Paper No. 5082.

"The Flow of Water in Short Channels." †

By Charles Frederick Jason Lisle, B.Sc., Assoc. M. Inst. C.E.

Correspondence.

Mr. E. H. Essex observed that he had already shown * that the use of a formula of the type $N=f\frac{V^x}{R^y}$ could not comply with dimensional similitude unless the sum of the values x and y was 3. It could be shown that the Author's curve for type III in *Fig. 2* (p 308 §) for the critical depth was altogether inaccurate, and, in fact, it was bound to be so, if only on account of its being based upon the logarithmic formula of Mr. A. A. Barnes, namely, $V=92{\cdot}1R^{0{\cdot}602}S^{0{\cdot}466}$, or $S=0{\cdot}000063\frac{V^{2{\cdot}15}}{R^{1{\cdot}29}}$, the sum of the indices x and y of which was 3·44. Mr. Essex did not like any of the Barnes formulas because it was tedious work comparing them with the value of C in the Chezy formula. The best logarithmic formula to use for earth channels, if such a formula had to be adopted, was that of Professor T. Claxton Fidler, namely $N=A\frac{V^{2{\cdot}1}}{R^{1{\cdot}5}}$, where V varied as $R^{0{\cdot}60}$. Mr. Essex had used it with slide-rule calculations to compile Table XIII (p. 426) for comparison with Table I on p. 293 §. Table I showed what would happen in a channel of type I (Appendix I, *Fig. 2* (p. 308 §)) having a fall of 1 in 1,000 with a discharge of 900 cusecs—that was to say, a channel not in regime. Table XIII, however, showed that the critical depth of the same channel was 7·5 feet for a discharge of 900 cusecs if laid to a fall of 1 in 1,000, and also showed what would be the critical velocities and discharges at various depths for the same fall.

Table I showed that the channel required a velocity of 10·6 feet per second at a depth of 4·9 feet—quite an inadmissible velocity in an earth channel of the type under discussion.

On p. 307 § the Author expressed the hope that engineers who had available the results of measurements of non-uniform flows on full-size works would make such measurements known in the Correspondence. Mr. Essex was not clear as to what advantage was to be gained by the calculation of non-uniform flows in channels, but if agreement were reached upon the correct method of designing channels for uniform flow,

† Journal Inst. C.E., vol. 7 (1937–38), p. 287 (December 1937).

* *Correspondence on Paper on* "The Effect of the Form of Cross-section on the Capacity and Cost of Trunk Sewers," by Mr. T. Donkin ; p. 410, *ante*.

§ Page numbers so marked refer to the Paper. (Footnote (†) above.)—Sec. Inst. C.E.

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TABLE XIII.

Fall of Channel 1 in 1,000; Chezy $C = 13{\cdot}7 \log VR/\nu$; Froude number $= 0{\cdot}433 \log VR/\nu$.

Depth of flow: feet.	Area: square feet.	Perimeter, P: feet.	Hydraulic mean depth, R: feet.	$R^{\cdot 060}$	V: feet per second.	Q: cusecs.	VR.	$\log VR/\nu$	C.	$V/\sqrt{R}$.	From Barnes's formula. C.	From Barnes's formula. Gradient required, C^2R/V^2.
7·5	159·5	37·1	4·30	2·40	5·65	900	24·3	6·29	86·2	2·72	78·0	825
7·0	143·8	35·3	4·07	2·32	5·46	785	22·3	6·26	85·8	2·71	77·0	810
6·5	128·5	33·5	3·84	2·24	5·28	680	20·3	6·21	85·2	2·69	76·0	800
6·0	114·0	31·7	3·60	2·16	5·09	580	18·3	6·17	84·6	2·67	75·5	800
5·8	108·5	31·0	3·51	2·12	5·00	542	17·5	6·15	84·3	2·66	75·0	800
5·6	103·0	30·2	3·41	2·09	4·92	507	16·8	6·14	84·1	2·65	74·5	790
5·4	97·8	29·5	3·32	2·06	4·85	474	16·1	6·12	83·8	2·65	74·0	780
5·2	92·6	28·8	3·22	2·02	4·76	441	15·3	6·09	83·6	2·64	73·5	775
4·9	85·0	27·7	3·07	1·96	4·61	392	14·2	6·06	83·2	2·63	73·0	770

