building included several basements the load causing settlement was, of course, reduced by the weight of the excavation, and the depth to which borings should be made was correspondingly reduced.

Ordinary wash borings were useless for examining the strata. The borings should be made dry, if possible, and the minimum requirement should be a careful visual examination and handling of the soil brought up in the auger. Where large buildings were concerned, undisturbed soil sampling and laboratory tests on the soil might be necessary in order that the theoretical bearing capacity of the soil or the probable settlement of the structure might be calculated.

*** No reply had been received from the Author at the time of going to press.—Sec. Inst. C.E.

CORRESPONDENCE

ON PAPER PUBLISHED IN JANUARY 1941 JOURNAL

Paper No. 5236.

"Laboratory Experiments on Bellmouth Spillways." †

By Alfred Maurice Binnie, M.A., Assoc. M. Inst. C.E., and Reginald Kenneth Wright, B.A., B.Sc.

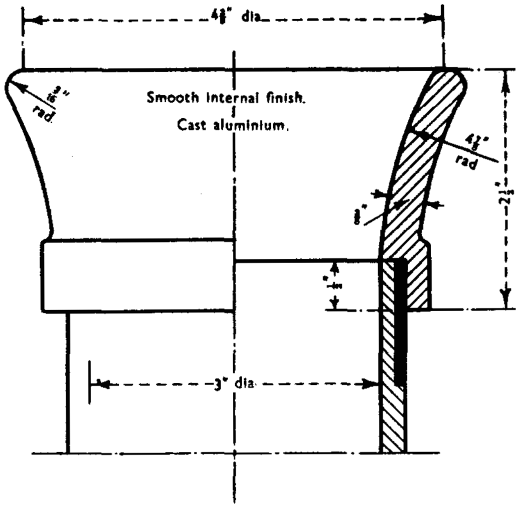
Correspondence.

Mr. A. C. Buck observed that some years ago he had had occasion to undertake a few experiments on bellmouth outlets, differing in two essential respects from those described in the Paper. Firstly, the ratio between the crest diameter and the diameter of the tail-piece was much smaller; secondly, the tail-piece was only 5 inches long, so that the discharge was governed almost entirely by the shape of the inlet.

Three forms of outlets were tested. The first consisted of the entrance to the discharge pipe with the inner edge bevelled; the other two were bellmouths, as illustrated in *Figs. 16 a* and *b*. A change in the law governing the flow occurred in all three types as the head was increased. At first the discharge was approximately proportional to $H^{\frac{3}{2}}$, tending

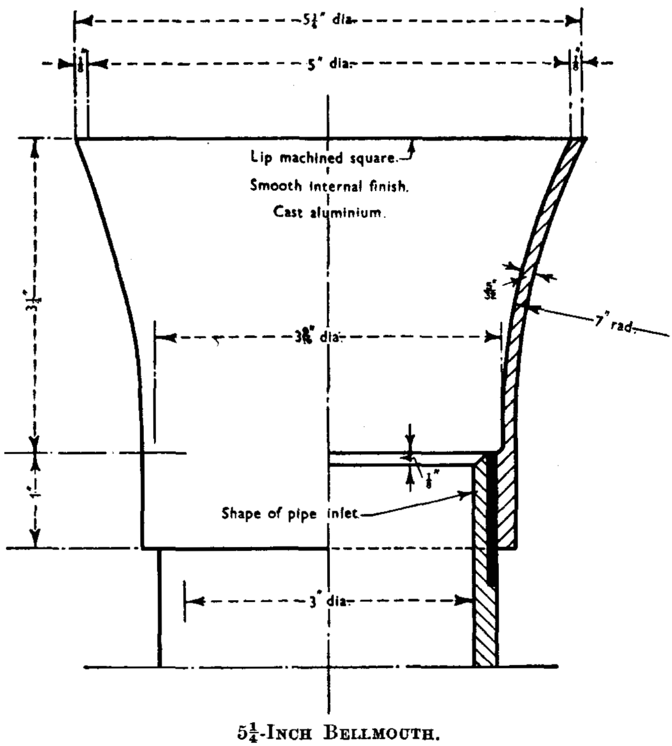
† Journal Inst. C.E., vol. 15 (1940–41), p. 197 (January 1941).

