Discussion

Mr Townend, introducing the Paper, said that the Authors as humble members of the Public Health Engineering Division had been encouraged by the invitation of the Board of the Hydraulics Division to give an account of the present state of hydraulic thought in sewage practice, and to set out some examples of problems which still remained for elucidation and research. It had been difficult to cover such a wide field without sacrificing detail, with the rather disappointing result that so many points had had to be dealt with rather sketchily and in a fairly elementary manner.

Some of the questions mentioned in the Paper in themselves presented quite wide fields of study on which there had been a great deal of work and discussion over many years. For example, the problem of storm-water run-off had only recently been reviewed [6] at a meeting of the Public Health Engineering Division and the subject obviously remained one of vigorous controversy. In the present Paper, the question had had to be dismissed in less than one page.

Again, much work had been devoted to the subject of sedimentation over a long period of years, particularly from the theoretical point of view. Yet the problem remained one where there was an urgent need for still more work to be done in the field of practical application.

In each section of the present Paper, the Authors had tried to give a summary of the current position and to make suggestions for further advance and improvements. In spite of the need, it was widely felt that very little progress was being made in present circumstances. Whilst drainage authorities in general could help to some extent, their problems were likely to be local in character and their resources were always limited. Research on problems of a more general and national character undoubtedly could be carried out most efficiently by research bodies properly organized and equipped for such a purpose and employing specialist personnel who could devote the whole of their time to that particular purpose. That kind of concentrated effort was very rarely possible with the ordinary authority mainly pre-occupied by routine work and for that reason the recent work carried out by the Hydraulics Research Board and at certain Universities was extremely welcome. The Authors looked forward to an extension of those activities. In that connexion, they wished to assure Sir Claude Inglis that the position of his Board in being handicapped by the lack of adequate financial resources was fully appreciated, and it was hoped that perhaps the list of the further work required in the sewage field might give an additional reason for securing more liberal funds.

The national expenditure on capital works for sewerage and sewage

[6] C. D. C. Braine, "The effect of storage on sewerage design." Proc. Instn Civ. Engrs, Pt III, vol. 4, p. 446 (Aug. 1955).

disposal must now amount to upwards of £30 million a year, and even if only one-thousandth of that figure were set aside, there would become available a fund of £30,000 a year which would cover quite an extensive amount of work. There was no question that such an expenditure would yield a very handsome dividend by way of improved efficiency and economy.

In carrying out research work of that kind, the Authors had stressed that above all it should be directed to the final objective of practical application, in co-operation with authorities responsible for sewage operations. They had mentioned the work of the Water Pollution Research Board in furthering the advance of sewage purification over the previous 30 years as an outstanding example of what could be achieved in that way. Much of their work had been carried out at local field stations, such as Birmingham and Coventry, which had been set up at existing sewage works in close collaboration with the staff of the authorities concerned ; in that way, very much better results had been achieved than by either side working on its own. The Birmingham station had in fact been in continuous operation for 17 years.

Mr Wilkinson, in introduction, said that he wished to refer again to the use of step-by-step methods for non-uniform flow in the investigation of flow over side weirs. Figs 10 and 11 illustrated the results obtained in that way, for both single and double weirs. In each case the channel was 2 ft wide and of rectangular cross-section ; the gradient was 1 in 184, and it was desired to reduce the flow from 20 cusecs upstream to 5 cusecs downstream. The uniform-flow depth for 5 cusecs in the channel was 0·58 ft, which was therefore the height which the weir crest had to be set above the invert, and was also the depth to which the flow had to be reduced at the downstream end.

It was stated in the Paper that Dr Coleman's experiments[7] had shown that the water surface at the upstream end of the weir was drawn down to about the critical depth for the quantity at that point ; in fact, those experiments had indicated that the upstream depth was rather less than the critical value, depending on the ratio of the height of the weir to the critical depth.

In the given examples, as in Dr Coleman's experiments, shooting flow occurred along the full length of the weir, and accordingly the upstream end of the channel might be regarded as a control section, from which the calculations proceeded in a downstream direction.

Utilizing the energy equation, then for any length δl (feet) along the weir :

$$\delta l = \frac{\delta H}{S_f - S_o} = \frac{\delta Q}{C h_m{}^{3/2}} \text{ for the single weir, and}$$

[7] G. S. Coleman and Dempster Smith, "Discharging Capacity of Side Weirs." Selected Engng Paper No. 6. Instn Civ. Engrs, 1923.

$$\delta l = \frac{\delta H}{S_f - S_o} = \frac{\delta Q}{2Ch_m{}^{3/2}} \text{ for the double weir.}$$

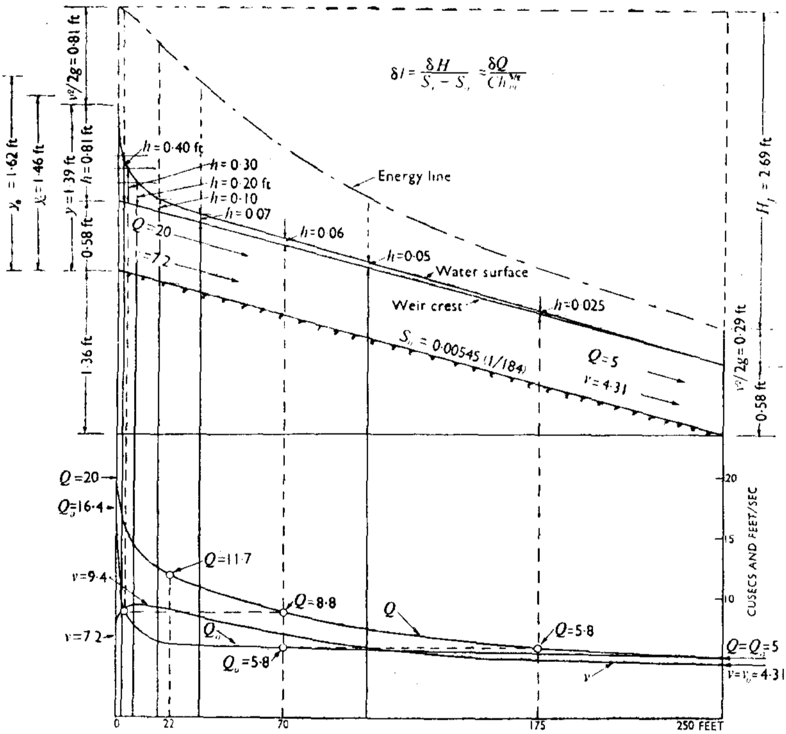

FIG. 10.—SINGLE WEIR. 2-FT-WIDE RECTANGULAR CHANNEL

where

δH denoted the difference in height of the energy line above the invert (feet).

S_f ,, the mean slope of the energy line.

S_o ,, the slope of the channel.

δQ ,, the flow over the weir (cusecs).

C ,, the coefficient of discharge for the weir.

h_m ,, the mean head on the weir crest (feet).

Fig. 10 showed the results obtained in the case of the single weir. It would be seen that the water surface fell very steeply at first, but flattened out downstream along the weir. In making the calculations, the portion of the water surface from $h = 0{\cdot}06$ ft to $h =$ zero had been assumed to be a straight line, and on that assumption the total estimated length of weir was about 250 ft.

The curve (Q) showed the rate of flow in the channel alongside the weir, and as might have been expected, the rate of reduction of flow was rapid at first, but gradually decreased as the head decreased towards the downstream end. The curve (V) showed the variation in velocity in the channel alongside the weir; it increased at first, from 7·2 to 9·4 ft/sec, and thereafter gradually decreased towards the downstream end. A similar increase in velocity along the weir had been observed by Dr Coleman in his experiments, but it would appear that his experimental weir, which had had a maximum length of 18 in., had not been long enough to develop sufficient energy loss due to friction to cause a decrease in velocity such as was indicated in the present method of analysis. The velocity increased when the slope of the energy line was less than the slope of the water surface and decreased when it was greater. The curve (Q_o) showed what the rate of flow in the channel would be, for the various depths, under conditions of uniform flow.

Now it was usual in applying the Coleman formula to fix the downstream head on the weir at about 0·06 ft, and to assume that when the water surface had been drawn down to that value the flow in the channel would be reduced to that for uniform flow at the corresponding channel depth. For example, in the present case, the flow in the channel under conditions of uniform flow would be 5·8 cusecs at a depth of 0·64 ft, as shown on the Q_o curve. The curve for Q, however, showed that when h was 0·06 ft the flow had only been reduced to 8·8 cusecs, at a distance of about 70 ft along the weir. To reduce the flow to 5·8 cusecs would require a weir length of about 175 ft. In each case the water surface would rise in the channel downstream of the weir, to the corresponding depth for uniform flow.

The length according to the Coleman formula, to reduce the head to 0·06 ft was about 22 ft. According to the present analysis, at 22 ft the flow in the channel would only be reduced to about 11·7 cusecs and the head to 0·09 ft.

Fig. 11 dealt with the same conditions except that two weirs were used, one on each side of the channel In that case, at the point in the channel where the head had been reduced to 0·06 ft, the flow in the channel had only been reduced to 11·8 cusecs at a distance of 17 ft along the weir, as compared with the 5·8 cusecs required. To reduce the flow to 5·8 cusecs required a weir length of about 140 ft, whilst the length required to reduce the flow to 5 cusecs, and the head to zero, was about 230 ft, assuming the water surface to be a straight line from $h = 0·03$ ft to $h =$ zero. The Coleman formula gave a length of about 11 ft for a downstream head of 0·06 ft by halving the length of single weir, at which point the present analysis showed the flow to be reduced to only 12·5 cusecs. It might now be remarked that it was nowhere suggested in Dr Coleman's Paper that the downstream head in his formula was related to the uniform flow rate

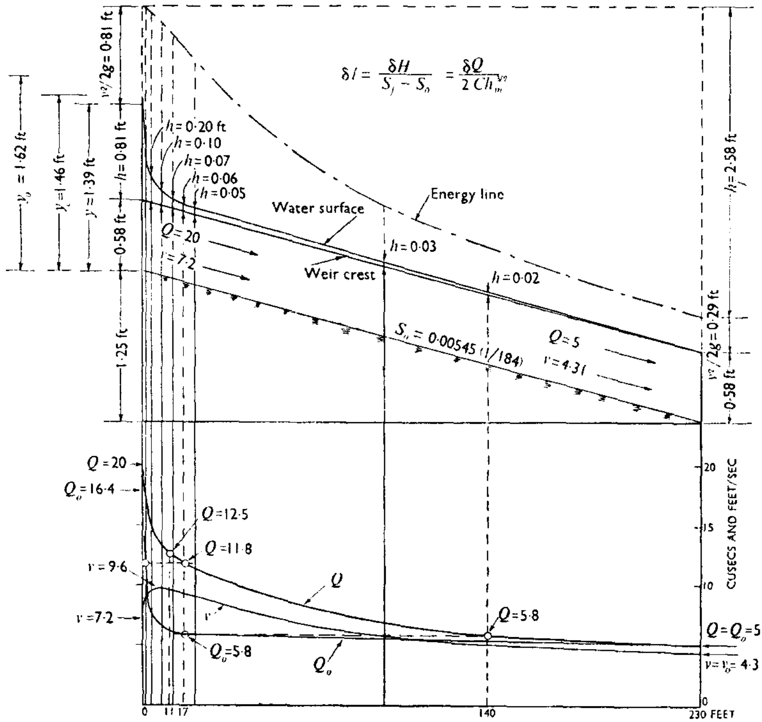

FIG. 11.—DOUBLE WEIR. 2-FT-WIDE RECTANGULAR CHANNEL

at the corresponding depth in the channel; in fact, in the one example of the use of his formula given in that Paper, no mention had been made of either the upstream or downstream rate of flow.

