In connection with (2) above it is clear that a pipeline sloping positively downwards away from the valve at A (towards B) would increase the spacing of the intermediate cavities and a negative upward slope away from the valve would decrease the spacing of the cavities.

It would be possible with very short pipelines to encourage or even prevent the occurence of intermediate cavities.

CONCLUSION

The problem of water column rupture outlined above has not been solved so much as disclosed in its greater complexity. The initiation of a cavity from an 'effective nucleus' remains a very difficult physical problem which depends on the pipeline and the cleanliness or otherwise of the water being conveyed. There does not seem to be any doubt that a cavity will form wherever circumstances require it, since complete de-earation of the water and a completely smooth interior are not contemplated in normal engineering applications.

Further research should be aimed at the determination of the influence of the factors listed under the Implications for Large Pipelines.

REFERENCES

1. Bergeron, L. J. P., - "Water Hammer in Hydraulics and Wave Surges in Electricity" (trans from the French) Wiley (1961).
2. Gayed, Y. K., and Kamel, M.Y.M., "Mechanics of Secondary Water Hammer Waves", Proc. Inst. Mech. Engs. Advance Copy 34/58, (1958)
3. Carstens, M. R. and Hagler, Jr. T. W. "Water Hammer Resulting from Cavitating Pumps", Proc. ASCE Journal Hydraulic Division 90, (1964).
4. Lieberman, P., "Blast-Wave Propogation in Hydraulic Conduits", Trans ASME, Journal Engineering for Power, 87 , Series A, pg 19, (1965
5. Sharp, B. B., "Cavity Formation in Simple Pipes due to Rupture of the Water Column", Nature, 185 , pg. 302 (1960).
6. Sharp, B. B., "The Growth and Collapse of Cavities Produced by a Rarefaction Wave with particular reference to Rupture of the Water Column", Dissertation submitted for a Ph.D., University of Melbourne, (1965).
7. Johnson, Virgil E., "Cavitation in Hydraulic Structures: Mechanics of Cavitation", Proc. ASCE, Journal Hydraulics Division., 89, pg.251 (1963).
8. Lupton, H. R. "Graphical Analysis of Pressure Surges in Pumping Systems", Journal Instn. of Water Engineers, 1, pg. 87, (1953).

EVALUATION OF MANNING'S n FOR STEADY NON-UNIFORM FLOWS

By

K.V.N. SARMA
Senior Research Fellow
Civil and Hydraulic Engg. Dept.,
Indian Institute of Science
BANGALORE, INDIA

and

S.R. SASIKANTH
Scientific Officer,
Civil and Hydraulic Engg. Dept.,
Indian Institute of Science
BANGALORE, INDIA

ABSTRACT

Manning's frictional coefficient for steady flows in open flumes is usually evaluated from the flow measurements in the channel reaches where the flow can be considered to be uniform. This warrants the presence of very long test sections for the establishment of uniform flow. In this paper it is demonstrated that by using the energy slope Manning's n can be evaluated with very good accuracy for any irregular non-uniform, steady flow in a uniform channel where the banks and the bed are of the same roughness. It is also shown that by this method the test reaches required are very small. Experiments conducted to verify the method in very short reaches have yielded consistent and comparable values.

INTRODUCTION

Manning's formula is the most commonly used formula for computing velocities of flow in channels with steady discharges. But the use of this formula requires the parallelism of the water surface and the bed of the channel. Successful use of this formula can be made only when the frictional coefficient n is known accurately.

To estimate the Manning's n it is thus far conventional to consider test sections where the flow can be reckoned to be uniform without much error. Actually no flow, in nature, is uniform. Even in the laboratory studies, it is very difficult to arrive at uniform flow conditions, even if very long channels of uniform section are at hand. In many cases of gradually varying non-uniform flow, the uniform flow is approached as a limit far away from the inlets to uniform channels. So to get at a reach where the non-parallelism of water surface and the bed of the channel is to be negligible the channel need be very long. Thus, whenever the quantities of flow in the channels are evaluated by using Manning's coefficient, proper care should be taken not to include any non-uniform flow reach in the test section under consideration.