Fig. 16a.



4⅜-INCH BELLMOUTH.

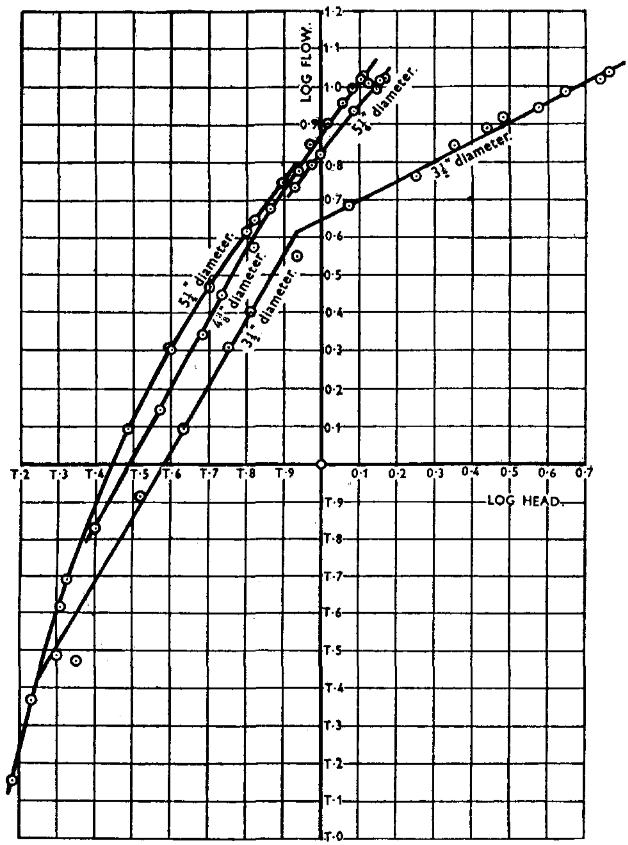
Fig. 16b.



5¼-INCH BELLMOUTH.

finally, so far as it was possible to tell, within the limits of the test, to a value where the index was less than 1. It was possible, and it was suggested by the test on the $3\frac{1}{2}$-inch outlet, that that value might ultimately become $\frac{1}{2}$. In each case the interval of transition was unstable and covered a fairly wide range of heads during which the outlets could

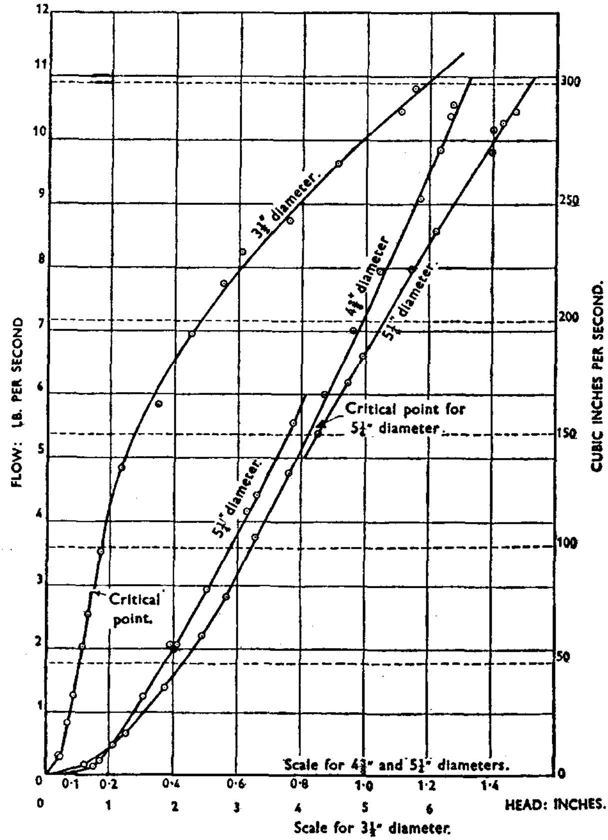
Fig. 17.



