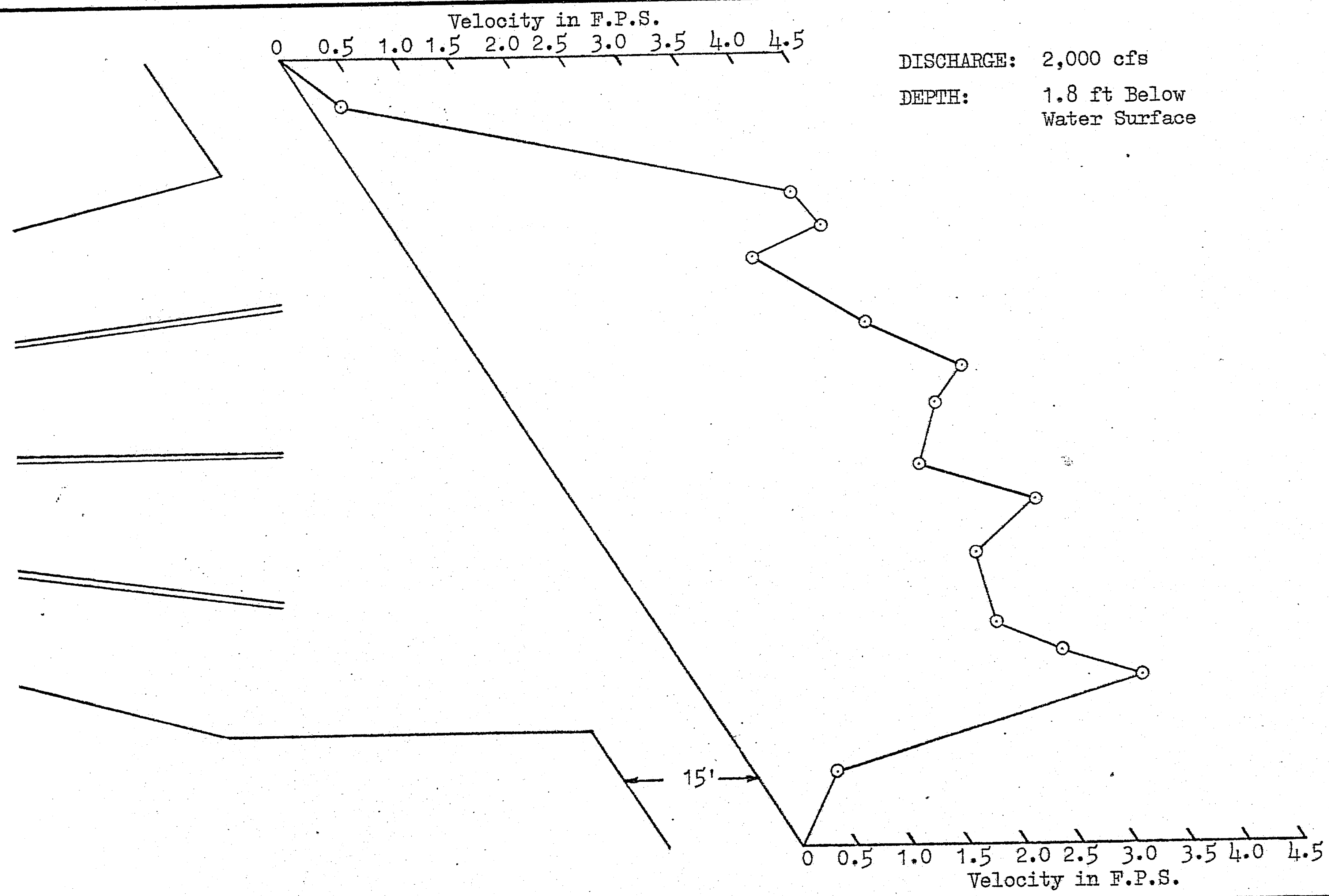


Fig. D-10

VELOCITY PROFILE AT OUTLET

MODEL B-1

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

"B-2" which is shown photographically in Fig. D-11. Velocity profiles shown for the discharge of 1700 cfs in Fig. D-12 are too large when measured at the same distance into the river as for the previous models (15 ft). This occurs because the model outlet area has been reduced at the river line by moving the model closer to the river. Measurements at the same discharge of 1700 cfs but taken at 30 ft into the river are satisfactory, as shown in Fig. D-13.