When the channel available is not of adequate length for the establishment of uniform flow and when the water surface profile is irregular no mention is made, to the authors' knowledge, about the method of evaluating Manning's n . In this paper, a generalised formula is suggested to evaluate Manning's n .

METHOD OF ESTIMATION OF n

Manning's equation for uniform flows reads as follows :

$$V = \frac{1.486}{n} . R^{2/3} . S_o^{1/2} \quad ..... \quad (1)$$

where V = Mean velocity of flow in fps ;
R = Hydraulic radius in ft. = Area of flow/wetted perimeter;
S_o = Slope of the channel bed ; and
n = Coefficient of friction.

When the flow is gradually varied, S_o in equation (1) is replaced by the friction slope S_f.

It appears logical to use the energy slope, S_e , in the Manning's formula when accelerative and decelerative terms are not of a negligible magnitude. Energy slope S_e will be equal to friction slope S_f when the accelerative and decelerative terms are negligible and to the bed slope S_o when the flow is uniform. Then the formula to be used is

$$V = \frac{1.486}{n} . R^{2/3} . S_e^{1/2} \quad ..... \quad (2)$$

where V and R are the mean velocity and hydraulic radius at the section under consideration respectively and S_e is the slope of the energy line at that section. With the above assumption, the calculations for the evaluation of n will be as follows.

Solving for S_e , we get

$$S_e = \frac{V^2 . n^2}{(1.486)^2 . R^{4/3}} \quad ..... \quad (3)$$

Considering an elementary reach of length ΔX and assuming the energy line in that reach to have the same slope throughout, we get

$$\Delta E_L = S_e . \Delta X = \frac{V^2 . n^2 . \Delta X}{(1.486)^2 . R^{4/3}} \quad ..... \quad (4)$$

where ΔE_L is the energy loss within that elementary reach. Then, for the entire length of the reach, (i.e. from $X = 0$ to $X = L$), the total energy loss, E_L , is given by

$$E_L = \sum_{X=0}^{L} S_e . \Delta X = \frac{n^2}{(1.486)^2} \sum_{X=0}^{L} \frac{V^2 . \Delta X}{R^{4/3}} \quad .. \quad (5)$$

Here, it is assumed that the friction coefficient n is not changing along the length of the channel or with the stage of flow, i.e. the bed and sides are of the same roughness. From equation (5) ,

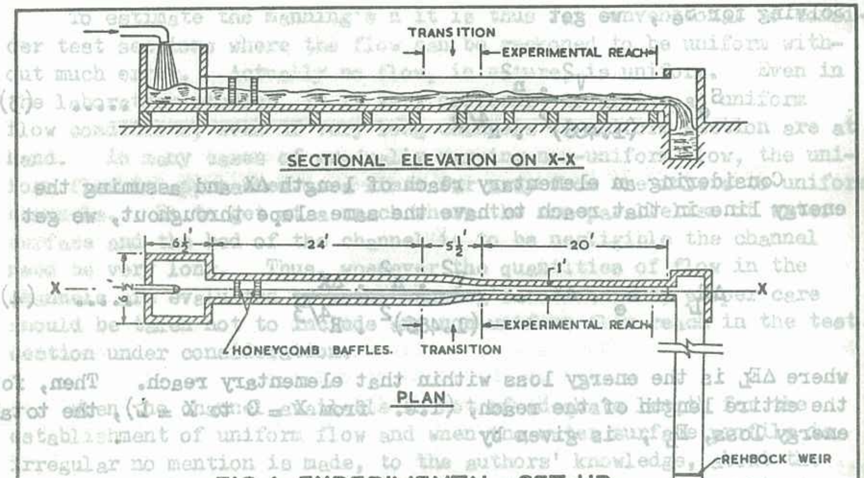
$$n = \left[\frac{E_L}{\sum_{X=0}^{L} \frac{V^2 . \Delta X}{(1.486)^2 . R^{4/3}}} \right]^{1/2} \quad ..... \quad (6)$$