In summarizing the results obtained by the above-mentioned method of analysis, the following conclusions appeared to be justified:

(1) Where the height of the weir crest was less than the upstream critical depth, and there was no backing up, the upstream depth was closely equal to the critical depth.

(2) The flows in the channel at intermediate points along the weir were greater than for uniform flow at the same depths. The usual method of applying the Coleman formula, therefore, appeared to be unsound.

(3) Except for very small reductions in flow, the lengths of weir required were likely to be too great for practical application. That pointed to the necessity for some form of throttling device in the downstream channel so as to increase the head on the weir and the rate of overflow.

Hydraulic conditions other than those described in the above-mentioned examples might occur at side weirs, depending on the upstream and downstream conditions, height of weir crest, and channel gradient. The depth of flow might increase continuously in the downstream direction alongside the weir, or might decrease at first and then increase, either gradually or through a hydraulic jump.

Mention was made in the Paper of the significance of the "states of flow" in sewerage design. Fig. 12 illustrated an instance where recognition of the distinction between the tranquil and shooting states was of practical importance in that respect; it showed a longitudinal section

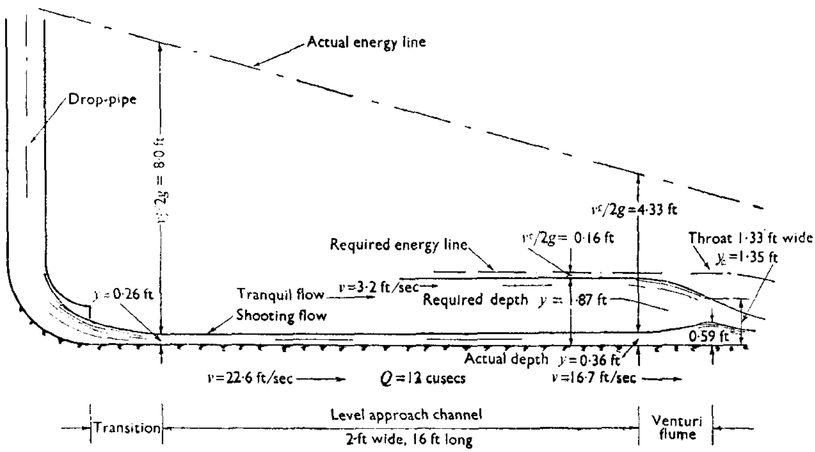

FIG. 12.—TRANQUIL AND SHOOTING FLOW IN A VENTURI FLUME

through the approach channel of a Venturi flume installed for the purpose of measuring sewage flow.

The discharge of a Venturi flume varied as $H^{3/2}$, H being the total energy head (depth plus velocity head) at the entrance, and for the most favourable operating conditions and accuracy it was necessary that there should be tranquil flow in the approach channel, with the ratio of depth to total energy head as near unity as practicable. In that example the flume had a throat width of 1·33 ft requiring an upstream water depth of 1·87 ft for a discharge of 12 cusecs. The velocity head was 0·16 ft and the ratio of depth to total energy head was 0·92. The flow, however, was discharged into the 2-ft-wide approach channel through a drop-pipe terminating in a 90 degree bend, at a velocity of about 23 ft/sec and a depth of about 0·26 ft. Owing to friction loss along the channel the depth at the flume entrance was about 0·36 ft and the velocity about 17 ft/sec, the flow was shooting with a velocity head of 4·33 ft, and the ratio of depth to total energy head was only about 0·08 ; under those conditions the installation could not fulfil its intended purpose. In that case, the provision of a water cushion at the foot of the drop-pipe would have destroyed the excess kinetic energy and permitted the flow to assume the proper depth and velocity for operation of the flume.

Similar conditions could arise where the gradient of the upstream sewer was critical or steeper, in which case a long and relatively flat approach channel was required.

Mr E. H. Ely (Senior Assistant, Messrs J. D. & D. M. Watson, Consulting Engineers, London) referred to the question of multi-stage drop-pipes and said that he had seen them in operation under heavy flows and working very satisfactorily.

On the question of the relation of pipe-diameter to length in distributing the flow between various units in sewage works, it was unfortunately often impracticable to have a perfectly symmetrical arrangement of pipes. Whereas there might be an ideal relation between D and L, use had to be made of the commercial sizes of pipe available and they frequently did not permit the ideal to be realized. Hydraulic calculations were as yet still somewhat speculative, and he made a strong plea for the symmetrical pipe arrangement of repetitive equal bifurcation in which the water reaching any particular tank passed through exactly the same conditions as any other water in the system. In that case, no reliance at all had to be placed upon calculation so far as uniform distribution was concerned.

He then described a method of solving trial-and-error calculations, so frequently encountered in hydraulic problems, by graphical analysis which required no advanced mathematics. He took as a simple example the calculation of the division of flow in the case of a storm-water overflow weir controlled by a flume. The flume was so designed that the weir commenced to discharge when 6 times the dry-weather flow was coming down the sewer. However, when the sewer was discharging Q, which was considerably more than 6 D.W.F., the depth of flow over the weir which was discharging Q_1 created a corresponding additional head on the flume which caused it to discharge 6 D.W.F. plus or Q_2. By hypothesis, $Q_1 + Q_2 = Q$ but, in a given trial-and-error calculation, $Q_1 + Q_2$ would generally be greater or smaller than Q.

The graphical method consisted of plotting the plus or minus error as ordinates against some convenient variable in the calculation (such as the depth of flow over the weir) as abscissae. Initially, three calculations should be made and the three errors plotted. Where the curve which passed through those points cut the vertical axis, the error was zero and the corresponding head over the weir could be read off. By substitution, the correct value for Q_1 could be obtained and thence Q_2, by deduction from Q. By a suitable choice of scales, the error curve could be virtually a straight line. It was, of course, desirable that the errors be kept relatively small.

If some simple feature of the design were altered, such as the length of the weir, and the graphical analysis repeated, the second error curve would be parallel to the first. In most of such cases, it would then be sufficient to plot the error from only one calculation and draw the error curve through that point parallel to the first curve, the new solution being read off as before. By that means, the effect of a number of different weir lengths could be quickly compared or, perhaps, the behaviour of the overflow under different conditions of sewer discharge could easily be studied.

Dr C. F. Colebrook (Senior Engineer, Messrs Binnie, Deacon, & Gourley, Consulting Engineers, London) observered that the hydraulic conditions in the vicinity of a side-weir overflow in a sewer were extremely complex and varied considerably according to the type of flow, tranquil or shooting, upstream of the weir. He considered, therefore, that the ordinary side weir was a very unsatisfactory hydraulic structure and should, if possible, be avoided.

A much more efficient type of overflow had been designed and several examples were now operating successfully. In that design the overflow weir was at a level considerably above the sewer invert level, and the hydraulic gradient on the surcharged sewer downstream of the overflow chamber accurately controlled the amount of sewage passing on and ensured that the surplus storm-water passed over the weir uniformly from end to end in a normal manner. To prevent excessive turbulence in the chamber, it was desirable to reduce the velocity head of the incoming flow to about one-quarter of that in the sewer; that could be effected by a gradual divergence of the soffit and invert of the sewer for a short length upstream of the chamber so as to increase the area to about twice the area of the sewer. That also effected a partial regain of velocity head.

With regard to velocity formulae for flow in pipes, the Authors had referred to a considerable number of formulae, many of which were of the exponential type. Most engineers had a preference for one formula and the Manning formula $v = \frac{1{\cdot}486}{n} m^{0{\cdot}67} i^{0{\cdot}5}$ was probably the most widely used for the design of sewers. Many engineers had design Tables based on the Manning formula and Dr Colebrook proposed to examine it critically in the light of the more recent experimental and theoretical work of Nikuradse, Prandtl, and von Karman in Germany and Professor White and himself in England.

The experimental work of Nikuradse on hydraulically smooth pipes had shown that the coefficient f in $h = \frac{flv^2}{2gd}$ was a function of the Reynolds number, $R = \frac{vd}{\nu}$ only and in Fig. 13 that function was represented by a single curve. Prandtl and von Karman had made a theoretical analysis of the problem of flow in smooth pipes and had developed the formula $\frac{1}{\sqrt{f}} = 2 \log \frac{R\sqrt{f}}{2{\cdot}51}$, which fitted that curve perfectly.

Further experimental work by Nikuradse on artificially roughened pipes, in which grains of sand of uniform size had been stuck to the inside surfaces of a number of pipes of various diameters, had confirmed that when turbulence was fully developed in rough pipes the coefficient f, for any given pipe, was independent of the Reynolds number and was a function only of the relative roughness d/k where k denoted the sand-grain diameter and d the diameter of the pipe. Prandtl and von Karman had developed the formula for fully developed turbulence $\frac{1}{\sqrt{f}} = 2 \log 3{\cdot}7 \frac{d}{k}$ and that was represented in Fig. 13 by a number of horizontal parallel lines.

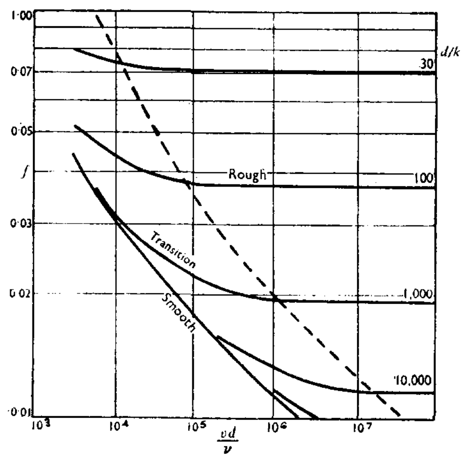

FIG. 13.—VARIATION OF f WITH $\frac{vd}{\nu}$ FOR VARIOUS VALUES OF THE RELATIVE ROUGHNESS d/k

There was a wide transition zone between the smooth-pipe type of turbulence and the fully developed rough-pipe type of turbulence, and the transition formula developed by Dr Colebrook in collaboration with Professor C. M. White had been found to agree closely in that zone with the experimental data on a large number of new commercial pipes. The formula was $\frac{1}{\sqrt{f}} = -2 \log\left(\frac{k}{3{\cdot}7d} + \frac{2{\cdot}51}{R\sqrt{f}}\right)$ and could be represented in Fig. 13 by a family of curves asymptotic to the " smooth " and " rough " pipe curves.

Sewers were usually designed by using the Manning formula with values of n of 0·013 (for vitreous pipes) and 0·015 (for concrete and brick) and the flow in such sewers was considered to obey the velocity-squared law. Dr Colebrook proposed, therefore, to compare the Prandtl-von Karman rough-pipe formula with the Manning formula. The rough-pipe formula $\frac{1}{\sqrt{f}} = 2 \log 3{\cdot}7 \frac{d}{k}$ could be converted into an exponential form which, for a range of $d/k = 10$ to 2,000, gave values of the velocity v to within $\pm 5\%$ of those calculated using the more accurate rough-pipe formula. The derived formula was :

$$v = \frac{1{\cdot}486}{k^{0\cdot17}/30{\cdot}8} m^{0\cdot67} i^{0\cdot5}$$

Thus, Manning's $n \approx \frac{k^{0\cdot17}}{30{\cdot}8}$ where k was measured in feet.