Subsequent to the changes described above to the outlet end of the structure, changes were made at the pipe end where the origin of the two interior walls were first moved from 10 to 15 ft from the end of the pipe, model "B-3", and then to 20 ft, model "B-4", so that the final model tests were made on the design exactly as shown in Fig. A-1. Velocity profiles are not shown here for model "B-3" but are available in the project files; they meet the required objectives on the traverse line at 30 ft into the river. The velocity profiles for model "B-4" are presented in Figs. D-14 and D-15 for the discharges of 1700 and 500 cfs, respectively, and they, too, meet the required objectives at the traverse line 30 ft into the river.

Figures D-16 and D-17 show the maximum and minimum instantaneous velocities which were measured while recording the averages shown in Figs. D-14 and D-15. The averages from the latter figures are also plotted for comparison. Large eddies cannot be avoided at high discharges and these are responsible for the high and low instantaneous velocities. It is not believed that these fluctuations will be harmful to river traffic.

Final tests on model "B-4" were performed to determine for which discharges supercritical flow would occur at the pipe outlet followed by a hydraulic jump in the divided passages. This was the case down to about 1000 cfs. For smaller discharges there was no clear jump. (This explains why at lesser discharges most of the flow went into the center two channels without spreading.) Because of varying roughness and Reynolds number between model and prototype, the hydraulic jump may not occur at exactly the same discharges on the prototype.

It should be observed that the model was operated without a roof; hence, there was always a free surface. If a roof is built with its under surface near the water level of the river pool, there will be surges or gulping associated with the hydraulic jump and with river level changes. There should be a large air shaft in the vicinity of the junction of the pipe and outlet to

Fig. D-11

PHOTOGRAPH OF THE OUTLET END OF
MODELS B-2, B-3 AND B-4.