ENERGY LOSS IN THE REACH CONSIDERED

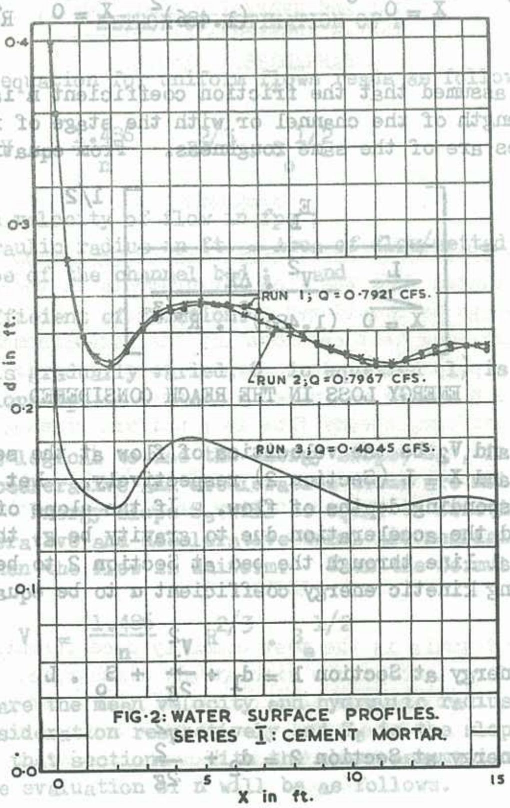
Let V_1 and V_2 be the velocities of flow at the sections $X = 0$ (Section 1) and $X = L$ (Section 2) respectively. Let d_1 and d_2 be the corresponding depths of flow. If the slope of the channel bed is S_o and the acceleration due to gravity be g, then considering the horizontal line through the bed at Section 2 to be the datum, we have (assuming kinetic energy coefficient α to be equal to one)

$$\text{Total energy at Section 1} = d_1 + \frac{V_1^2}{2g} + S_o . L \quad ..... \quad (7)$$

$$\text{and total energy at Section 2} = d_2 + \frac{V_2^2}{2g} \quad ..... \quad (8)$$



FIG·1. EXPERIMENTAL SET-UP.



FIG·2: WATER SURFACE PROFILES.
SERIES I: CEMENT MORTAR.

Hence, the energy loss E_L in the test reach is given by

$$E_L = (d_1 + \frac{V_1^2}{2g} + S_o . L) - (d_2 + \frac{V_2^2}{2g}) \quad ..... \quad (9)$$

For evaluating n the value of E_L from equation (9) should be substituted in equation (6).

EXPERIMENTAL SET-UP

The experiments were conducted in a one foot wide rectangular flume, 20 feet in length, constructed with 1/240 slope inside a 2'-wide rectangular flume with a horizontal bed. A smooth transition was provided to connect the two channel sections (See Fig. 1).

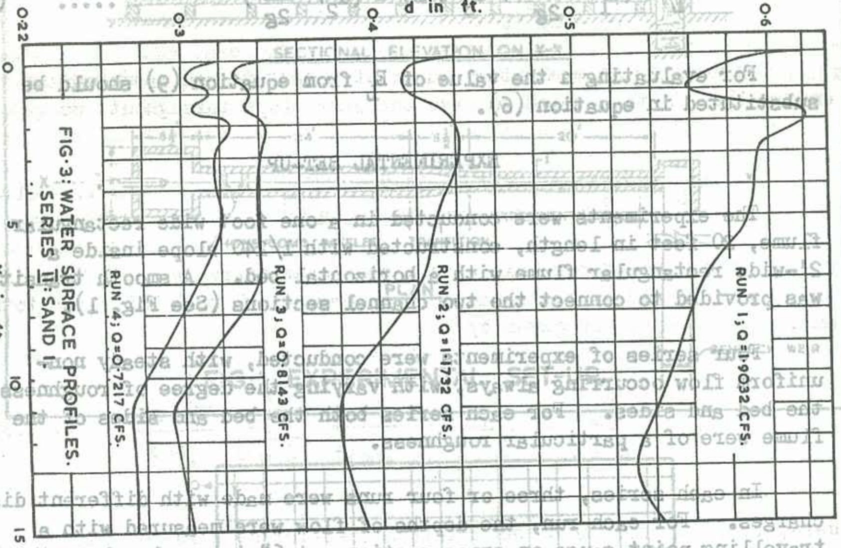
Four series of experiments were conducted, with steady non-uniform flow occurring always, with varying the degree of roughness of the bed and sides. For each series both the bed and sides of the flume were of a particular roughness.