The equivalent sand roughnesses k for various values of n were as followed :—

n	0·011	0·013	0·015
$k \approx$	0·02″	0·055″	0·13″

and the minimum velocities, in ft/sec, for fully developed turbulence in water at 55°F were :

Pipe diameter	$k = 0{\cdot}02''$	$k = 0{\cdot}055''$	$k = 0{\cdot}13''$
1′–0″	8	2½	1
12′–0″	10	3½	1½

In sewers having a roughness equal to or greater than 0·055″ (Manning $n \geqq 0{\cdot}013$) the flow was fully developed turbulence, even in the largest sizes, at velocities of about, and greater than, 3 ft/sec, and for all practical purposes the Manning formula was quite satisfactory for the design of such sewers.

Mr H. R. Oakley (Partner, Messrs J. D. & D. M. Watson, Consulting Engineers, London), referring to the comments on p. 663 on the importance of further research, made a plea for more support from the profession for research on such subjects in universities. He thought it was not always realized that university research served a number of functions. It was concerned not only with getting results which were of interest to engineers, but with the training of men in the manner described by the Authors, and producing a steady stream of research-minded engineers who were interested in research and were willing to do some when the opportunity occurred.

Dr Colebrook had already mentioned the relation between the empirical formulae and the fundamental formulae, and Mr Oakley thought that the Report of the Institution's Sub-Committee, published in 1948, had been given due consideration by those using such formulae regularly. He stressed that, in dealing with sewers, it was the roughness k which determined the coefficient to be used in the empirical formula.

It was no use being optimistic about the value of k. Dr Colebrook had given some figures, and Mr Oakley wished to give some more. Considering a 24-in.-dia. sewer flowing full at 3 ft/sec and converting the fundamental formula to more familiar terms—in this case Chezy C—a value k of 0·1 in. corresponded to a Chezy coefficient of 96, and a value of 0·01 in. to a coefficient of 122. In comparison with that, a Crimp and Bruges formula gave $C = 117$ and the particular variation he used of the original Ganguillet and Kutter formula gave a coefficient of 101, which he submitted was the more realistic figure for sewers in a worn condition.

On p. 672 the Authors had dealt briefly with certain questions connected with the design of pipes and channels in sewage-disposal works. They had stressed the difficulties that arose because of the variation of flows that had to be allowed for in dealing with liquid which contained suspended matter. It was well known to most engineers that channels were very much more amenable than full pipes in that respect, in that one could utilize the variation in depth of flow.

Mr Oakley wished to stress that very often the head losses, which were of a small order, were largely determined by the losses at bends, T-junctions, and discontinuities of that sort and not by the pipe-friction losses. That was a subject on which there was too little information. Frequently the absurdity arose of having to try to extrapolate results obtained on a 6-in.-dia. pipe for a 30-in.-dia. pipe or channel. Whilst aware that it was not permissible to do so, the attempt had to be made because the information on the larger sizes of pipes and channels simply was not available. If the Authors and others in their position could publish the actual observed losses at bends, T-junctions, and so forth in the larger sizes of pipes and channels, they would be doing the profession a service.

On p. 674 the Authors had dealt with the velocity of fall of granular particles. He found it a little difficult to follow the statement: "The opposing forces are the frictional resistance of the water and its viscosity." He assumed that the intended interpretation was that the opposing forces were those of viscous drag and turbulent energy losses. He granted that the turbulent energy losses did decay because of viscous shear, and were thus a function of viscosity, but if the intention in that passage was to point out that there were two modes of resistance depending on whether the particle was moving quickly or slowly, he did not think that the expression was quite clear.

On p. 676 the Authors had dealt with grit channels. He thought that the assumption was made very often in the design of channels that the

Hazen theory which had been outlined by the Authors was valid. He questioned whether that was in fact a correct view of the way in which grit channels worked, although no doubt it sufficed for design purposes. He thought it was more correct to look upon grit channels as keeping the lighter particles in suspension by virtue of the turbulent energy interchange in the channel, and, by virtue of the scouring forces along the bottom, picking up any that did settle. One had only to reflect that with a velocity of $2\frac{1}{2}$ ft/sec in a sewer the turbulence was sufficient to keep practically all particles from settling. In a grit chamber the turbulence was reduced deliberately so that one class only of particles settled. That led to the supposition that a possible development in design might be to devise a channel where that turbulence could be controlled and, if necessary, varied to suit the particular conditions in the works, possibly by means of air-diffusion (in a simpler arrangement than the spiral-flow channel), or something of that kind.

The section of the Paper dealing with sedimentation tanks was useful from a qualitative point of view but gave insufficient quantitative data to be of assistance in the design of sedimentation tanks. The reason was, of course, that there had not been enough attention paid to the study of full-scale tanks in operation. He recalled Holroyd's Paper [8] on the operation of a sedimentation tank at Dagenham ; it gave a great deal of very useful tabulated information. He thought that more study of operating tanks on those lines would be of value.

The theoretical considerations were so complex that a theoretical attack on the problem just was not possible ; so one turned inevitably to model experiments, and experiments on a sedimentation tank presented very much the same sort of difficulties as model experiments of rivers and estuaries, in that the governing forces which determined the conditions for similarity, were not necessarily reconcilable. The conditions which suited the comparison of flow, for example, were not compatible with the conditions which suited the comparison of the sedimentation of the smaller particles. He felt that the basic knowledge required for guidance in the design of model tanks of that nature was available but it had not always been wisely applied. There was a considerable literature on model tanks, from which it would be seen that nearly all the model tanks had been operated under streamline flow conditions, whereas he had little doubt that the full size tank of conventional design operated under turbulent flow conditions ; the difference was important. It would be seen, also, that the scale of those models had been too small. The problem was an analogous problem to that of tidal estuary models ; the same depth exaggeration was necessary but even so it would also be found impossible to reconcile all the desirable requirements.

[8] A. Holroyd, "The Operation of the Dagenham Prüss Tank." J. Inst. Sew. Purn, Part I (1937), p. 62.

For those reasons, he thought it was necessary that before the correct conditions for model experiments were established, a model should be operated side by side with the full-scale prototype tank, and the importance of the scale factors evaluated. He hoped that Mr Townend might find it possible to make facilities of that sort available.

Mr P. O. Wolf (Lecturer in Fluid Mechanics and Hydraulic Engineering, Department of Civil Engineering, Imperial College of Science and Technology) observed that engineers responsible for research were aware of many of the fundamental problems which needed further investigation, and the Authors needed to have no fear of lack of interest. A major difficulty at the present time of great activity in design and construction was caused by the shortage of men prepared to spend a few years on research before going into practice, or to devote themselves to research as a career, when attractive employment in other fields was open to the young engineer immediately after graduation. The few engineers in the universities and research institutions were at the same time faced with urgent requests to assist in the solution of the many problems encountered in the design of various works of great importance. He personally found such *ad hoc* research a great stimulus to good teaching and to fundamental research, yet paradoxically, by absorbing time and facilities, it often prevented the successful study of the very problem it had brought to light.

Mr Wolf was concerned with an advanced course of study in engineering hydrology—of which he thought storm-water sewerage formed a part—which would include studies, of the kind which the Authors had advocated, of the run-off from rainstorms both over model catchments and in open or piped drainage systems. The aim of such studies would be the determination of the variables which defined the capacity of a drainage system for the modification of flood peaks.

Whilst agreeing with the Authors on the usefulness of models in the study of drainage systems, Mr Wolf did not share their pessimism which had led them to say that full-scale results were unlikely to be of general application. It seemed to him that the effects of local features would have to be isolated in the course of the search for a full understanding of the interaction of all contributory phenomena. For that reason he would plead for an extension and intensification of full-scale observations.

Turning to a number of details, Mr Wolf questioned the ratios quoted on pp. 666 and 667 of maximum to minimum velocities in circular sewers. Even for uniform flow the values given there were too low, for the maximum velocity occurred when the sewer was about four-fifths full. The discussion on Public Health Paper No. 11[9] had included many references to the high velocities of flow reached during the rise of a flood and to the low velocities

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[9] C. D. C. Braine, "The Effect of Storage on Sewerage Design." Proc. Instn Civ. Engrs, Part III, vol. 4, p. 446 (Aug. 1955).

after the flood peak had passed. Upstream of a junction the water might indeed be stagnant in one branch for the period of a flood in the other.

He had been very interested in the section of the Paper which dealt with side-weir overflows, in Mr Wilkinson's introductory remarks on the same subject, and in Dr Colebrook's condemnation of the device. Dr Colebrook's sketch of an alternative device which would probably be more costly in many cases, namely a stilling chamber with plain overflow weirs and with a downstream control in the form of a drowned pipe, reminded him of the design for the Ashop works which R. W. S. Thompson had described [10] some years earlier. At the Ashop side weir the control of the discharge was also downstream, but it was achieved by a sudden increase in the wetted perimeter owing to contact with a rough flat roof; the head over the side weir was, therefore, quite large over its whole length from the moment when the maximum permissible flow was exceeded. The special problem of the solids in sewage required a careful study of scum-board arrangements, etc., but in general—particularly for such purposes as aqueducts for water-supply or hydro-electric schemes—he thought there would be many cases where side-weir overflows would be economical. He hoped that there would before long be an Institution meeting for the detailed discussion of the recent progress in the theory of flow over side weirs, made both in Great Britain and on the Continent.

Finally he asked the Authors for an explanation of the particle path shown in Fig. 7. Assuming uniform inflow into the sedimentation tank from a vertical pipe in the centre, he had drawn on Fig. 7 a flow-net from the centre to the periphery. The spacing of the stream-lines on that flow-net was not a measure of actual velocities, but it indicated the relative values of the velocities across any vertical section of the tank. The lowest mesh close to the centre was the largest of all, representing a small total velocity and an even smaller horizontal component. It appeared, therefore, that the radial surface velocity was decreasing from the centre outwards, and he saw no justification for the concavity of the path as shown in the diagram. In practice, he presumed, low velocities near the centre of the floor of the tank were a decided advantage, to be added to the list of other advantages given on p. 679.

Mr D. A. Brown (Senior Assistant, Sir Murdoch MacDonald and Partners, Consulting Engineers, London) said that there were more unknowns in hydraulics than in almost any other branch of civil engineering. He was therefore somewhat disappointed to find that the Authors had not seen fit to quote any comparisons between the design assumptions and the actual performance of the hydraulic structures for which they were responsible. That would have enabled them to reach some more definite conclusions.

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[10] R. W. S. Thompson, " The Diversion of the Ashop River." Min. Proc. Instn Civ. Engrs, vol. 229 (1929-30), p. 388.

For instance, with regard to flow in pipes and channels, they had quoted the various formulae which had been used in the past for the design of sewers which were working today, but they did not appear to favour any particular formula. He would suggest that the Crimp and Bruges formula had a limited application : it was only Manning's formula with a fixed value of n. In the case of channels and pipes running only part full, had not Manning's formula now superseded the others ?

He wished particularly to raise the question of side weirs, which had limitations which were not generally appreciated. In designing intakes on mountain streams for hydro-electric works, a device for limiting inflow of water to aqueducts was often required. In such situations discharges considerably in excess of six times the dry-weather flow were encountered, which meant that any regulating device had to work as it was designed.

In a first attempt to design a suitable device, Mr Brown had used the Coleman and Dempster-Smith formula. That equation was not dimensionally correct, and the assumption of a value for h_0 was open to question. He thought that a careful reading of Gibson's remarks in his well-known book [11] suggested that he had not been too confident about it.