the measurement of friction-loss for excess flows in a channel was bound to follow on the same lines; in that respect he thought that Mr. Gerald Lacey's Paper * should be studied; the only drawback to that Paper was that the calculations were based on Manning's logarithmic formula, modified by Mr. Lacey from $C=\frac{1{\cdot}486}{N}\sqrt[6]{R}$ to $C=\frac{1{\cdot}3458}{N_a}\sqrt[4]{R}$ when VR was unity. That modification complicated the consideration of that Paper, but if Mr. Lacey's Table II on p. 430 of his Paper * were further modified and extended to comply with the simple Chezy formulas as shown in Table XIV opposite, Mr. Lisle would be able to make a comparison therewith to enable him to derive a basis for the correct measurement of frictional loss.

Table XIV gave concisely and fairly a summary of Mr. Lacey's views on friction-loss, and of his general views of the rules governing the hydraulic flow in streams carrying silt; as such the figures might supply the answer to Mr. Lisle's query.

Mr. W. H. R. Nimmo, of Brisbane, observed that the Author expressed the hope that engineers would make known any measurements of non-uniform flows in full-size works. That the behaviour of large quantities of water under various conditions of non-uniform flow could be predicted with reasonable accuracy by calculation had been confirmed by experiments made in 1923 by Mr. Julian Hinds †, and by those carried out by Mr. Nimmo in 1922–23 ‡. The correctness of the theoretical results obtained had been supported by numerous practical applications made during recent years.

* "Uniform Flow in Alluvial Rivers and Canals." Minutes of Proceedings Inst. C.E., vol. 237 (1933–34, Part 1), p. 421.

† "Side Channel Spillways: Hydraulic Theory, Economic Factors, and Experimental Determination of Losses." Trans. Am. Soc. C.E., vol. 89 (1926), p. 881.

‡ "Side Spillways for Regulating Diversion Canals." *Ibid.*, vol. 92 (1928), p. 1561.

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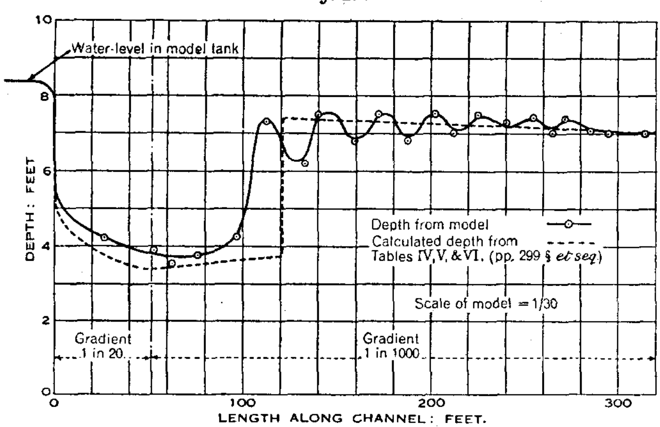
TABLE XIV

NOTE.—Log $VR/\nu = 5$, at $VR = 1$ for water at a temperature of 70° F.