In each series, three or four runs were made with different discharges. For each run, the depths of flow were measured with a travelling point gauge on cross-sections at 6" intervals along the length of the channel and at 2" intervals along the transverse direction at each cross-section of the test channel. At each point, the measurements for the channel bed and the water surface were taken. Measurements were taken starting from the section where the 1'-0" wide channel began and were continued over a distance of 15' to 17' downstream of the starting point. The flow from the test flume is collected in another channel and was led over a Rehbock weir which was calibrated previously. The flow discharges were calculated from the measured heads over the Rehbock weir. The flow, in each case, was steady, the head over the Rehbock weir being the same throughout the time taken for measurements of depths.

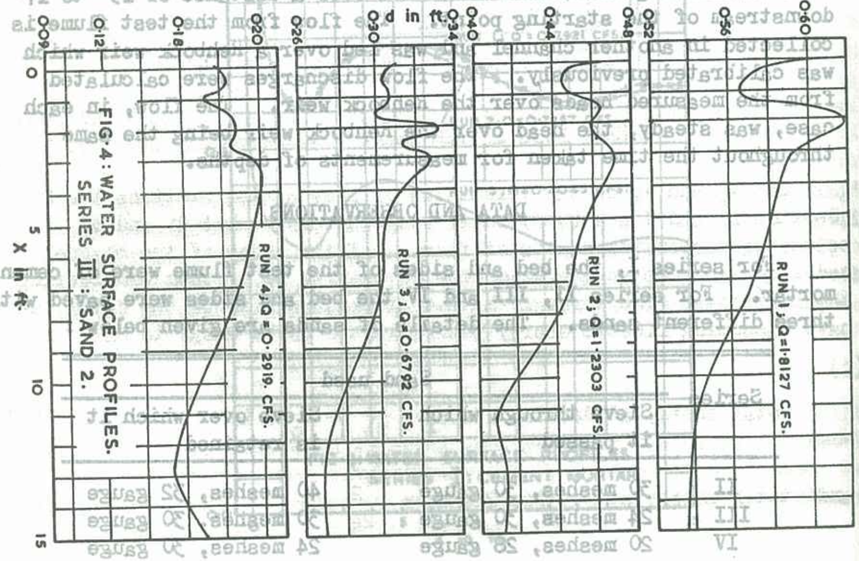
DATA AND OBSERVATIONS

For series I, the bed and sides of the test flume were of cement mortar. For series II, III and IV the bed and sides were paved with three different sands. The details of sands are given below :