A study of the experiments of Favre and Braendle [12] and the earlier experiments of De Marchi [13] and also Gentilini [14] showed that the conditions envisaged by Coleman and Dempster-Smith were a special case, namely, with velocity above critical. Where the flow in the main channel was tranquil, the water-surface profile dropped sharply to the commencement of the side weir, but thereafter rose gradually along the weir. That was exactly opposite to the Coleman and Dempster-Smith results, and it was attributable to the reduction in velocity in the main channel The same effect had also been noted by Merrill and Mackintosh in the discussion on Braine's Paper.[15] De Marchi had further drawn attention to the fact that in his experiments, where the length of side spillweir was great, an appreciable transverse slope developed on the water surface opposite the side spillweir ; he had therefore concluded that his theory was applicable only to short weirs. Gentilini's experiments had been made on weirs of a maximum length of four times the channel width.

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[11] A. H. Gibson, " Hydraulics and its Applications." 5th Edn, Constable, London, 1952.

[12] H. Favre and F. Braendle, " Expériences sur le mouvement permanent de l'eau dans les canaux découverts, avec apport ou prélèvement le long du courant " (" Experiments on the steady motion of water in open channels, with inflow or outflow from the main flow."). *Bull. Tech. Suisse Romande*, vol. 63, p. 93 (10 Apr., 1937) ; p. 109 (24 Apr., 1937) ; p. 129 (22 May, 1937).

[13] G. De Marchi, " Saggio di teoria del funziamento degli stramazzi laterali " (" Report on the theory of lateral discharge (Longitudinal spillways) "). *L'Energia Elettrica*, vol. 11, p. 849 (Nov. 1934).

[14] B. Gentilini, " Richerche Sperimentali Sugli Sfioratori longitudinali " (" Experimental research on side weirs "). *L'Energia Elettrica*, vol. 15, p. 583 (Oct. 1938).

[15] C. D. C. Braine, " Draw-Down and Other Factors relating to the Design of Storm-Water Overflows on Sewers." J. Instn Civ. Engrs, vol. 28, p. 576 (Apr. 1947).

For the works that Mr Brown had in mind, he had concluded that if weirs were designed in accordance with the Italian findings, there were grounds for expecting that they would work in accordance with theory: they were, however, inefficient and he had ultimately decided to abandon attempts to design side spillweirs and to rely instead upon devices to restrict flow.

Mr A. R. Thomas (Consultant) said that he could think of at least three branches of the subject which public health engineers had in common with irrigation engineers. One was the transportation of solids; another was the dissipation of energy; and the third the measurement of flow. There were probably others.

Referring to the question of transport of sediment, why was it that public health engineers, when they wanted to deposit grit in their grit channels, should demand a constant velocity, whereas irrigation engineers, when they wanted to maintain a channel which did not silt or scour, demanded that the velocity should be proportional to the square root of the depth? Did economy enter into the question or was it really necessary to have a constant velocity of 1 ft/sec, as stated on p. 672: "Minimum velocities should be related to the material in suspension. In preliminary plant and channels, before removal of grit, this should be 1·5 ft/sec; grit chambers are usually controlled at 1·0 ft/sec . . ." Surely both problems were fundamentally the same. He suggested that sewerage engineers and irrigation engineers might gain much by consulting each other's literature. That might also apply to the other two questions he had mentioned—the dissipation of energy and the measurement of flow.

Model experiments were invaluable if they could be done; but Mr Thomas suggested that perhaps a great deal could be done by taking observations in existing sewage-disposal works. An instance which came to his mind was the sedimentation tank: by taking observations at various depths the turbulence could be measured, and much information could be gained by comparing results of different types of tanks under very similar conditions. He made that suggestion because, although it seemed obvious, he did not remember seeing any results, outside America, of such measurements and observations in sedimentation tanks.

Mr C. D. C. Braine (Partner, G. B. Kershaw and Kaufman, Consulting Engineers, London) said that hydraulics research equivalent to that being done in sewage disposal by the Pollution Research Board at Birmingham was urgently necessary and he was quite sure that if the Hydraulics Research Station would second a couple of men to say, the Birmingham Tame & Rea District Drainage Board and to Mogden for a few years, their work there would pay for itself many times over. It was surely in the national interest that such research work should be undertaken without delay.

There was plenty of scope for research. For instance, the question that Dr Colebrook had raised in regard to pipes and pipe formulae needed investigation in relation to sewers. If, in a new clean sewer, velocities were low, a certain amount of grit would soon accumulate on the invert so that the top of the pipe might be smooth and the bottom very rough. During periods of low flows, the value of Kutter's n in Manning's formula might be of the order of 0·02. whereas when the silt and the grit were in proper suspension, the value of n might be completely different. What was the value of n in a sewer with grease and fungi sticking to its sides ?

Experiments were usually made in laboratories on pipes which were relatively new, but a sewer was usually only put to the test when it was about 30 years old and when the population it served had fully developed. Research on such sewers was needed to determine to what extent the value of n varied with age. The relative values of n in brick sewers and concrete pipe sewers would probably show considerable changes as the sewers aged.

In that connexion, some years ago he had had occasion to measure very carefully the flows in some big brick conduits. The brickwork, although it might have been 50 years old, had been in excellent condition. Once the conduits had been cleaned of some heavy debris, the coefficient, so far as one could tell, was 0·014. When the West Middlesex scheme had first started up, there had been a test on the effluent outfall ; the brickwork had been of first-class workmanship, and the value of n had been found to be 0·013, which was higher than had been anticipated.

He had often besought Mr Townend to carry out experiments in later years to see whether that value had altered to 0·014 or even worse. That was practical information of a kind which was very badly needed. The concrete pipe sewers of West Middlesex had all been designed on the basis of 0·015, in the belief that the cement would disappear to a certain extent, and the pipes would roughen. It was appreciated that in their early years that coefficient might be much lower. Now, after 15 years of effective life as sewers it would be interesting to know what the actual coefficient had become. If a few weeks' research could prove that the coefficient was still only say 0·013, what considerable economies could be made throughout the country in the future !

It was for those reasons that he supported Mr Townend's view that research staff should be seconded to various selected places where they would have facilities for such work. He was quite sure that a great deal of very important information could be obtained in that way.

With regard to Mr Thomas's remarks, he thought he was right in saying that Lacey [16] had shown many years ago that grit would not stay in suspension when the velocity of flow fell below 0·88 ft/sec. He imagined that that bore a very close relation to the rule of thumb of 1 ft/sec for grit chambers.

[16] Gerald Lacey, " Stable Channels in Alluvium." Min. Proc. Instn Civ. Engrs vol. 229 (1929–30), p. 259. See p. 283 and conclusion (6) on p. 285.

With regard to side weirs, he entirely agreed that they should be abandoned wherever possible on sewers. When a hydro-electric engineer wanted to get rid of surplus water, he could usually use any means he wished without hindrance, for it was clean water; but when a sewerage engineer wanted to get rid of water, he was putting a foul liquid into somebody else's watercourse, and that was a very different proposition. The pollution caused by side weirs set low on sewers seemed to rule them right out of court. The operation of side weirs required practical research.

There was another point on which he felt that research was very much needed. Mr Oakley had referred to model experiments on sedimentation tanks. In experiments which Mr Braine had carried out on a small circular sedimentation tank with a single-blade scraper, the scraper blade had caused the bottom layer of water next the floor to circulate around the tank so that it seemed to follow the blade; if permanganate crystals were put in the water ahead of the blade the colour began to move long before the blade had reached the crystals; but the water layers above the blade, flowed radially towards the side weir as one would expect them to. He posed the question as to whether or not the same thing occurred in tanks of normal size. He was not sure. With the model tank, when the speed of revolution of the blade was too fast, turbulence was set up and eddies developed along the blade. The side effect of the wall damped the velocities near the wall, so that the first place at which eddies appeared was in the middle of the blade. That might be expected merely because the side wall was too remote for it to have any effect. Near the centre of the tank, the blade moved very slowly so that Reynolds numbers were low and the flow in front of the blade remained laminar and therefore free from eddies.

The hydraulics problem was complex and research was needed to establish what took place in full-sized tanks. Obviously, if a scraper blade moved too fast, fine sludge would only settle with difficulty—if at all—because of blade eddies. In America, where rectangular tanks were used extensively for light secondary sludges, the link-belt type of mechanism was popular. He had often wondered—and he would very much like to see experiments made in some large rectangular tanks—whether the same currents he had found in small-scale work with the circular tanks did not also occur in rectangular tanks fitted with link-belt sludge scrapers. There again, he felt that research was required, not in a laboratory but on the job where the experiment could be to full scale and using the actual material—sewage.

When a number of tanks required to be fed equally from a single source, he was in favour of hydraulic symmetry. The only difficulty in applying it and using the formula that was given in the Paper, was that channels were more commonly used than pipes, and more often than not the flow was non-uniform instead of uniform, with the result that the ratios given in the Paper were of doubtful applicability. With hydraulic symmetry that could be avoided but the difficulties were many.

In his view, practical research in the field, on the job, was urgently needed and the Hydraulics Research Station staff were surely the people best qualified to undertake it.

Mr P. G. Smales (Partner in the firm of J. D. & D. M. Watson, Consulting Engineers, London) said that he had some criticism to make of the type of overflow described by Dr Colebrook. It was satisfactory with clean water; a head could be determined and the calculation very carefully made. A few inches did not make much difference. But what happened in practice? It was evidently a combined sewer, and it might be, say 30 in. in diameter and the outlet pipe would be perhaps only 10 in. in diameter, which would choke very easily, even in dry weather.

Mr D. K. McKenzie endorsed the plea that had been made for more research. Since there was great economic difficulty, however, he suggested that university students should not be allowed to do their research at the universities, where they would very probably evolve formulae like Coleman's for the side weir. He thought they would learn far more if, during their vacations, they went to such places as Mogden where, subject to the permission of the engineers, they could, if they were nearly final-year students, do a great deal of useful research and help the practical engineers in that way.

To give an example of how universities could put one wrong, 25 years ago Coleman and Johnson of Manchester and Ormsby and Hart of London had criticized the Lloyd-Davies formula, but the main assumption they had made was that at all times, if one took a point say on top of the roof and a point a mile down the sewer, for all storms and for all conditions of storms and in all periods of the storms, the time of concentration from that roof to that point in the sewer remained the same. It was known now that there was drawdown and drawback, and that, as a matter of fact, there were varying permeabilities and times of entry. That was the kind of conclusion which might easily arise from purely academic considerations. He suggested that it was far better to have investigation on sites, where there was plenty of time to analyse the areas which were draining.

There was one point on which research could help greatly in the classification of flows in sewers. It was difficult to assess storm-water, but on many large sewer systems there were pumping systems, and if there were an indicator chart, a set of pumps, and a long length of sewer, it would be possible to pump at varying periods into that sewer and examine how the drawdown occurred. He had proved that, with a short pumping time, the peak could very easily be reduced to 50 per cent. He suggested that that was something that students could very usefully do; it could, he thought, be done on the Colne valley system and on similar systems.

It was usually said that the soffits of sewers should theoretically be on the same level. He thought they should be so designed that when the maximum flow was occurring they were at the same level. The theoretical level should be 0·05 times the diameter difference. He thought there was the further argument, especially in a surface-water sewer, that where there was a small pipe coming into a large pipe it was usually the case that the time of concentration of the small pipe was very much shorter than the time of concentration of the large pipe. Therefore, the storm from the small pipe would pass down when the large pipe was a long way from being full, unless there was either a very prolonged storm—in which case probably neither pipe would be full—or one storm happening within, say, an hour of the other, in which case they might coincide. He thought that in general, in surface-water sewer design, there was no object in connecting the branch sewer anywhere, even in theory, above the half-way line of the large sewer.