by no means be used as an instrument for measuring accurately the flow. The points of transition in the cases of the $3\frac{1}{2}$-inch and $5\frac{1}{4}$-inch outlets occurred suddenly, but not necessarily always at the same head when the vein, which at the lower heads adhered to the inner surfaces of the outlets, detached itself and formed a jet which fell clear of the pipe. That was not observed to happen with the $4\frac{3}{8}$-inch bellmouth up to the maximum head tested, but for heads in excess of about 0·85 inch, at which point a fairly decided change of slope in the log.-flow—log.-head curve occurred, the discharge was unstable. It was interesting to note that the points of transition for each outlet, as judged from the curves shown in *Fig. 17*, (in which log. flow was expressed in lb. per second and log. head in inches) were most marked at about the same head, namely 0·85 inch.

At times the flow was very noisy. The discharge was silent at first until the transition-point was approached, when a sucking noise com-

Fig. 18.



menced, which gradually became louder and louder with increase of head. Above that point the flow became much quieter, and eventually returned to its initial silence.

The appearance of the discharge also varied. The surface of the vein was perfectly smooth at first, but an increase in the flow tended to corrugate it, and the corrugations became most pronounced just before the transition-point was reached, after which vortexes appeared and continued until the head was increased considerably.

Some comparison between the two sets of experiments was possible. As the length and diameter of the tail-piece did not appear to influence the flow until the critical head was reached, the head-discharge curve was a characteristic of the bellmouth itself, and therefore it was possible to compare the performances of different shapes when the crest diameters were the same, or even, with some degree of accuracy, when the shapes were geometrically similar.

In spite of its greater diameter, the discharge from the $5\frac{1}{4}$-inch bellmouth was inferior to that from bellmouth A. At a head of 0·59 inch the discharges were 96 cubic inches and 116·8 cubic inches per second respectively. Bellmouth B was inferior to both, with 75 cubic inches per second (*Fig. 18*).

Should the resistance of the tail-piece be reduced it would appear that a time would arrive when the Q–H curve would possess two critical points, one when the orifice type of flow came into operation, and the other when the friction of the tail-piece predominated.

Mr. James Williamson wished to comment on three points on the Paper, namely, the form of a vertical-shaft bellmouth, the pipe resistance, and the form of weir crest.

Figs. 19 (*a*) represented a bellmouth with vertical and horizontal pipe flowing fully charged. The velocity V_0 was determined essentially by the resistance in the pipes and the bend between the throat A and the outlet O, the resistance in the bellmouth itself being relatively small. *Figs. 19* (*b*) illustrated the condition as to height which should obtain in a suitably-contracted spillway bellmouth, the height H being equal to, or preferably a little more than, $\frac{V_0^2}{2g}$. With that arrangement, the water would not converge into a united stream at the throat A until the fully-charged capacity of the pipe system were reached. The reason that bellmouth A in conjunction with the 20-foot horizontal pipe was satisfactory (p. 208 §) was that that condition was met. When bellmouth A was used in conjunction with a vertical outlet pipe and no horizontal pipe the conditions were not good, because the depth of the bellmouth was insufficient for the increased velocity. It was not difficult to deduce that, with a vertical outlet, the bellmouth should taper all the way to the discharge-point.