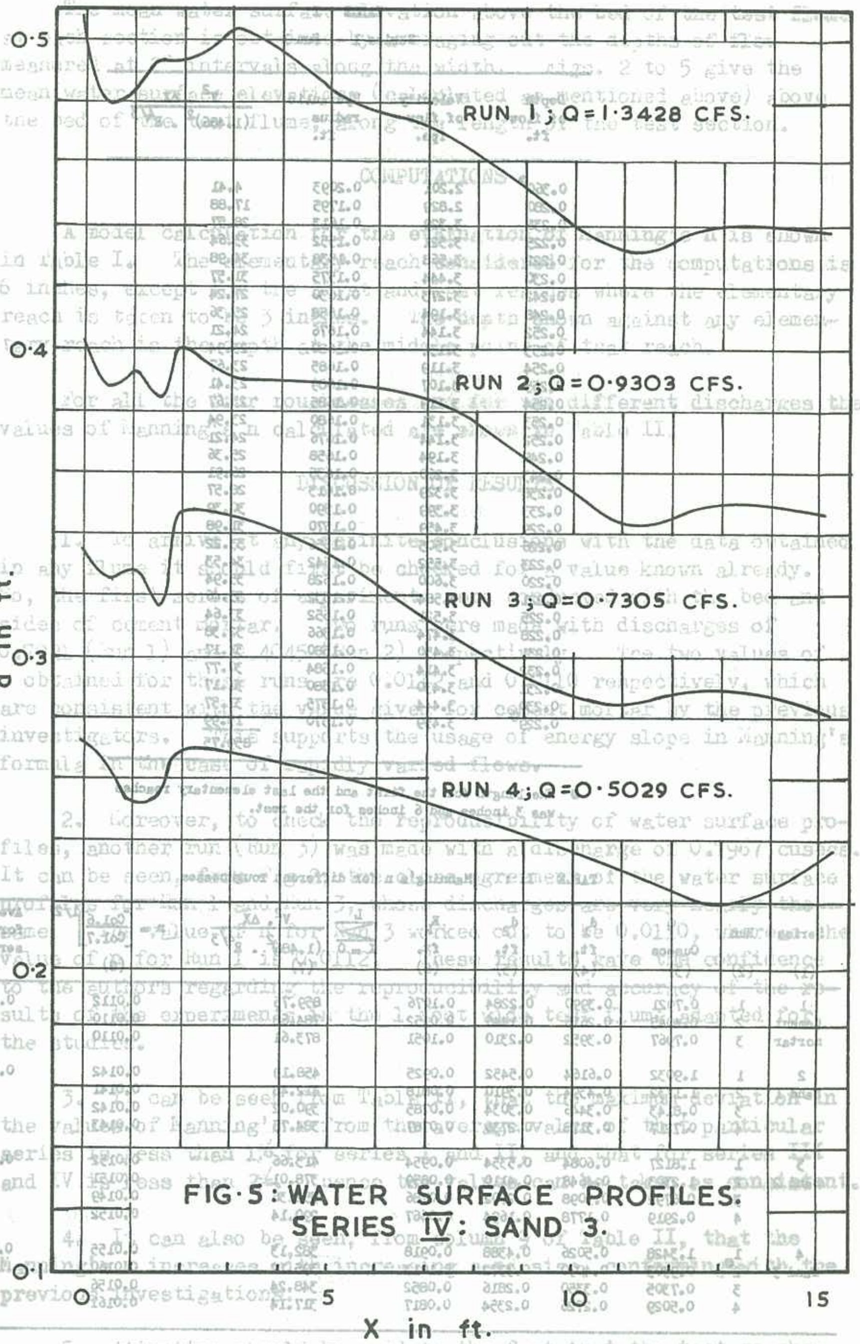
Series	Sand used	
	Sieve through which it passed	Sieve over which it is retained
II	30 meshes, 30 gauge	40 meshes, 32 gauge
III	24 meshes, 30 gauge	30 meshes, 30 gauge
IV	20 meshes, 28 gauge	24 meshes, 30 gauge



FIG·3: WATER SURFACE PROFILES. SERIES II: SAND I.



FIG·4: WATER SURFACE PROFILES. SERIES III: SAND 2.



FIG·5: WATER SURFACE PROFILES. SERIES IV: SAND 3.