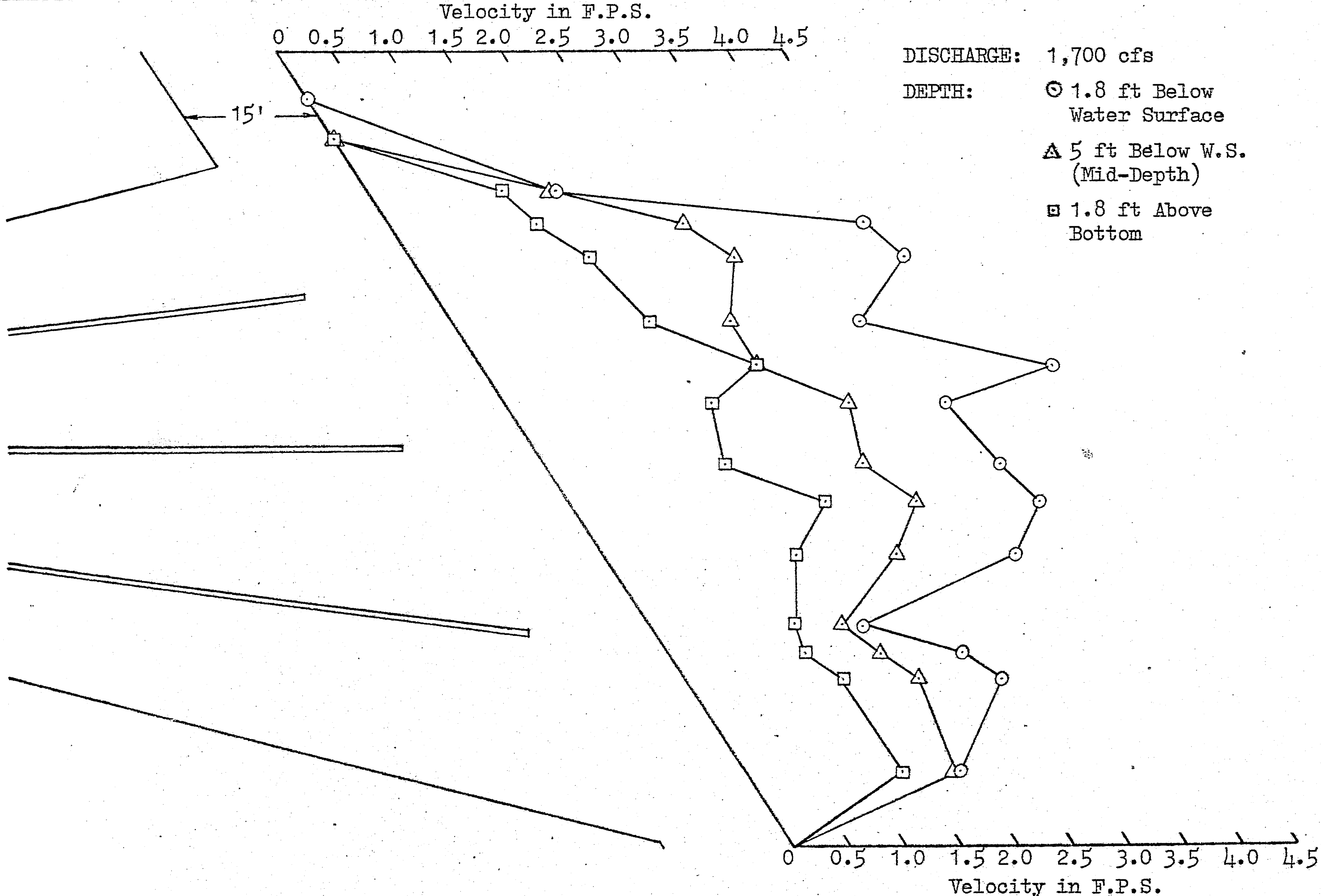


Fig. D-12

VELOCITY PROFILE AT OUTLET

MODEL B-2

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

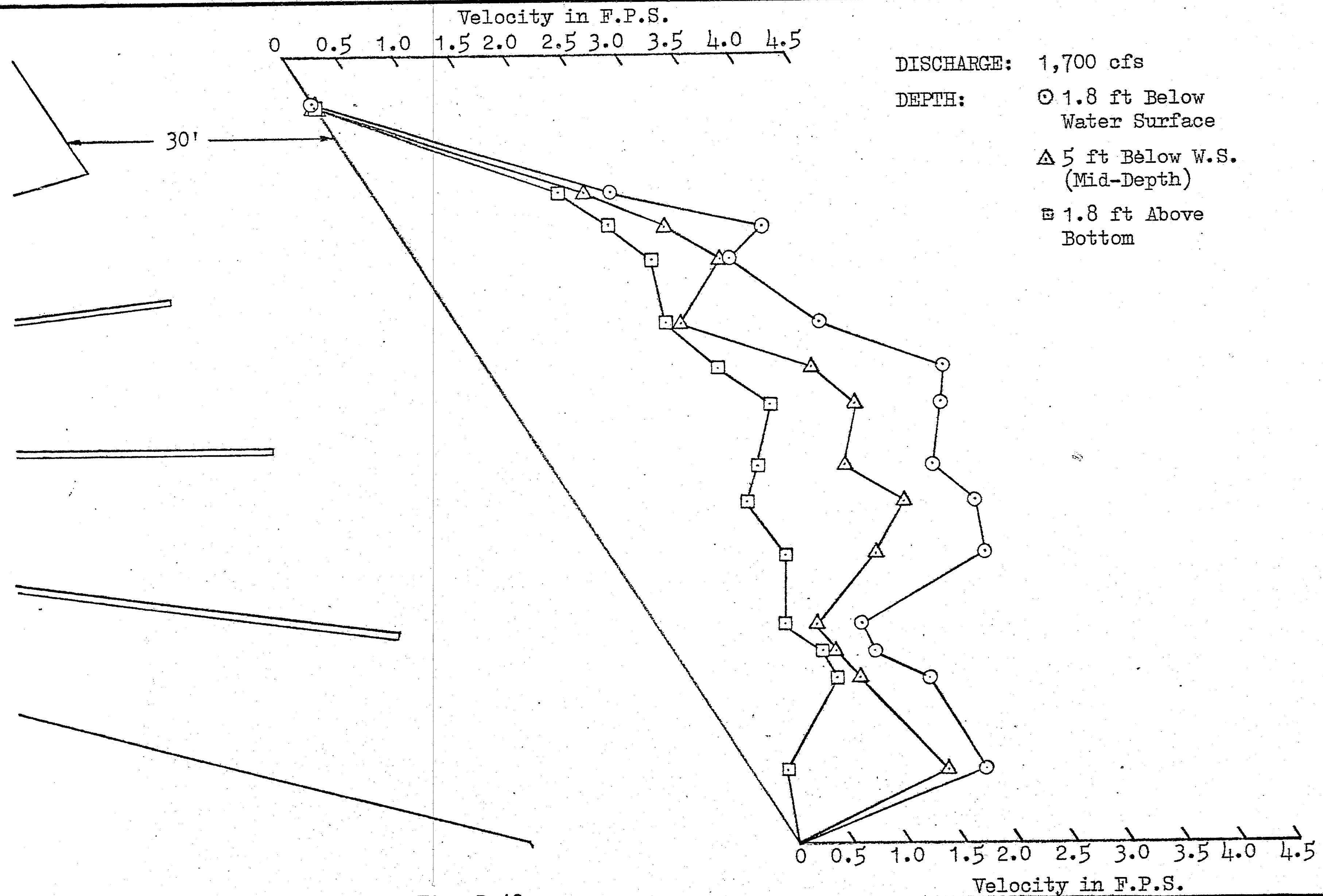