*** **Mr L. B. Escritt** (Principal Assistant to Chief Engineer, Main Drainage Division, London County Council) referred to the Authors' statement: "The general opinion is fairly widely held that current practice results in sewers larger than necessary, mainly because the balancing effect of storage capacity is neglected in design calculations. Unfortunately there appears to exist no generally accepted method of taking this factor into account."

That was controversial. During the past 20 years four or five engineers had expressed the opinion that sewers designed according to the Lloyd-Davies method were unduly large: but that did not amount to a general or widely held opinion although it might perhaps become one if many more statements of the same kind were made.

Such a view had never been held by any one of the engineers or mathematicians who had made significant contributions to surface-water theory. It had been found at an early date that the Lloyd-Davies method, far from over-estimating flows, under-estimated them because it did not make any allowance for the effect of irregular distribution of impervious area or "shape of catchment." Accordingly, Reid, Wearing, Riley, Coleman, Johnson, Professor Ormsby, and Hart had devised differing methods to allow for that inadequacy, all of which had been used to no small extent.

For confirmation of the fact that the Lloyd-Davies method did not exaggerate, one need go no further than Lloyd-Davies's original Paper[17] for evidence. It should be remembered that the Lloyd-Davies theory had arisen out of the results of careful experiments made with the aid of rain gauges and flow records in a number of areas in Birmingham.

*** This and the following contributions were submitted in writing after the closure of the oral discussion.—SEC.

[17] D. E. Lloyd-Davies, "The Elimination of Storm-Water from Sewerage Systems." Min. Proc. Instn Civ. Engrs, vol. 164 (1905–6, Pt II), p. 41.

The opinion that present-day practice led to over-sizing of sewers might have arisen from the fact that it was virtually impossible by casual inspection to see a surface-water sewer running full, together with the paucity of the kind of automatic records that would show the relation between rainfall and run-off.

That storage was a significant factor in properly designed surface-water sewerage systems was a fallacy, due perhaps to confusion between the cubic capacity of the sewerage system and the amount of storage that could be occupied in practical conditions. The storage that took place in a surface-water sewer was a function of the impervious area and the time of concentration. At the end of a storm of the " once-a-year " category, that storage was expressed by the formula :

$$C = 178{\cdot}5 \times A_p \times t^{3/8}$$

where :

C = storage in cubic feet.
A_p = impervious area in acres.
t = time of concentration and duration of storm in minutes.

Thus, when the impervious area and the time of concentration were known, the amount of storage that was occupied at the end of the time of concentration and under the effect of a storm of that duration was known, regardless of what was the cubic capacity of the sewerage system.

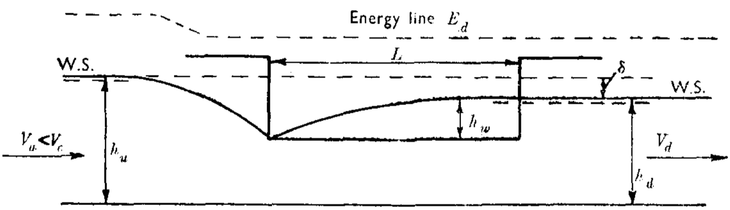
Should a storm occur of greater intensity than that for which the sewers had been designed, surcharge would occur and with it more storage capacity would be taken up. The effect of the surcharge was to increase considerably the discharge of the sewer. But examination of the mathematics of the problem showed that in the majority of instances the extra storage had very little effect from a practical point of view, for the increased rate of discharge increased the velocity, thereby reducing the time of concentration. That meant that the operative storm became one of shorter duration and higher intensity, and that necessitated a larger storage capacity in the system.

It might be true to say that there was no *generally accepted* method for calculating storage. For any method to become generally accepted it should be studied and understood by more than a minority, and that was more than could be expected to happen to any method dependent on other than the simplest mathematics. But there was a method which had been in use for many years, and whilst it might not have come to the notice of all and sundry, it had been proved mathematically sound by individual check, and no one had succeeded in demonstrating any inadequacy in it, either from the practical or mathematical aspect. Mr Escritt did not give details, for the main formulae had been given in the discussion on a recent Paper [18] by Mr Braine.

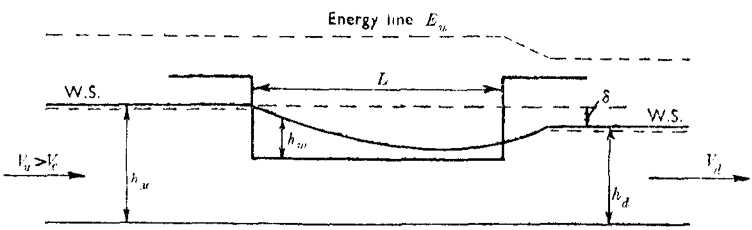
[18] C. D. C. Braine, " The Effect of Storage on Sewerage Design." Proc. Instn Civ. Engrs, Part III, vol. 4, p. 446 (Aug. 1955).

The Authors had made sundry references to the "Civil Engineering Code of Practice No. 5, Drainage (Sewerage) 1950." That publication was inferior in many respects and did not deserve the title "Code of Practice." The section dealing with surface-water sewerage was, at the best, a description of the practice of 23 years ago. There was a need for up-to-date guidance on surface-water sewerage such as could only be prepared by men who had not only picked up the elements of the work in practice, but had also made a deep study of the literature and theory. The Institution could do good work by getting together a suitable committee for that purpose.

Mr T. M. Prus-Chacinski (of the staff of C. H. Dobbie, Consulting Engineer, London) drew attention to the work of Professor Marchi of



(a) FLOW SUB-CRITICAL



(b) FLOW SUPER-CRITICAL

FIG. 14

Milan University in connexion with the Authors' statements on the limitations of Coleman's formula and the need for model study or analytical treatment of individual cases. Professor Marchi's work, carried out before the 1939–45 war, related to side weirs.

The form of surface over a side weir depended on the character of approaching flow, whether sub-critical or super-critical. In the first case the water surface over the weir was rising and in the second case was falling (see Figs 14a and 14b). In the case of sub-critical flow the shock losses were concentrated at the upstream end of the side weir; for super-critical flow the shock losses were concentrated at the downstream end of the side weir. Ignoring the losses along the length of the weir, it was assumed that specific energy in the channel along the length of the weir was constant :

$$E = E_d = h_d + \frac{\alpha v d^2}{2g} \text{ (for sub-critical flow)}$$

$$E = E_u = h_u + \frac{\alpha v_u{}^2}{2g} \text{ (for super-critical flow)}$$

where E was a constant in both cases.

In general : $$E = h + \frac{\alpha Q^2}{2gA^2}$$

where h denoted maximum depth of channel
- Q ,, discharge
- A ,, cross-sectional area at depth h
- α ,, St Venant's coefficient of correction for curvilinear velocity distribution (normally assumed as $\alpha = 1{\cdot}2$).

Differentiating the above expression along the length of the side weir x and putting $dE/dx = 0$; it could be shown that when $dh/dx > 0$, or when the water surface along the weir was rising :

$$\frac{A^3}{B} > \frac{\alpha Q^2}{g} \text{ (the condition for sub-critical flow).}$$

When $dh/dx > 0$, or when the water surface along the weir was falling :

$$\frac{A^3}{B} < \frac{\alpha Q^2}{g} \text{ (for condition for super-critical flow),}$$

where B denoted width of channel at water level.

The above could be used for a semi-graphical estimate of the length (L) of a side weir providing that L was not very large and that Q_w (discharge over a side weir) was smaller than Q_u (discharge of the approaching channel) The procedure was as followed :—

(1) A rating curve of the side weir was drawn :

$$Q_{w1} = Ch_w{}^n$$

where Q_{w1} denoted discharge over a unit length
h_w ,, head over the side weir

c and n were constants depending on the form of crest, etc.

(2) A depth discharge curve for a constant energy was drawn :

$$Q = \sqrt{\frac{2g}{\alpha}}\sqrt{E - h} \,.\, A \,;$$

where $E = E_d$ for the sub-critical case

and $E = E_u$ for the super-critical case.

(3) Normally the elevation of the crest of the side weir was known or assumed, together with the upstream discharge of the channel Q_u and the discharge over the weir Q_w. The problem was to find the necessary length L of a side weir. That as yet unknown length L was divided into small portions ΔL. Proceeding step by step and employing the above curves discharges Δq_w, Δq_{w2}, etc., over ΔL_1, ΔL_2, etc., were calculated until their sum $\Delta q_1 + \Delta q_2 \ldots + \Delta q_n = Q_w$ (the total discharge over the weir) and then $\overset{n}{\Sigma} \Delta L = L$ (the total length of the side weir).

For sub-critical discharge in the channel a start was made at the downstream end of the side weir and for super-critical discharge a start was made

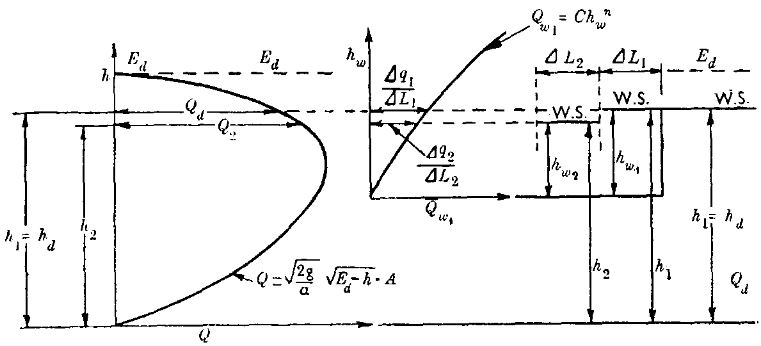


FIG. 15.—SUB-CRITICAL CASE

at the upstream end of the side weir. Fig. 15 showed the operation for a sub-critical case.

For $\qquad Q = Q_d = Q_u - Q_w\,;\ \ h = h_1 = h_d$

the following was obtained:

$$\Delta q_1 = Q_{w1} \times \Delta L_1\,;$$

Now $Q_2 = Q_d + \Delta q_1$ and $h = h_2$

then for h_2 $\qquad \Delta q_2 = Q_{w2} \times \Delta L_2$, etc.

until $\qquad Q_d + \Sigma \Delta q = Q_u$ (upstream discharge)

and $\qquad \Sigma \Delta L = L$

Mr Prus-Chacinski had applied the above method in a few cases with good results, but he was not aware of any experimental investigation affording a check of Marchi's method.

Referring to flow downstream of bends the Authors advised the application of guide vanes in order to destroy the cross-currents which caused non-uniform distribution of suspended matter across the channel. Mr

Prus-Chacinski, having made many experiments on the flow in beds of open channels, added the following information.

If there was no disturbance at the entry to a bend there existed a single spiral flow in a bend of such a nature that the flow near the bed was deflected towards the inner wall and flow near the surface was deflected towards the outer wall. As stated by the Authors, suspended matter would tend to be deflected towards the outer wall. However, much of the material tended to be dragged along the bed and such material would on the contrary tend to be deflected towards the inner wall. Motion of a similar sort persisted for a considerable length downstream of a bend. However, if the entry to a bend was disturbed by cross-currents, the resulting flow in the bed might be very complicated, taking the form of two or even three spirals. That would greatly complicate the distribution of suspended matter both in the bend and in the downstream straight.