TABLE I

Series 1 - Run 1

Depth of flow ft.	Velocity of flow fps.	Hydraulic radius ft.	$\frac{V^2 . \Delta X}{(1.486)^2 . R^{4/3}}$
0.360@	2.201	0.2093	4.41
0.280	2.829	0.1795	17.88
0.238	3.329	0.1613	28.57
0.225	3.521	0.1552	33.64
0.222	3.568	0.1538	34.98
0.230	3.444	0.1575	31.57
0.242	3.273	0.1630	27.24
0.248	3.194	0.1658	25.36
0.252	3.144	0.1676	24.21
0.253	3.131	0.1680	23.94
0.254	3.119	0.1685	23.67
0.255	3.107	0.1689	23.41
0.254	3.119	0.1685	23.67
0.253	3.131	0.1680	23.94
0.252	3.144	0.1676	24.21
0.248	3.194	0.1658	25.36
0.243	3.260	0.1635	26.91
0.238	3.329	0.1613	28.57
0.233	3.399	0.1590	30.39
0.229	3.459	0.1570	31.98
0.226	3.505	0.1556	33.22
0.223	3.552	0.1542	34.53
0.220	3.600	0.1528	35.94
0.221	3.584	0.1532	35.46
0.225	3.521	0.1552	33.64
0.228	3.474	0.1566	32.38
0.231	3.430	0.1580	31.17
0.232	3.414	0.1584	30.77
0.231	3.430	0.1580	31.17
0.230	3.444	0.1575	31.57
0.229@	3.459	0.1570	15.99
			859.75

@ The length for the first and the last elementary reaches was 3 inches and 6 inches for the rest.

TABLE II : Manning's n for different roughnesses

Series (1)	Run (2)	Q Cusecs (3)	d_1 ft. (4)	d_2 ft. (5)	E_L ft. (6)	$\sum_{X=0}^{L} \frac{V^2 . \Delta X}{(1.486)^2 . R^{4/3}}$ (7)	$n = \left[\frac{Col.6}{Col.7}\right]^{1/2}$ (8)	Average n for each series (9)
1 Cement mortar	1	0.7921	0.3990	0.2284	0.1076	859.75	0.0112	0.0111
	2	0.4045	0.2616	0.1446	0.0952	784.68	0.0110	
	3	0.7967	0.3952	0.2310	0.1051	873.61	0.0110	
2 Sand 1	1	1.9032	0.6164	0.5452	0.0925	458.19	0.0142	0.0142
	2	1.1732	0.4394	0.3910	0.0818	412.48	0.0141	
	3	0.8143	0.3446	0.3034	0.0785	390.02	0.0142	
	4	0.7217	0.3178	0.2736	0.0787	384.75	0.0143	
3 Sand 2	1	1.8127	0.6084	0.5354	0.0954	413.66	0.0152	0.0151
	2	1.2303	0.4648	0.4110	0.0859	378.01	0.0151	
	3	0.6792	0.3098	0.2702	0.0786	352.36	0.0149	
	4	0.2919	0.1778	0.1694	0.0667	290.14	0.0152	
4 Sand 3	1	1.3428	0.5026	0.4388	0.0918	382.13	0.0155	0.0158
	2@	0.9303	0.4030	0.3372	0.1013	395.12	0.0160	
	3	0.7305	0.3350	0.2816	0.0852	348.24	0.0156	
	4	0.5029	0.2726	0.2354	0.0817	317.14	0.0161	

@ Length of experimental reach for this run was 17 feet and 15 feet in the case of all the other runs.

The mean water surface elevation above the bed of the test flume at each section is obtained by averaging out the depths of flow measured at 2" intervals along the width. Figs. 2 to 5 give the mean water surface elevations (calculated as mentioned above) above the bed of the test flume, along the length of the test section.

COMPUTATIONS

A model calculation for the evaluation of Manning's n is shown in Table I. The elementary reach considered for the computations is 6 inches, except for the first and last reaches where the elementary reach is taken to be 3 inches. The depth shown against any elementary reach is the depth at the middle point of that reach.

For all the four roughnesses and for the different discharges the values of Manning's n calculated are shown in Table II.

DISCUSSION OF RESULTS

1. To arrive at any definite conclusions with the data obtained in any flume it should first be checked for a value known already. So, the first series of experiments were conducted with the bed and sides of cement mortar. Two runs were made with discharges of 0.7921 (Run 1) and 0.4045 (Run 2) respectively. The two values of n obtained for these runs are 0.0112 and 0.0110 respectively, which are consistent with the value given for cement mortar by the previous investigators. This supports the usage of energy slope in Manning's formula in the case of rapidly varied flows.