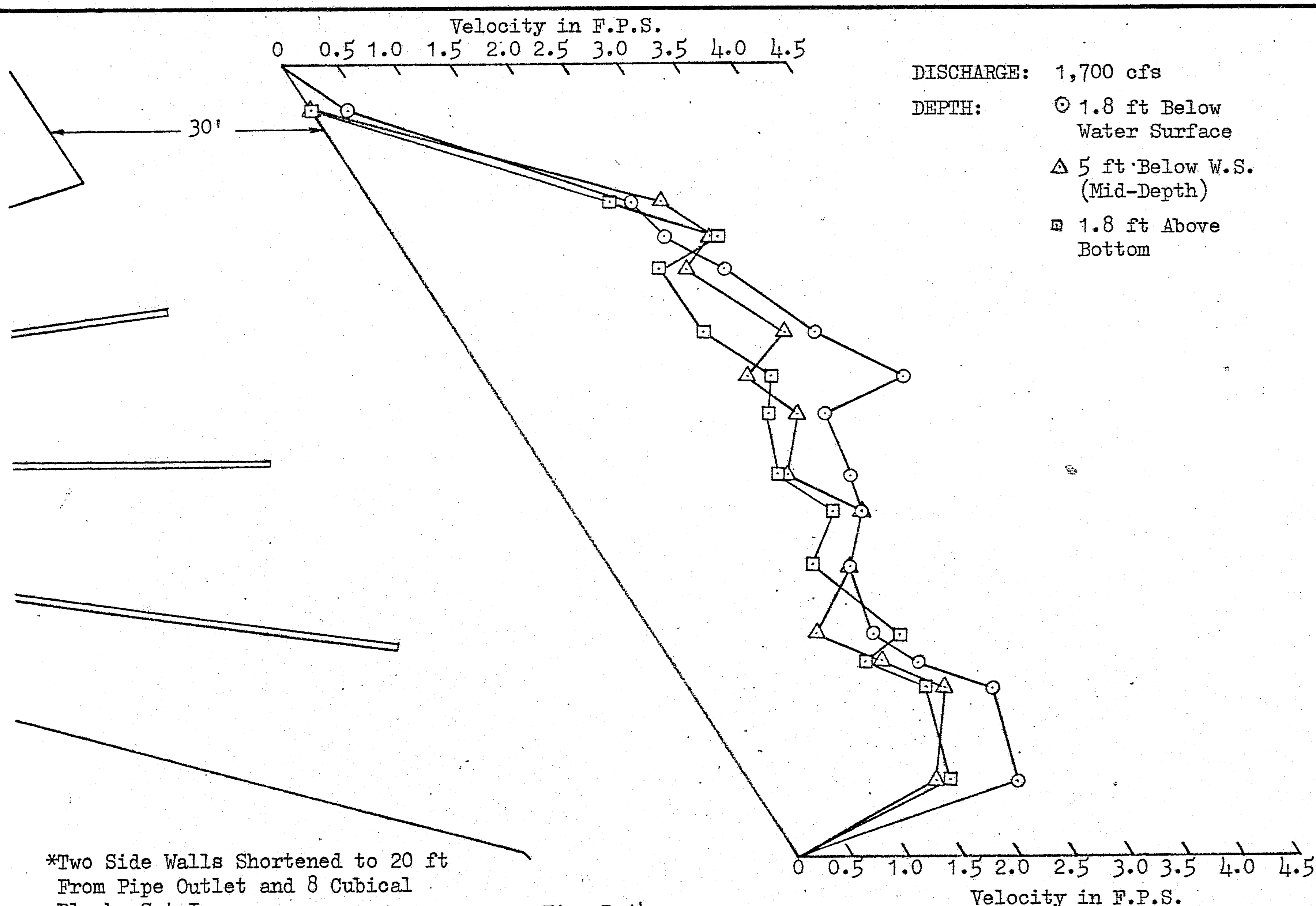


Fig. D-13

VELOCITY PROFILE AT OUTLET

MODEL B-2

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.