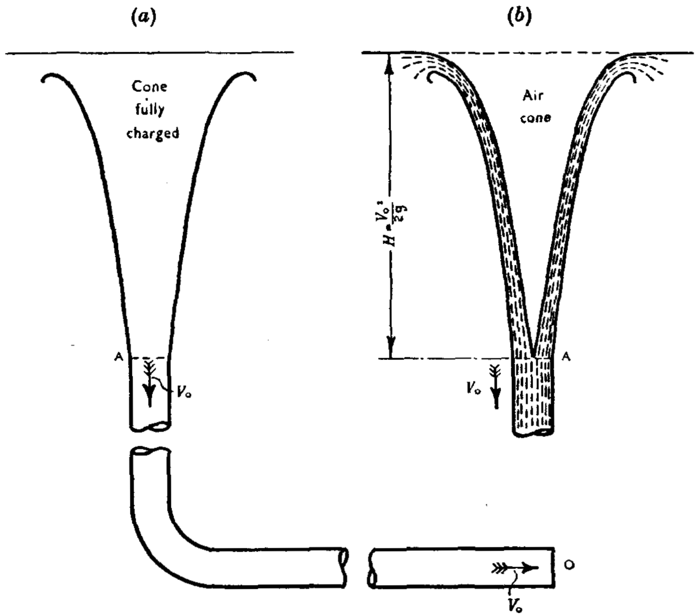
At the throat A (*Fig. 19* (*a*)) the pressure-head, in feet, was $H - \frac{V_0^2}{2g} = 0$, since $H = \frac{V_0^2}{2g}$; that was to say, the water at A was at atmospheric pressure, as at the corresponding point in *Figs. 19* (*b*). Therefore conditions in the two pipes below the throat were identical in respect of pressure and loss of head, and the discharges were equal. The gradual filling of the air-cone in *Figs. 19* (*b*) by a rising reservoir would

§ Page numbers so marked refer to the Paper (Journal Inst. C.E., vol. 15 (1940–41), p. 197 (January 1941).—Sec. Inst. C.E.

therefore have a negligible effect upon the quantity discharged. The practical capacity of the pipe system was reached when the condition of *Figs.* 19 (*b*) was attained.

Mr. Williamson considered that bellmouth B had not been made of sufficient depth to suit the maximum discharging capacity for any of the pipe arrangements with which it was used. As the free-fall height was too small, the throat became temporarily blocked at flows of less than the fully-charged capacity. A column of water united in the pipe at the throat, producing an accelerating head which caused the column to fall away and release the block. Air was thereby drawn in, causing reversion

Figs. 19.



to free-fall in the bellmouth, and the water joined up again into another temporary block. That "make-and-break" action continued until the flow became sufficient to fill the bellmouth and charge the pipe system.

If the model-apparatus were still available, two further tests relative to the height of the bellmouth would be useful, namely, (*a*) with bellmouth A and the horizontal pipe shortened to 10 feet; (*b*) with a new bellmouth of the "A" type, but 13 inches deep, in conjunction with the shortened horizontal pipe. The theoretical considerations propounded above would indicate that in (*a*) the conditions would not be satisfactory, whereas in (*b*) they should be about right, the velocity V_0 being estimated at about 8·1 feet per second, for which $\frac{V_0^2}{2g} = 12$ inches, the overall head being maintained at about 60 inches.

The Authors had pointed out that with a bellmouth of proper height the pressure at the throat would be atmospheric. With any height less than that, the pressure at the throat would be below atmospheric when the system was fully charged. That was shown in *Figs. 13 and 14* (pp. 214 and 215 §), the extreme example being *Fig. 14* for bellmouth B. In that case a negative head of 2·3 feet was measured at $2\frac{1}{2}$ inches below the crest. If the model were copied to twelve times the scale, that was with a 5-foot diameter crest, a 12-inch pipe, and an overall depth of 60 feet, the negative head at the throat would be about $12 \times 2{\cdot}3 = 27{\cdot}6$ feet, which was sufficiently close to complete vacuum to be troublesome.