The experiments mentioned above proved that guide vanes were a very powerful remedy and confirmed the Authors' suggestion.

Mr F. G. Hill (formerly Deputy Chief Engineer, Ministry of Health, now retired) congratulated the Authors on drawing attention to the more important hydraulic problems of sewerage and sewage disposal which required clarification and, in some cases, further research. With respect to research, the suggestion that co-operation should be achieved between the Hydraulics Research Board and the engineers of sewerage and sewage disposal authorities in order to determine reasonable and scientific solutions to those problems should command whole-hearted support. As the Authors had pointed out, such co-operation existed already between those engineers and the Water Pollution Research Board on the chemical and biological aspects of sewage purification, with inestimable benefit to all concerned with that subject.

If the co-operation suggested could be achieved, strong support must be given to the view expressed of the necessity for following laboratory or small-scale experimental work by full-scale field tests.

Mr Hill commented as followed on the cases discussed by the Authors.

Sewerage: storm-water run-off.—As a member of the committee which produced the original Ministry of Health formula about 26 years ago, Mr Hill wished to mention that the committee did not ignore the storage capacity of sewers but, as the formula submitted provided only for storms of intensity occurring about once a year, that storage capacity was assumed to go some way towards meeting storms of greater intensity occurring less frequently. Dr Glasspoole's rainfall intensity graphs were more reliable than those available to the committee at the time and, generally, the Ministry's formula did produce sometimes, for flat drainage areas, sewers of unnecessarily large dimensions; on the other hand, however, for steeply graded drainage areas, the sewers designed from it had sometimes proved inadequate in capacity. Future investigation should correct those faults and should also seek to determine, for both branch and trunk sewers independently the appropriate category of storm intensity, i.e., storms occurring in one or more years, to carry partially separated or combined flows respectively.

The range of flow in combined sewers in times of storm intensity would often exceed the 50 : 1 mentioned by the Authors and, at concentration points and upstream of overflows, might reach a ratio of up to 100 : 1 but, even so, the upper limit of velocity would rarely exceed the maximum laid down in the Civil Engineering Code of Practice. Such high velocity had the merit of cleansing deposits from combined sewers but incidentally provided one of the several disadvantages suffered at the sewage treatment works receiving combined drainage.

The Authors had mentioned the use of drop-pipe manholes to check high sewer velocities. Mr Hill had noted a tendency for engineers to use that device unnecessarily ; whereas, provided the length of sewer in question did not include a storm overflow and the maximum advisable velocity was not exceeded, the elimination of drop-pipe manholes reduced the cost of the work appreciably and also the tendency to produce hydrogen sulphide.

Storm overflows.—The Authors had confined their remarks to side-weir overflows although a number of other devices for separating excessive storm-water existed. For their larger sewers, the Americans used float-operated gates or penstocks, one on the main combined sewer and another on the branch interceptor which passed flows up to about 2 $\times$ D.W.F. to the sewage-treatment works. For smaller sewers, side-weir or leap-weir overflows were generally used. Mr Hill endorsed the Authors' criticism of the application of the Coleman formula for side-weir overflows—both in Britain and America few storm overflows in common use were either reliable or satisfactory in operation. Probably the siphonic overflow for large sewers, an improved side-weir overflow for intermediate sizes, and the Venturi and stilling-pond overflows for small sewers were most worthy of further investigation and research.

Sewage disposal.—The importance of the control of velocities and levels had been wisely stressed by the Authors but they did not mention the special need for care in that respect where the whole flow received at the treatment works was pumped—particularly at the entrance to the works.

Unless the kinetic energy of the pump discharge was effectively dissipated, the proper operation of screens would be affected and part of the capacity of the grit chambers would be rendered useless. Inadequate attention to that point had been noted in the design of even important modern works.

Separation of suspended solids.—The importance of surface area in the settlement of suspended solids had not been fully realized by British engineers until comparatively recently, whereas American engineers had tended to stress surface area to the neglect of other factors.

Grit chambers.—In addition to the horizontal channel, the vortex chamber and the upward vertical-flow chamber deserved further study as a means of removing grit.

Sedimentation tanks.—In such tanks a device was employed which had been in use for several thousand years and yet much remained still to be learnt of their most effective design and operation. The shape of the tank, its inlet, and outlet were all factors offering scope for study and improvement and, furthermore, the characteristics of the various solids to be settled must be fully taken into account.

Distribution of flow.—That important function was often given insufficient attention by the designer. The Authors had provided a good lead to improvement in that respect but there was little doubt that the subject would repay further investigation. With regard to bends in distribution channels, a relation existed between the radius of the bend and the effect of centrifugal force and the friction on side walls. Research should determine the optimum radius of bend, in relation to the width of the channel, which would reduce the effects of centrifugal force and friction to the minimum. That had been done for pipes by Alexander about 40 years ago.

In their conclusion the Authors had mentioned a number of other hydraulic aspects of sewerage and sewage disposal. Siphonic overflows had already been referred to by Mr Hill and of the other points the most important, in his opinion, was the pumping of sewage sludge. The treatment and disposal of sewage sludge was a costly business and the density at which sludge could be economically handled and transported affected that cost appreciably.

Mr J. F. McIllwraith (of the Metropolitan Water, Sewerage & Drainage Board, Sydney, New South Wales) agreed substantially with the views expressed by the Authors, but thought that some comment based on the operation of the sewerage system of the authority to which he was attached would be appropriate.

The sewerage system under the control of the Board catered for a population of about 1,460,000 persons. About 97% of the sewage was discharged into the Pacific Ocean *via* three major and three minor outfalls; the remaining 3% being discharged to a number of small independent treatment works. Since the ocean outfalls had been constructed, surf bathing had developed as a national summer pastime and now the need had arisen for the installation of such treatment at the outfalls as would eliminate any possibility of nuisance on the beaches. Partial treatment had already been provided on one outfall, and as time passed the extent and degree of treatment would be progressively extended until primary treatment was provided on all outfalls.

With the development of treatment works chemical and biological problems had arisen that previously were non-existent. However, for many years to come the problems associated with Sydney's sewerage would be predominantly physical ones of which the hydraulic aspects played an important part.

The first outfall at Sydney was installed on the combined system, but since then separation had been carried out on most of the area and all subsequent works had been constructed as separate systems. By and large therefore the Sydney sewerage system might be regarded substantially as a separate foul-water system.

From time to time various allowances had been utilized for the design of Sydney's sewerage system. At the present time the method employed was to convert commercial and industrial areas into an equivalent residential population on a flow basis. Sewer pipes were then designed flowing full to cope with the following requirements :

$$Q_w = D\left[0{\cdot}001447\left\{P + Cp + E.P.\left(\frac{L}{3}\right)\right\}^{0{\cdot}774}\right] \quad . \quad . \quad (1)$$

Where Q_w denoted the wet-weather discharge in cusecs.

D denoted a dilution factor having the following values :—

For 6-in. pipes $D = 6$

For 8-in. pipes $D = 4{\cdot}5$

For 9-in. or larger pipes $D = 4{\cdot}0$

P denoted the total residential population.

C denoted a dispersion factor to be applied to industrial peak flows to provide for the greater dispersion of those peaks.

p denoted the totalized equivalent population (E.P.) of all industrial and other discharges in which the peak flows were unlikely to occur simultaneously or during the period of the residential peak.

L denoted the pumping rate in cusecs of any low-level pumping stations.

Equation (1) stated algebraically that pipes flowing full should be designed to cope with the following requirements :—

6-in. pipes to cope with 6 times peak dry-weather flow.

8-in. pipes to cope with 4·5 times peak dry-weather flow.

9-in. or larger pipes to cope with 4·0 times peak dry-weather flow.

Tables were available based upon actual sewer gaugings showing the equivalent population to be expected from various types of industry.

The average annual rainfull for Sydney for the 30-year normal period 1911–1940 was 44·80 inches. That rainfall was spread over the whole year and frequently resulted in relatively high intensities. The stratas encountered in sewer trenches covered soil, shale, sandstone rock, sand (frequently water-charged), and clay. The latter reached almost to the surface, and contracted and expanded with varying degrees of moisture content. A large proportion of the reticulation sewers were laid in that clay, the relative movement of which frequently caused the pipes to crack and even fracture. Under those conditions flooding of surface fittings and ground-water infiltration combined to create relatively high wet-weather flows in the system. Naturally the design allowance quoted in equation (1) did not cope with wet-weather flows resulting from sustained heavy rainfalls. Those excess flows were overflowed at convenient points into the adjoining storm-water drains. The major storm-water drains were generally controlled by the Board, whereas the subsidiary drains were under the control of Local Government Authorities. The overflows from the foul-water sewers were set at such a level that the overflows did not come into operation until the wet-weather flows exceeded a minimum of 3 times peak dry-weather flow, based on the assessed ultimate development of the area. In Sydney's sewers the peak dry-weather flow amounted on the average to about 3 times average dry-weather flow at a distance of 1,700 ft from the head of the catchment, diminishing to about 1·5 times average dry-weather flow at 15,000 ft from the head of the catchment. Wherever possible baffles were included in the overflows when the discharge was into open storm-water drains in residential areas. Four types of overflows were normally used :—

(1) Straight pipe with bellmouthed outlet from the manhole.
(2) Overflow from manhole *via* baffle, weir, and overflow chamber.
(3) Siphonic.
(4) Side weir.

The last two types were frequently used on the larger sewers, the side-weir type being preferred on account of its more rigid control of the height of the hydraulic gradient. Coleman's formula for the length of the side weir had been found to give results of an accuracy comparable with most other sewerage calculations.

Manning's velocity formula was adaptable for graphical and other ready means of application. It was therefore preferred for the determination of velocities in pipes and channels. Under certain conditions it was subject to substantial errors, but such errors were far less than those introduced by the assumption of uniform flow and the estimation of the net sewer area and smoothness of surface.

Maintenance of adequate sewer velocities was desirable to ensure that grit and sludge were not deposited on the sewer inverts with resultant generation of hydrogen-sulphide gas. Experience on the Sydney sewerage system, however, indicated that velocities substantially less than the 1·5 ft/sec quoted by the Author were quite satisfactory for dry-weather-flow conditions. The present practice at Sydney was to design sewers to give a cleansing velocity at peak dry-weather flow equivalent to that obtained with a velocity of 2 ft/sec when the pipe was flowing full. That was tantamount to the substitution of peak dry-weather flow in the Sydney sewers for the minimum daily average flow as proposed in the final report [19]

[19] "Minimum Velocities for Sewers." J. Boston Soc. Civ. Engrs, vol. 29, No. 4, p. 286 (Oct. 1942).

of the committee appointed to study limiting velocities of flow in sewers.

Sydney's sewage was aged up to about 12 hours and naturally hydrogen-sulphide-gas attack was a problem, but not a major one. The fact that the low minimum velocities and the stale sewage did not create a gas problem might be due to the fact that every time it rained the system was flushed out by the abnormally high wet-weather flows.

With regard to maximum velocities it was considered that high velocities at the upstream ends of a sewerage system where the sewage was fresh were not unduly objectionable, but as the sewage aged in the bigger sewers velocities in excess of about 8 ft/sec tended to create turbulence at inlets, even when the relative flow levels were quite good. Text-books frequently quoted maximum permissible velocities of about 12 to 15 ft/sec for storm-water drains. So far as Sydney was concerned there were many concrete and brick drains that had operated without noticeable invert wear for the past 50 years with velocities in excess of 60 ft/sec.