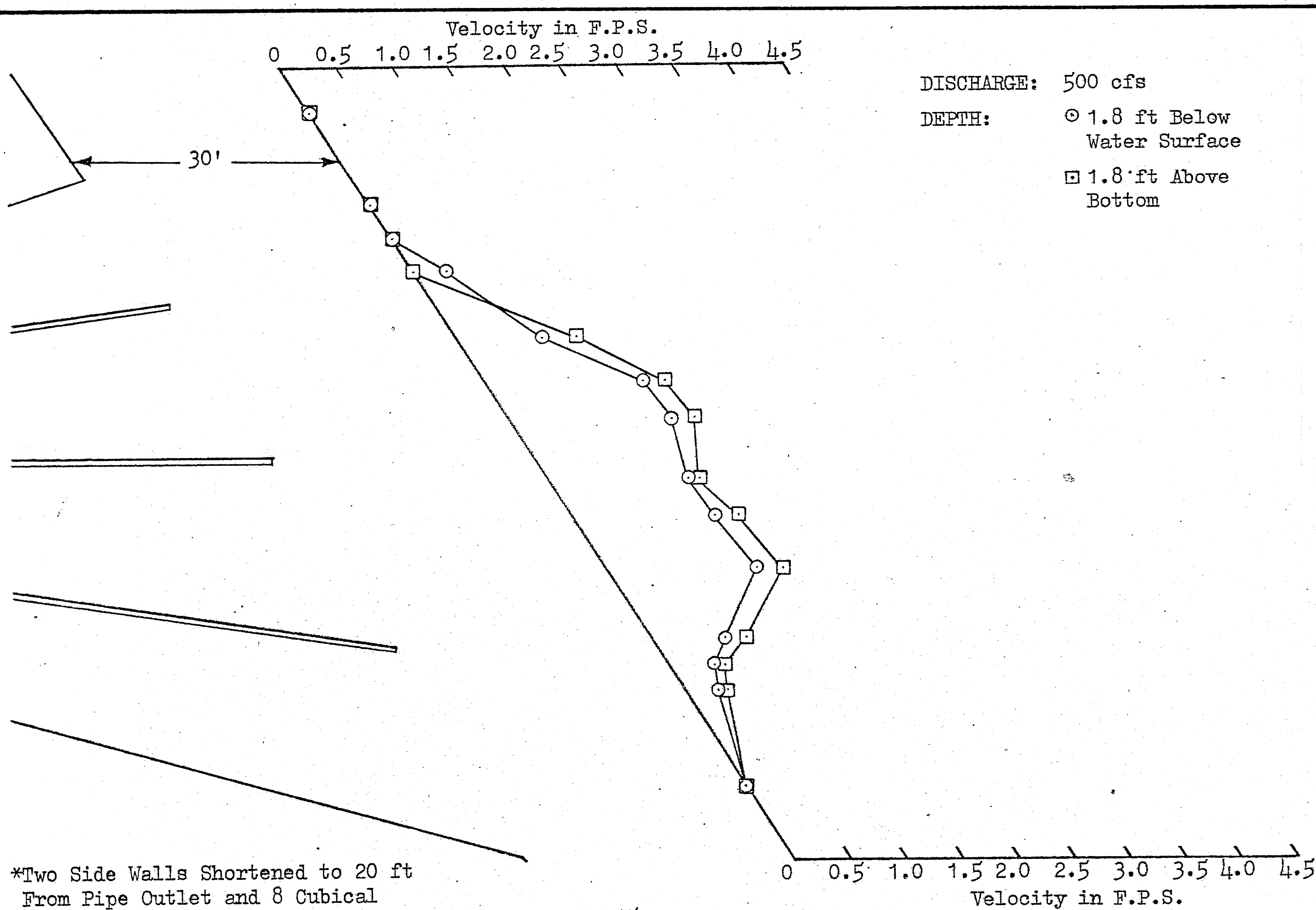
*Two Side Walls Shortened to 20 ft From Pipe Outlet and 8 Cubical Blocks Set In.