Calculations of the friction coefficient f for the 1-inch pipe by the Weisbach formula $f = \frac{S \times 2gd}{V^2}$, where S denotes the hydraulic gradient, yielded a value of $f = 0{\cdot}0209$ (mean) for velocities of between 12·2 and 12·4 feet per second, and of $f = 0{\cdot}0234$ for velocities between 6·3 and 6·4 feet per second. The corresponding coefficients on the Author's basis (using $m = \frac{d}{4}$ instead of d in the formula), were 0·0052 and 0·0058. The coefficients, and the variation with velocity, indicated a very smooth surface and that the flow was in the difficult transition stage of partial turbulence. The value of Manning's n for the higher velocities was about 0·0066, and it was probable that the ultimate constant value for the fully-turbulent region ($S \propto V^2$), which would be reached at much higher velocities, would not exceed 0·0064. Almost any large-scale reservoir outlet system would fall within the rough-law region $S \propto V^2$, whereas the exponential law for the short range of small-pipe velocities from 6 to 12 feet per second was $S \propto V^{1.82}$. It was therefore very difficult to make an accurate comparison between a small-scale model of the very smooth class and a large-scale apparatus, particularly if the pipe resistance were a material factor, as in the case discussed. If, in the small model, the pipe-surface were made sufficiently rough to bring the flow into the V^2-law region, comparison with full size would be easy by means of the Manning formula, provided that the value of the coefficient n were known, or could be determined for both surfaces. With roughness $K = 0{\cdot}025''$ ($n = 0{\cdot}011$) the flow in the small pipe would have been practically within the V^2-law region. In the latter region, the Reynolds number was no criterion, and could be left out of account.

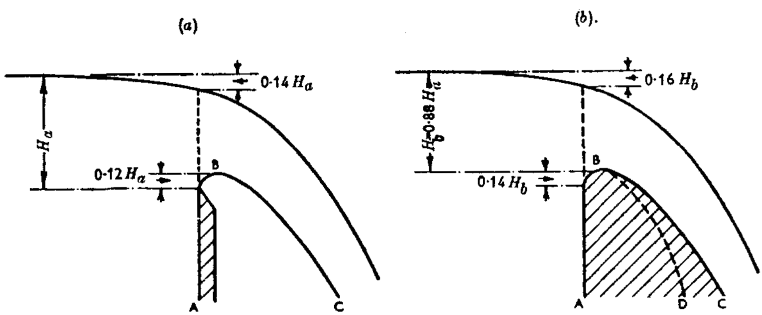
The basis for the most efficient form of round-top straight weir would be studied best from the form taken by water flowing over a sharp-edge weir. *Figs. 20* (*a*) showed the approximate form of the flow over such a weir, and *Figs. 20* (*b*) showed, for the same discharge, the ideal form for the round-crest weir in which the shape ABC fitted the under-water curve produced by a sharp-crested weir. If the forms were duplicated, approximately the same quantity would pass over (*b*) as over (*a*), but the height H_b would be 0·88 of the height H_a. The capacity of (*b*) in relation to the height over the crest would, therefore, be greater than that of (*a*). The ratio of the coefficients in the weir formula for the optimum condition could be expressed by

$$C_a \times 1^{\frac{3}{2}} = C_b \times 0{\cdot}88^{\frac{3}{2}},$$

whence
$$\frac{C_b}{C_a} = 1{\cdot}20.$$

That indicated a possible excess of 20 per cent. over the discharge for a

Figs. 20.