Although high velocities were permissible in reticulation sewers they did not create a serious surcharge problem at manholes unless the location and grading of the sewers were such that turbulence at the manholes was avoided. For instance acute bends and steep grades meeting flat ones caused velocity heads to be converted into static heads and turbulence at the manholes, the covers of which frequently blew under wet-weather conditions.

It was agreed that there was a tendency today to overdesign storm-water drains. In the past those drains were undoubtedly under-designed, but since the introduction of frequency analysis the pendulum had swung to the other extreme. Peak rainfall intensities produced storm run-offs which were largely damped out by depression storage, minor floodings in the street-gutters, and in channel storage. The hydrograph of flood flows in small urban catchment areas was a very steep one and in consequence it did not take much storage to flatten out the peaks. The position could be improved somewhat by adopting a minimum time of concentration—say 15 min—for all catchment areas, however small. The coefficient of run-off appeared to be over-estimated for rainfall intensities of recurrent intervals less than about 10 years. When the ground became thoroughly saturated, as in storms of rare frequencies, the values normally quoted for the coefficient of run-off might, however, be appropriate or even low. The question of storm-water run-offs in Australia was at present being investigated by the Institution of Engineers, Australia, and in due course it was hoped that considerable improvement in storm-water design would result therefrom.

Mr F. C. Temple (Consulting Engineer in private practice) said the two brief paragraphs on sewer junctions on p. 669 omitted any reference to one effect common when sewers were joined soffit-to-soffit or dry-weather water level-to-dry-weather water level, that whenever the sewers were not running full the junction was likely to become in fact a right-angled junction. If the smaller incoming sewer was cut off high in the side of the main sewer its liquid would turn and fall by the shortest route to the bottom of the main sewer, and often cause the solids that it carried to pile up into a small heap on the invert of the main sewer just upstream of the point at which it reached the invert of the main sewer. That could only be avoided by sweeping down the small sewer so that the two sewers met invert-to-invert to make a junction streamlined in section as well as in plan, or by making a drop in the small sewer as suggested in the second paragraph.

Side-weir overflows in practice could be surprisingly ineffective. A V-shaped open surface-water drain about 3 ft deep on a gradient of 1 in 60 with a 12-ft-long side-weir 6 in. below its edge, when flowing full, shed a negligible discharge over the side weir until the drain was slightly throttled at the end of the weir and the drain itself roofed over for half the length of the weir to hold down the fountain that was the first result of the throttle.

In discussing the flow through settling tanks no mention was made of the effect of temperature changes. In India, in a pair of oblong settling tanks 10 ft deep intended to allow 2¼ million gal to pass through each in 6 days, in which the inlet and outlet were both over weirs the full width of the tanks, on several occasions the fresh water from the river had reached the outlet and overflowed in less than 3 hours. The hot water from the river skimmed over the colder water in the tanks. That was cured by inserting a baffle at the inlet end so that the incoming water had to mix with the colder water about 2 ft below the surface.

Mr F. C. Vokes (Engineer to the Birmingham Tame & Rea District Drainage Board) observed that the breaching of the Mohne and Eder dams was the result of a brilliant conception, tireless research, skilful design and production, and unswerving purpose regardless of sacrifice. The result was a noteworthy event in the field of hydraulics. From a sewage disposal point of view it was not without importance. The project was conceived during the 1939–45 war at a time when progress seemed to have got bogged down and the result, although regrettable, was a great success.

Mr Vokes suggested that today the science of sewage disposal had become similarly bogged down and that a new conception must be sought before substantial progress could be made. To that end research was needed on a national scale by a team of scientists devoted solely to that work. The sewage works of Britain would be for the research team one vast experimental plant on which observations could be made and from which conclusions might be drawn as the result of statistical research, analytical results, and hydraulic and other experiments.

As a result, expenditure on future works might not only be considerably curtailed but used to better purpose.

Mr W. H. Hillier and **Mr R. V. Collman** (both of the City of Bradford

Sewage Department) observed that for many years, sewerage and sewage works designers, whether as consultants or as engineers in the service of local authorities, had been acutely aware of the need for organized research into the fundamental hydraulic problems of their work. The Authors rightly acknowledged the contributions that had been made from time to time by the authorities responsible for drainage and sewage disposal, and it was probable that the volume of such work was greater than was generally realized. Many minor researches were carried out as a preliminary to the design, say, of weir chambers, detritus channels, and similar works; but the results of such investigations were not published unless they were of such magnitude as to justify a full-length Paper. Researches had also been successfully carried out by universities and technical colleges in collaboration with their neighbouring authority. One of the functions of a national research organization might well be to collect and collate those scattered contributions.

It was equally true, however, that the types of hydraulic research work most useful to the designer were now outside the scope of any local authority. Research did not pay quick dividends and useful results could be obtained only by the full-time employment of well-qualified staff. No local authority was prepared to ask its ratepayers to subsidize the national research effort in that way. The very practical suggestion, therefore, that the Authors put forward, of a central organization to undertake full-scale researches in collaboration with the various authorities, was one which all sewerage and sewage works engineers would welcome. There would be no lack of scope for such an organization.

The absence of fundamental research had resulted in the design of sewerage and sewage disposal works being subject, perhaps more than in any other civil engineering field, to the methods of empiricism and tradition. The Authors had pointed out that those hydraulic problems were complicated by the peculiar and variable characteristics of sewage itself. In the investigations, for example, into flow through pipes and channels little regard had so far been given to the effect of the presence of transported matter such as grit and stones, or of organic matter including grease and detergents; or even to the formation of a surface film which made nonsense of any theoretical "roughness factor". It was perhaps for such reasons that sewer designers had fought shy of the more precise formulae derived from laboratory work, and had generally preferred the simple Crimp and Bruges formula which at any rate was derived from experimental work carried out in real sewers.

Again, with reference to side-weir overflows, the Authors were right to criticize the inadequacy of the data on which the Coleman and Smith formula was based. That formula had been blessed by the Civil Engineering Code of Practice and was accepted by every examination syllabus. It possessed an aura of accuracy by quoting its coefficient to three decimal places, but practical designers had recognized its limitations by providing their overflows with adjustable weir-boards and baffle-boards, so that the worst consequences of faulty design might be mitigated in the light of experience.

The designer would share too the Authors' plea for more fundamental knowledge in the design of sedimentation tanks. During the past 30 years the choice of design, whether upward-flow, horizontal rectangular, or horizontal radial, appeared to have been based on fashion rather than on the results of sound research. Designers were also painfully aware of the need for satisfactory tank inlet arrangements, but at present were confused by a bewildering assortment of inlets, combining bellmouths, baffles, buckets, tees, and grids of all kinds and shapes. Some positive results would probably emerge if a competent research team were put on to that problem.

Theory and practice were more at harmony with regard to grit extraction, which was now adequately dealt with in most new designs. The Paper was unduly modest concerning the part played by one of the Authors in establishing a sound basis for the design of constant-velocity flume-controlled grit channels.

The list of hydraulic problems given in the Authors' final paragraph could well be amplified. In many of those cases, such as the flow of sludge through pipes and its behaviour in pumps of various kinds, there was almost a complete absence of fundamental data. It was to be hoped that the Paper, which so clearly demonstrated the need for further research, would be instrumental in giving new impetus in that neglected field.

Mr W. A. M. Allan (Divisional Engineer (Main Drainage), London County Council) congratulated the Authors for their salutary reminder of how little was known concerning the velocities, cross-sectional areas, and depths of flow which would obtain in an ordinary sewerage system. In most instances calculations of those properties were based on the assumption that each size of sewer had a long length of uniform gradient, that it was perfectly straight and had no interfering influences such as sewer and drain connexions. Actual conditions, particularly in densely populated areas, might be very different, and so might the actual velocities and flows occurring.

The London County Council had recently embarked upon a major review of the London sewerage system and the first task had been to obtain a reasonably reliable set of dry-weather-flow figures. Flow data calculated by the Crimp and Bruges formula (usually considered as giving slightly conservative results) from ordinary depth gaugings taken at points selected for best uniform gradient and as reasonably remote from large incoming sewers had in places shown such a lack of consistency as to require further investigation. A new technique had, therefore, been developed which might briefly be described as a development of the electrical salt-velocity method applied to a "round the clock" set of depth recordings.

The method, which was believed already to give fairly accurate results, would no doubt be further improved by longer experience in its use. It was already clear that ordinary calculations for velocities in sewers flowing partly full might be up to 50% in error and the discrepancies were no doubt due not only to some difference in the roughness coefficient of the velocity formula, but also to minor irregularities in gradient, interferences by tributory sewers and drain connexions. Further research into the relative magnitude of those various forms of interference was badly needed.

The Authors stated at the top of p. 667 that with a combined sewer with a 50 : 1 range of flow the maximum velocity need not be more than 3·75 ft/sec. That might be so with a large sewer, but with the smaller sewers the minimum depth of flow would almost certainly be greater than the theoretical 0·1 proportional depth with corresponding reduction of velocity below the desired minimum of 1·5 ft/sec.

Mr Allan agreed with the Authors that complete reconciliation of all the hydraulic factors involved was not practicable when two or more sewers joined, but he felt that generally it would be preferable to line up the sewers at proportional depths of 0·82 (depth when nominal full capacity of sewer was reached). The method as compared with soffit-to-soffit alignment gave slightly smoother confluence at maximum flow and reduced drop at mean dry-weather flow by about 40%.

Mr Allan was particularly interested in the remarks regarding drop pipes and ramps. In London the usual practice for the large brick sewers had been to provide stepped ramps, sometimes with a side channel for dry-weather flow where drops from one level to another were required. Whilst it was realized that some turbulence was caused, it was considered that the first consideration must be the safety of men working in and traversing the sewers, and therefore some form of stepped ramp was preferable.

The Authors' review of flows in sedimentation tanks was very interesting, and Mr Allan fully endorsed their contention that further research was required, particularly with regard to the design of inlets. Whilst that was important in sedimentation tanks of large capacity, it was probably even more important in small tanks serving small communities or isolated residential schools. He had in mind the great variation in behaviour of a number of small plants consisting of sedimentation tank, biological filter, and humus tank, where it had been found that there was on occasions considerable variation in the quality of the effluents, both from plant to plant and even on the same plant. There seemed to be no doubt that with such plants where it was impossible to provide adequate skilled maintenance, inefficient or badly designed inlets could have a very marked influence on the tank effluent.

It was noted that in describing the method for adjusting the size of channels for differences in their lengths (p. 683) the Authors used the classic formula for the loss of head in pipes, which was really the original Chezy formula in another form. That formula was now usually acceptable only if C was allowed to vary. It might be more consistent, therefore, if a formula of the Manning or Crimp and Bruges type were used, i.e., $v = CM^{\frac{2}{3}}i^{\frac{1}{2}}$, which gave $h = \frac{v^2 l}{c^2 m^{4/3}}$.