Fig. D-14

VELOCITY PROFILE AT OUTLET

MODEL B-4

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.



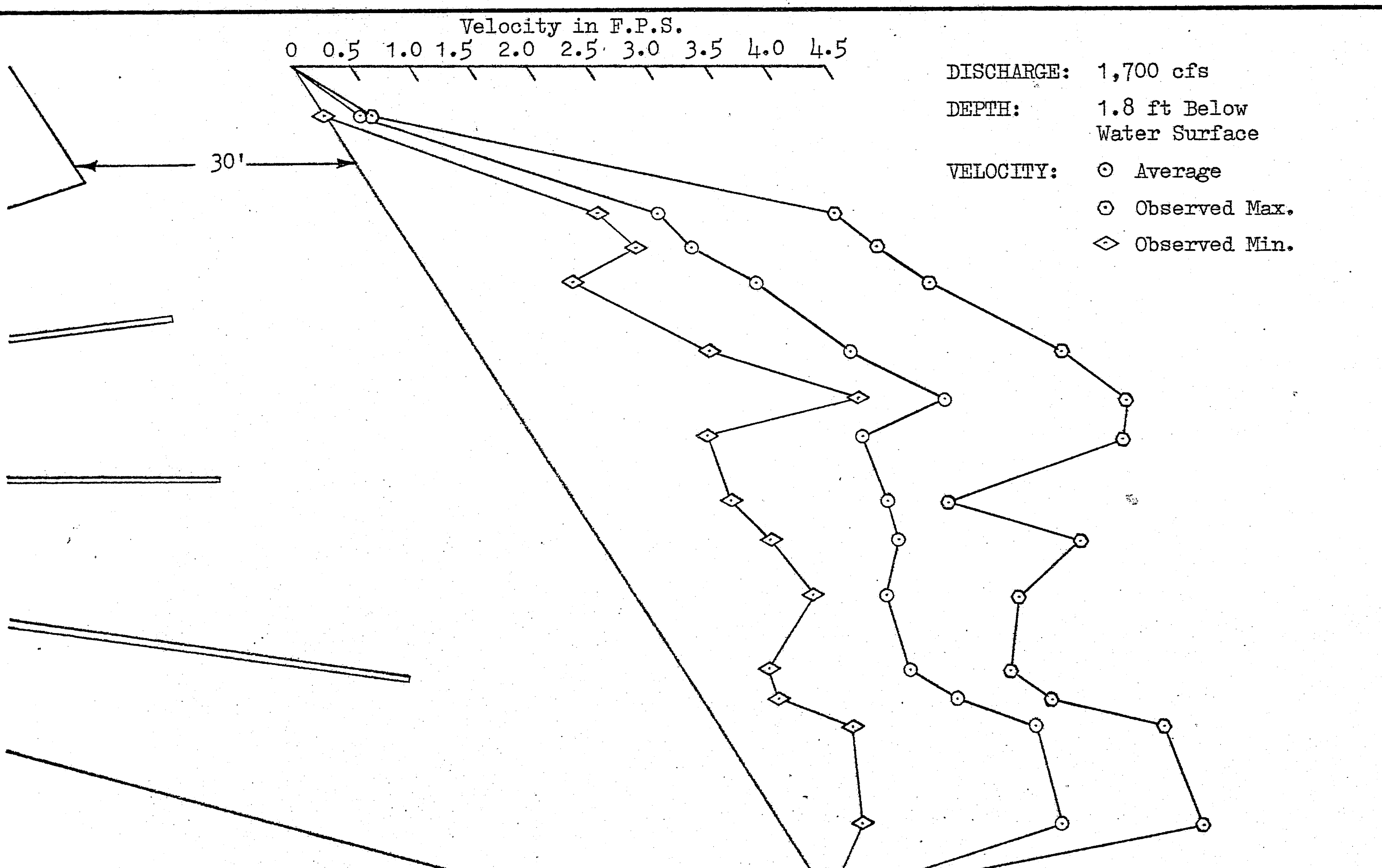
*Two Side Walls Shortened to 20 ft From Pipe Outlet and 8 Cubical Blocks Set In.

Fig. D-15

VELOCITY PROFILE AT OUTLET

MODEL B-4

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.



*Two Side Walls Shortened to 20 ft From Pipe Outlet and 8 Cubical Blocks Set In.