sharp weir, and the discharge formula would become $Q = 4{\cdot}0H^{\frac{3}{2}}$ for the optimum condition. A value of 4·0 for the coefficient had been realized in practice with round-crest dam spillways. The actual form of a round-crest spillway should be proportioned for the maximum depth of discharge, and not for any smaller depth. The curve required at the upstream side of the crest was not large, being less than 12 inches radius for a 6-foot maximum depth over the crest. The continuation of the crest-curve on the downstream side should take a free-fall form in scalar relationship to the maximum depth, and would be approximately parabolic. At maximum discharge, the water stream would then follow the contact-surface without exerting pressure or suction, and for smaller discharges the pressure would be positive. If the downstream surface were steepened or curved more sharply, as indicated by the dotted line BD in *Figs. 20* (*b*), the water would, within limits, still cling to the surface, but with less than atmospheric pressure, and a quasi-siphonic action would occur, producing increase of the discharge; that was, under such conditions the value of the coefficient in the round-crest formula could be higher than 4·0. Evidently that effect occurred in very accentuated form at the narrow rounded crest of bellmouth A, as was indicated by the fact that the coefficient reached the very high value of 4·7 (Table I, p. 203). In such a case, where atmospheric pressure of constant but limited amount was a factor in determining the flow, in conjunction with water heads which varied with the discharge and the scale of the apparatus, direct comparison between model and full size could not be made, although useful deductions might be based upon pressure-readings. The pressure-readings required for a bellmouth of type A, with crest too sharply curved on the downstream side, would be a series taken at the contact surface over a portion extending for some distance downwards from the crest. Further experiments would be useful if they were directed to finding the extent to which a bellmouth could be steepened in the upper part, in comparison with the ideal form, without causing separation of the stream from the surface, or producing detrimentally low pressure. The crest form of bellmouth A would probably give rise to adverse conditions if applied on a large scale.

Mr. Williamson observed that his remarks had particular reference to straight round-crest weirs. The most appropriate weir-form for a circular bellmouth could be determined by making model-experiments with a circular sharp-edged weir at a depth of flow proportionate to the maximum contemplated for the full size and measuring the under-water curve and the discharging capacity. The under-water curve would be a suitable form for a rounded weir of about equal capacity.

The Authors, in reply, welcomed Mr. Buck's account of his experiments with proportionately small bellmouths attached to a short tailpiece. That work was of value in showing how far the discharge through a uniform pipe could be increased by means of a simple and easily made improvement to the entrance. They also thanked Mr. Williamson for his interesting and illuminating remarks. They agreed that the depth of bellmouth B was insufficient. The cost of constructing a bellmouth was usually very considerable, and it was rarely (if ever) feasible to use the ideal deep form. The purpose of the experiments on the shallower but more practicable bellmouth B was therefore to compare its action with that of the much deeper bellmouth A. The surfaces of both were made smooth so that geometrical similarity with bellmouths of, say, diameter 150 ft., was more nearly achieved. In that connexion it might be recalled that the classical experiments of Sir Thomas Stanton and Mr. J. R. Pannell[1] were performed on brass pipes having different diameters but the same surface finish. When the measured friction coefficients were plotted on the same base of Reynolds numbers, it was found that the points relating to velocities above the critical lay not in a line but in a

[1] *Loc cit., ante.*

narrow band. That scatter was attributed to the fact that the requirement of complete geometrical similarity was not observed.

The Authors welcomed Mr. Williamson's suggestions of additional experiments. But, although the apparatus was still in existence, further work with it would have to be postponed until happier times.

CORRESPONDENCE

ON PAPERS PUBLISHED IN FEBRUARY 1941 JOURNAL

Paper No. 5251.

"The Mohammad Aly Barrages, Egypt." †

By Alec George Vaughan-Lee, M. Inst. C.E.

Correspondence.

Professor A. H. Naylor was interested in the discussion on the measurement of the hydraulic gradient. He believed that the only useful method was to sketch in or determine by model-experiment the flow network, a suitable depth of equivalent homogeneous porous ground being indicated from borings. Piping could commence only at the downstream surface, and for that to occur with a horizontal bed the local hydraulic gradient had to overcome the weight of the sand, that was, it had to be about 1 in 1 in excess of the gravity gradient. That local gradient became very great at a sharp corner, irrespective of its overall average value, and the gradient at the surface depended largely upon the vertical depth of the downstream toe. Sheet piling placed there would be much more efficacious in preventing piping than when placed anywhere else. In the centre of the base it would have almost no effect. No simple rule appeared to be safe.

He could not see how, as was suggested in the discussion, an increased perviousness at the interface of masonry and sand, or steel and sand, could affect the general flow lines, for the cross-section of the interface film was extremely small in comparison with the thickness of the pervious

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† Journal Inst. C.E., vol. 15 (1940–41), p. 237 (February 1941).