Mr J. R. D. Francis (of the Civil Engineering Department, Imperial College of Science and Technology) observed that the Authors had summarized engineers' requirements for velocity formulae for flow in pipes and channels, and had brought out the need for being able to predict the hydraulic constants of a pipe from the measurements of the roughness on the surface. That problem had been hardly touched as yet. It was now known that the relative roughness, i.e., the ratio of the sand-grain size to diameter of pipe, was inadequate to forecast the friction factor. The shape of the grains and even the amount of glue used to stick the grains to the pipe wall were important.[20] The original researches of Nikuradse on roughness of pipes that had been quoted so often were carried out on roughnesses of a very special sort; for example, uniformly sized sand grains stuck on to a wet varnished surface and then varnished over again. Thus some varnish might have filled interstices between grains, and the actual size of roughness in situ was likely to be different from the dry sand-grain size which was used as the definition of the roughness. Much more complicated formulae than Nikuradse's must therefore be expected for the friction factor if better accuracy was required and if those other properties of the roughnesses were taken into account. A considerable research effort would be required which would demand suitably qualified staff, a problem of some difficulty at the present time.

The Authors, in reply, said that they felt that the discussion had very adequately achieved the purpose which they had had in mind, namely, to set out in the Proceedings a clear indication of the magnitude of the research work that still needed to be done in the field of sewerage and sewage disposal.

They were sure that all Members would be interested in the statement of Sir Claude Inglis on the work of the Hydraulics Research Board, and that the Station would be open to visitors. Members would also take note of his implication that the type of work to be done might to some extent depend on how much pressure could be brought to bear from outside. In view of that, the Authors felt that work of a more general or national character might be in danger of being left behind, since there might not be enough individuals pressing for it to be done. One of the objectives of the Paper had been to elicit in the discussion a record of work still outstanding so that when the Board had time to spare, they could obtain from

[20] L. E. Prosser, R. C. Worster, and S. T. Bonnington, "Friction Losses in Turbulent Pipe-flow." Proc. Instn Mech. Engrs, vol. 165 (1951), p. 88. *Discussion*, p. 94; *See* R. F. McMahon, p. 101.

that record an indication of national requirements even though there might be nobody pressing individually.

In dealing with the discussion generally, the Authors wished to welcome the differing points of view of the many speakers rather than to give their own opinions on matters which should be finally decided authoritatively by collective opinion and research.

Mr Ely had expressed a preference for symmetrical arrangements in distributing the flow rather than a reliance on the relation of D to L which was not capable of commercial adjustment. That would be so with pipes and in that case there was something to be said for the alternative of symmetrical arrangements where they could be made ; but distribution was not always by pipes ; very often it was by culverts which were far more flexible in that way. It was quite easy to make culverts with hydraulic resistances which were similar, for which purpose, of course, they should be "drowned." The trial-and-error methods of analysis mentioned by Mr Ely were quite useful.

Dr Colebrook had mentioned that everyone had his own preference for a particular flow formula ; that was a state of affairs to which the Paper had drawn attention, and for which there was no excuse. It was 180 years since the first formula had been developed, and the general confusion had been recognized by the Institution itself as long ago as 1936, when the Sub-Committee on Velocity Formulae for Open Channels and Pipes had been set up "to evolve a basic formula which is universally applicable to pipes and channels." A further 19 years had gone by since then, and an authoritative verdict from the Institution was still awaited. The subject appeared to be one on which there could be an agreed standardization ; it did not come into the category of the usual code of practice where allowance should be made for initiative in the variation of practice from one place to another. Surely there should be some kind of authoritative verdict on what was the right thing to do.

Dr Colebrook's recommendation of the Manning formula had been shared by a growing body of opinion that it did in a relatively simple manner best fulfil the all-round requirements of good practice, but the Authors would stress that throughout the Paper they had tried to record the state of thought on those matters rather than express definite views on points which might still be controversial.

Dr Colebrook had given some interesting simplifications of the various coefficients. The formulae which had been developed on the Continent for pipe flows, particularly for rough pipes, had generally involved a coefficient which depended on the diameter, which in practice might mean quite considerable complication in actually making calculations. That was what was at the back of the Authors' minds in their statement on p. 666 that it was hoped that the work of the Committee would, in due course, enable them to make the difficult decision of recommending a standard practice which was both reasonably reliable and simple.

Mr Oakley had made a plea for support from the profession for work in the universities. Of course, the work of the universities was very much appreciated; they were now working in collaboration with the Hydraulics Research Board in many directions, and contributing materially to the integration of all national resources in that research. Mr Oakley had also mentioned the benefit of research activity at the universities in training an adequate number of research-minded engineers, which was vital to the success of such work.

Mr Oakley had referred to the very small head losses in various parts of works and the difficulty of extrapolation of results from the small sizes to the large. There was no doubt that a great deal of information could be obtained from existing works and that for such a purpose better results could be obtained by grafting research workers on to existing works organizations rather than hoping for the information from authorities who were finding it difficult merely to carry on with routine operations under present circumstances. Such an arrangement led to the achievement of quicker and more worthwhile results.

Mr Oakley had raised the question of the application of the Hazen assumptions in the operation of grit channels. There were two points to consider; the first involved differential settlement and prevention of settlement of solids which depended on the control of turbulence as Mr Oakley had described; for the actual settlement of the grit, however, design would definitely have to be based on Hazen's law. Obtaining a variation of conditions by the control of diffused air had been mentioned in the Paper, and it promised to effect a great improvement in general grit chamber practice.

In regard to Mr Oakley's remarks on sedimentation tanks, it had been the Authors' intention to state the principal requirements for perfection, which, of course, would probably never be achieved. If the whole question was not studied from the theoretical point of view and some advance towards perfection was not tried, there would never be any progress in sedimentation. Some people were inclined to say that because there were so many difficulties in connexion with temperature, wind currents and gravity currents, turbulence, and so on, perfection was unattainable and therefore there was no point in following theoretical considerations. The Authors did not accept that suggestion. Sound theory would point in the right direction and practical design must be adapted to achieving the ideal so far as possible.

Mr Oakley had also mentioned models as suitable for various purposes; the Authors agreed with that. Models could certainly indicate trends, but in the long run experimental work and research on full-scale tanks were essential. Mr Oakley had mentioned the possibility of doing that by obtaining research facilities on large-scale works. That was exactly what the Authors had in mind in the Paper—the grafting of research workers on to existing plants to obtain the best results in practical conditions.

Mr Wolf had pointed out the difficulty that the demand for research was greatest in times of maximum construction when fewer personnel were available for it. That was all the more reason for a well-planned, long-term programme for hydraulics research similar to that carried out by the Water Pollution Research Board, rather than research carried out haphazardly according to the requirements of individual authorities at particular times.

Mr Wolf questioned the concavity of the curve of the path of settlement of a particle entering the hyperbolic circular tank at the surface, shown in Fig. 7. However, under the assumption of constant radial horizontal velocity, both graphical and mathematical analysis showed that to be correct. For particles entering the tank at lower levels, the concavity of the path decreased with depth, there being, in fact, one level of entry for which the path was a straight line. Below that level the paths became increasingly convex, and particles entering the tank in the lower half of the depth actually reached the floor at levels higher than those at which they entered.

Mr Wolf's statement that the radial surface velocity was decreasing from the centre outwards was not understood by the Authors, since with the flow admitted to the tank for the full depth near the centre, the hyperbolic shape of the floor gave a constant area normal to the direction of flow, resulting in constant radial horizontal velocity.

Mr Brown's observations on the Coleman and Dempster Smith formula supported the Authors' conclusions. They were aware of the article on the design of side weirs, based on the work of De Marchi and Gentilini, in Schoklitsch's book.[21] The method of design referred to there was based on the assumption of constant specific energy—that was, that the energy line was parallel to the bed of the channel. The method the Authors used, however, allowed for the change in specific energy owing to friction loss along the channel.

On the problem of flow distribution, a valuable contribution [22] to the subject had been published since the Authors wrote their Paper. Whilst the work referred to was related to another branch of civil engineering, it had application to the design of sewage works, and was a good example of the type of problem that could be solved by models and mathematical analysis.

Mr Thomas had dealt with the subject from an irrigation angle and raised several points concerning velocities. The velocity of 1 ft/sec for the settlement of all heavy grit down to a certain fineness had been widely accepted both in Britain and America, and it worked extremely well in

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[21] A. Schoklitsch, "Handbuch des Wasserbaues." Vol. 1, 2nd Edn, Springer-Verlag, Vienna, 1950.

[22] John Allen and Brian Albinson, "An investigation of the manifold problem for incompressible fluids with special reference to the use of manifolds for canal locks." Proc. Instn Civ. Engrs, Part III, vol. 4, p. 114 (Apr. 1955).

practice. The work on irrigation to which Mr Thomas had referred was based on the rather opposite requirement of the suspension and transport of all minerals up to a certain size.

Mr Braine, like almost every other speaker, had referred to the urgency of research. In speaking of deposits in the inverts and accumulation of grease and fungi on the sides of sewers, Mr Braine had obviously been referring to very old and badly constructed sewers. When discussing modern practice related to the future it did not seem necessary to study such conditions. Certainly in the West Middlesex sewers not one bucket of grit was removed from one year's end to another. Modern sewers could be designed to prevent that; therefore variations of coefficient on that account did not occur.

Mr Braine had wished for information from West Middlesex on how the coefficients had altered during the 20 years that the undertaking had been in operation. The Authors believed that there had been no change whatever during the period, but they would be glad to arrange for that to be checked if the necessary personnel were available; all the facilities desired would be readily given.

The movement of water in front of a blade at the bottom of the sedimentation tank had been mentioned in the Paper. With modern equipment the Authors felt that the blades would become shallower in design with more continuous operation, and the effect of water movement progressively decrease.

Mr Smales's comment on the small pipe exit becoming blocked was very practical, but another point concerning the type of over-flow referred to was that the head was not always available to operate an arrangement of that sort; on the other hand, if a definite orifice working under a fixed head could be used, it was possible to obtain a far more accurate control.

The Authors felt sure that most authorities would facilitate the carrying-out of Mr McKenzie's suggestion regarding research by university students and that a great deal of useful work could be done in that way. That, of course, would require supervision, which perhaps could be done by the university itself. Although complete co-operation with the staff of the authority would always be possible, there might be difficulties in putting such staff on to direct supervision of the work of students.

Mr Escritt had made a valuable contribution to the controversy on the subject of surface water run-off which gave still more point to the urgent need for an authoritative decision on that question.

The points raised by Mr Prus-Chacinski had been dealt with in the Authors' remarks on the contribution by Mr Brown.

The Authors were grateful to Mr Hill for the comprehensive summary of work requiring attention which he had supplied as the result of close personal observation during many years of the work carried out in that aspect throughout Britain. They entirely agreed with his observations on drop-pipes on steep sewers and, in their opinion, drop-pipes found their best application at sewer junctions. Mr Hill's point concerning kinetic energy at discharge to screens and grit chambers was very important.

Mr McIllwraith's contribution was a timely reminder of the differences in conditions which occurred in other countries, and his account of the problems at Sydney was very interesting.

Mr Temple's reference to sewer junctions appeared to be directed towards the smaller sizes of pipes where the defects he described undoubtedly led to trouble. The effects of temperature on the operation of sedimentation tanks had been mentioned on p. 676.

Mr Vokes had expressed the visionary possibilities which might be attained in the future from a sound research programme.

Mr Hillier and Mr Colman had added to the general agreement on the urgent need for all types of research, and had emphasized the advantages of a central organization working in collaboration with sewage authorities.

Mr Allan had described a new technique used by the London County Council for sewer gauging which was of great interest. It was that kind of contribution to the pool of national effort which could be so valuable in making progress throughout Britain. The Authors hoped that in due course further details and results would become available. Mr Allan had also added his agreement to the general need for further research on an organized basis.

The comments of Mr Francis on the development of velocity formulae presented yet another aspect of that subject.

Correspondence on the foregoing Paper is now closed.—SEC.