Fig. D-16

INSTANTANEOUS VELOCITIES AT OUTLET

MODEL B-4

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

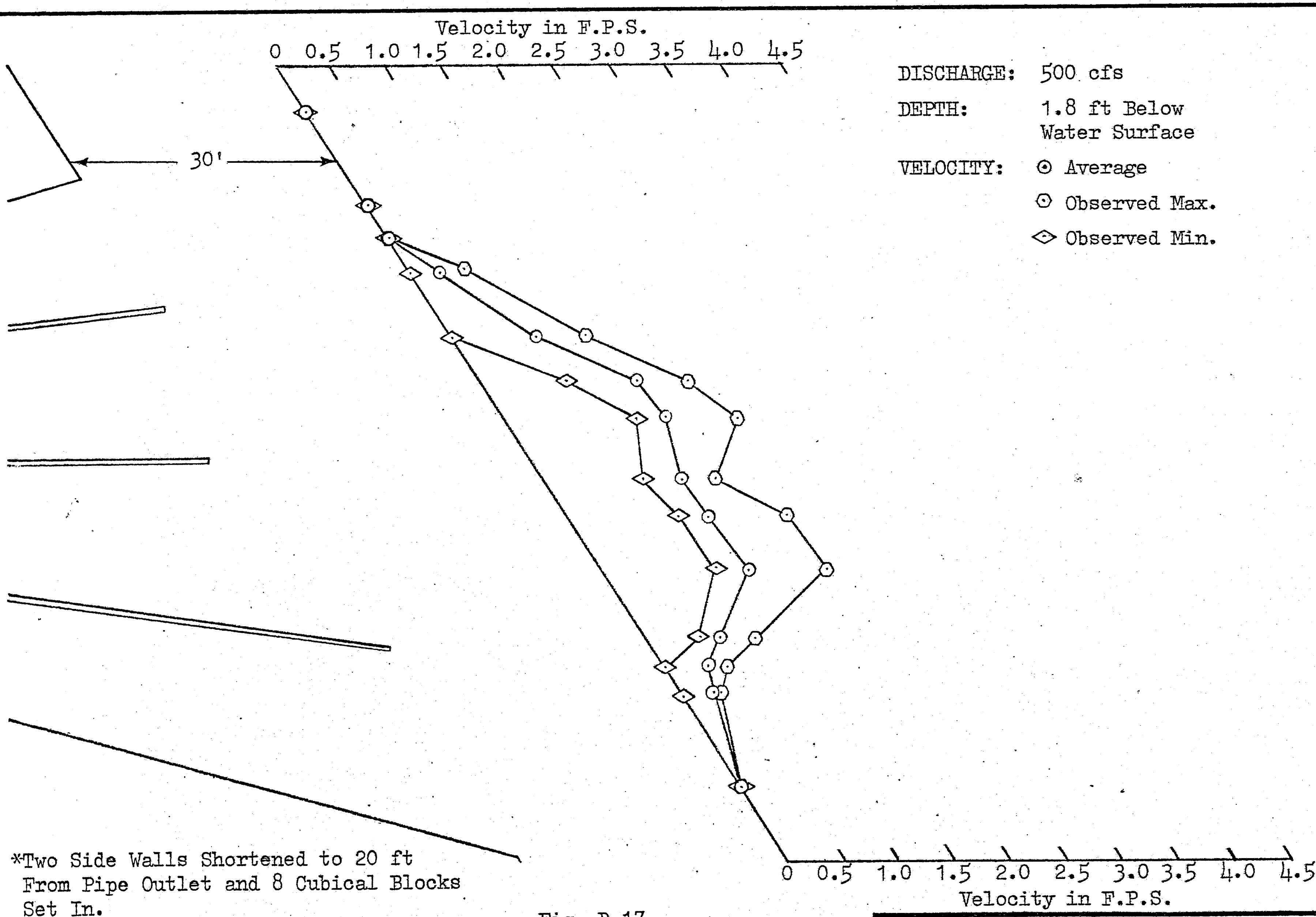


Fig. D-17

INSTANTANEOUS VELOCITIES AT OUTLET

MODEL B-4

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

relieve possible positive and negative pressures. Also, the roof should be designed to withstand pressures of several feet of water, either positive or negative. As mentioned in Part C, Ball measured pressures of $\pm$ 2.1 ft of water at an outlet to a sudden expansion.

These model study results have led to the recommendation in Part A and to the drawing in Fig. A-1.