N_a (Lacey).	N (Manning).	Chezy value at $VR=1$: $C = A \log \frac{VR}{\nu}$.	$A = C/\log \frac{VR}{\nu}$.	Froude No. $V/\sqrt{R} = C\sqrt{S}$ / Silt-factor c'. / $E_x \log VR/\nu$.	Efficiency-factor E for $V/\sqrt{R}$.	$(E/A)^2 = S$.	$1/\sqrt{S} = A/E$.	Lacey's silt-factor: $0.75\,(c')^2$.	Diameter of silt-grains: inches.
0·0150	0·0165	90	18	0·512	0·1024	0·0000323	176	0·200	—
0·0175	0·0193	77	15·4	0·697	0·1394	0·000082	110	0·365	0·0028
0·0200	0·0221	67	13·8	0·910	0·1820	0·000174	76	0·622	0·0055
0·0225	0·0248	60	12·0	1·151	0·2302	0·000369	52	1·000	0·016
0·0250	0·0276	54	10·8	1·422	0·2840	0·000690	38	1·530	0·040
0·0275	0·0304	49	9·8	1·720	0·3440	0·001230	28·5	2·240	0·080
0·0300	0·0330	45	9·0	2·047	0·4094	0·002070	22	3·160	0·155
0·0400	0·0442	33·5	6·7	3·550	0·7100	0·0112	9·45	9·500	1·55
0·0435	0·0480	31	6·2	4·280	0·8560	0·0191	7·25	13·80	3·00

Dr. Alexander Thom thought that it might be of interest to record an experiment, made in the James Watt Engineering Laboratories in

Fig. 11.



Glasgow, to verify the method used in the Paper to calculate the position of the hydraulic "jump." A model of the channel discussed in Example III (pp. 298 § *et seq.*) was made to a scale of one-thirtieth full size. The flume used was therefore 4 inches wide, and had a slope of 0·05 for the first 21 inches and a slope of 0·001 thereafter. The discharge of the channel being 700 cusecs, that of the model should be $700 \div 30^{2\frac{1}{2}}$,

§ *Ibid.*

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namely 0·142 cusec. The level in the tank was accordingly adjusted until that discharge was obtained. It was found necessary to round the entrance slightly to reduce the diagonal standing waves on the shooting flow in the first section of the flume. The depths were then measured at a number of points. They were shown in *Fig. 11* reduced to full scale, along with the calculated depths as in *Fig. 7* (p. 300 §). It would be seen that in the model flume the main standing wave was followed by a series of smaller waves, but that otherwise the agreement was as good as could be expected considering the somewhat rough agreement between the experimental conditions and those assumed in the calculations. The flume used was of planed wood painted with a bitumastic paint, and it had not necessarily the same coefficient of friction as that assumed by the Author.

The Author, in reply, stated that, with regard to the term "the critical depth," it appeared to him that Mr. Essex had confused it with "the critical velocity," which latter term was used to denote that velocity at which a flow ceased to be a streamline flow, or sometimes, in cases of river-flow, that velocity at which sediment commenced to move on the river-bed. By studying the definition of the critical flow given on p. 289 § it would be seen clearly that no friction-formula had been used in compiling the Tables from which the curves in *Fig. 2* (p. 308 §) were plotted.

The subject of the choice of a friction-formula was always a controversial one, and should be left with the engineer who had a knowledge of the particular features of the channel in question. The Author did not agree that a formula should be condemned for the sole reason that it was tedious work comparing it with the value of C in the Chezy formula. He had found, however, in the calculations for non-uniform flow—which were nearly all of short length—that the type of friction-formula used made a very small difference to the final calculated depths.

Mr. Essex cast some doubt on the advantage to be gained by the calculation of non-uniform flows. Such advantage was clearly to assist the designer to combine economy with safety, and to analyse existing works with a view to what would be the results in the event of a flood in excess of any experienced before. Such calculations would also save much time and expense in the building of scale models to ascertain the effects of peculiarities of a design. That was borne out by the results of the experiment on a model recorded by Dr. Thom, in so much as the recorded results were in general agreement with the calculated ones. The coefficient of friction of the model was evidently considerably less than that assumed in the calculations, and that would have the effect of throwing the hydraulic jump slightly upstream as, in fact, was the case. The appearance of the subsidiary waves following the main standing wave emphasized the advantage of following up calculations by small-scale models whenever possible.

§ *Ibid.*

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