2. Moreover, to check the reproducibility of water surface profiles, another run (Run 3) was made with a discharge of 0.7967 cusecs. It can be seen, from Fig.2, the close agreement of the water surface profiles for Run 1 and Run 3, whose discharges are very nearly the same. The value of n for Run 3 worked out to be 0.0110, whereas the value of n for Run 1 is 0.0112. These results gave the confidence to the authors regarding the reproducibility and accuracy of the results of the experiments in the 1 foot wide test flume adapted for the studies.

3. It can be seen from Table II, that the maximum deviation in the values of Manning's n from the average value of that particular series is less than 1% for series I and II, and that for series III and IV is less than 2%. Hence the values can be taken as consistent.

4. It can also be seen, from column 9 of Table II, that the Manning's n increases with increasing sand size, conforming with the previous investigations.

5. Attention should be paid to the fact that the test reaches considered are only 15 ft. to 17 ft. in length. These lengths are

very short when compared to the long reaches required for evaluating Manning's n from uniform flows.

CONCLUSIONS

1. The Manning's formula can be generalised if the slope term used is the slope of the energy line.

2. The above method is particularly advantageous when only short reaches of flumes are available and no uniform flow can be obtained.

ACKNOWLEDGEMENTS

The authors wish to thank Professor N.S. Govinda Rao and Professor K.Seetharamiah for the encouragement given and facilities provided.

MODEL CALCULATION

Series 1 - Run 1

Bed and Sides of Cement Mortar.
Slope of bed = S_o = 1/240 ; L = 15 ft.
Discharge = 0.7921 cusecs.

Total Energy at X = 0

$d_1 = 0.3990$ ft.

$\frac{V_1^2}{2g} = 0.0612$ ft.

$S_o \cdot L = 0.0625$ ft.

Total energy = 0.5227 ft.

Total Energy at X = 15

$d_2 = 0.2284$ ft.

$\frac{V_2^2}{2g} = 0.1867$ ft.

Total energy = 0.4151'

$$\text{Loss of Energy } E_L = 0.5227 - 0.4151$$

$$= 0.1076 \text{ ft.}$$

$$= n^2 \cdot \sum_{x=0}^{15} \frac{V^2 \cdot \Delta x}{(1.486)^2 \cdot R^{4/3}}$$

$$= n^2 \, (859.75) \quad \ldots \quad \text{(from Table I)}$$

$$n = \sqrt{\frac{0.1076}{859.75}}$$

$$= 0.0112$$

THE NON-DIMENSIONAL REPRESENTATION OF PROFILES OF GRADUALLY VARIED FLOW

By H.R. Vallentine
The University of Newcastle

ABSTRACT

Generalised profiles, in non-dimensional form, are presented for type M and type S water surface profiles of gradually varied flow in wide channels. The form of each type is seen to depend upon the Froude number of the uniform flow.

INTRODUCTION

It has been shown (1) that a pattern of similarity exists for M1-type profiles in rectangular channels of given width, and that these profiles can be represented on a non-dimensional plot. For a given maximum-depth to normal-depth ratio, all profiles plot in an ordered series according to their uniform flow Froude numbers, irrespective of their actual bed slopes, Manning n values and normal flow depths.

The generalised profiles in the non-dimensional presentation provide a ready means of investigating the effects of changes in the channel and flow characteristics on the form of the water surface profiles; and they provide the practising engineer with an approximate guide to the dimensions of profiles for particular cases, thus obviating the need for tedious computation in preliminary studies.

In this paper, the non-dimensional approach is extended to cover the M2, M3, S1, S2 and S3 profiles.

DEVELOPMENT OF PROFILES

The differential equation for gradually varied flow profiles in very wide channels, based on the Manning formula, is

$$\frac{dy}{dx} = S_o \frac{1 - \left(\frac{y_o}{y}\right)^{10/3}}{1 - \left(\frac{y_c}{y}\right)^3} \qquad 1.$$