

LABORATORY STUDIES OF STORM OVERFLOWS WITH
UNSTEADY FLOW

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CONTENTS

	PAGE
INTRODUCTION	1
DESCRIPTION OF APPARATUS	4
SEDIMENT SIMULATION	5
DESIGN OF OVERFLOWS	7
PROGRAMME OF TESTS	15
ANALYSIS OF RESULTS	17
CONCLUSIONS	23
ACKNOWLEDGEMENT	28
REFERENCES	28

CONTENTS (Contd)

FIGURES

1. Experimental apparatus
2. Low double side-weir overflow
3. Stilling pond overflow
4. Vortex overflow with central spill
5. Storage overflow
6. Vortex overflow with peripheral spill
7. Discharge - Time Curves. Pipe slope 1:500
8. Discharge - Time Curves. Pipe slope 1:100
 - (a) 1 and 2 minute waves
 - (b) 3 and 4 minute waves
9. Proportion of wave volume spilled. Pipe slope 1:500
10. Proportion of wave volume spilled. Pipe slope 1:100
11. Proportions of pollutants spilled. Pipe slope 1:500
12. Proportions of pollutants spilled. Pipe slope 1:100
13. Average concentrations of pollutants in spill as proportions of base-flow concentration. Pipe slope 1:500
14. Average concentrations of pollutants in spill as proportions of base-flow concentration. Pipe slope 1:100

PLATES

1. Stormwater overflows for sewers
 - (a) a low double side-weir
 - (b) a stilling pond
 - (c) a vortex
 - (d) a storage overflow
2. Vortex overflow with peripheral spill

NOTATION

A_p	effective impermeable area
D	pipe diameter
h_o	depth of base flow
h	depth of flow
i	rainfall intensity
L	length of pipe
Q_f	discharge capacity of pipe when just full
v_f	velocity in pipe flowing just full
V	volume of foul flow in pipe before storm wave
V_1	wave volume
x	distance along pipe

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INTRODUCTION

The main purpose of a storm overflow structure in a combined sewer is to restrict the flow passing to treatment as a result of a storm to a value which can be dealt with by the purification plant. To this end a part of the flow arriving at the overflow is diverted from the foul system and passed untreated to a river, an estuary or the sea. This is necessary for two reasons:

- (a) it is uneconomical to build a purification plant capable of treating all storm discharges or to provide sufficient storage volume to retain the full volume of storm sewage for later treatment at a slower rate;
- (b) it is uneconomical to build sewers capable of carrying the full storm discharge to the treatment plant.

Most existing overflows meet this requirement fairly satisfactorily, in that they reduce the flow to treatment during storms by overflowing storm sewage. It is the degree of control of the discharge and the pollution of the spilled sewage which cause concern, and which are the reasons

for this model investigation. The ideal overflow would start to spill when the flow to treatment reached a predetermined maximum value, any excess then being spilled with little or no foul content, almost all the polluting material being passed to treatment. In the experiments described here the performances of four types of overflow are compared, and the extent by which each falls short of the ideal is illustrated.

Other aspects of the behaviour of time-variable storm discharges in pipes were studied in two earlier investigations. In the first¹, in which the attenuation and rate of movement of a storm wave were measured as it travelled along a pipe, it was found that the velocity of the peak of the wave was greater than the velocity of the water. This phenomenon is well known in river engineering but less attention has been paid to it in the sewerage field. In the second investigation², in which the movement of the water of which the storm wave was originally composed was studied, it was found that relatively little longitudinal mixing took place between the existing steady foul flow and the storm water. It was evident, therefore, that at an overflow site some distance down a sewer an increase in depth and discharge would occur in advance of the arrival of storm water itself and that the front of this flood wave would consist of undiluted foul flow. This part of the wave is known as the first foul flush. In addition, the increasing velocities might set in movement deposits of grit and other heavy solids which had built up during the preceding dry period.

In the present experiments, the pipe, together with the apparatus to supply and control the discharge to it, was used as a model combined sewer on which to install the structures to be tested. The scale was rather small, the diameter of the pipe being only 3 in., and it was realized that the results obtained would be of use mainly in

comparing the performance of different types of overflow, not in predicting accurately the behaviour of full-scale structures of the same type. However the particular advantage of the apparatus was that the overflows could be studied under non-steady flow conditions, as occur in a sewer. Furthermore, another research programme was undertaken at Luton to test much larger models, three being of the same types, using real sewage, in co-operation with the Water Pollution Research Laboratory³.

The apparatus was equipped to provide a storm wave of fresh water, superimposed on a saline base flow representing dissolved pollution and fine suspended solids. Although these form by far the greater part of the total pollution load, the floating and coarse suspended solids are more noticeable and offensive when spilled. Grit, which may be in motion in sewers only during storm flows, may also be troublesome if spilled. It was decided therefore, to inject particles into the flow to simulate these materials and, for reasons which are described later, polythene, polystyrene and bakelite were the materials chosen to represent floating, suspended and heavy solids respectively.

The four types of overflow structure which were chosen for comparison in this investigation were:

- (a) a low double side-weir;
- (b) a stilling pond;
- (c) a vortex with central spill;
- (d) a storage overflow with high side-weirs.

Experiments were conducted at two pipe slopes, 1:500, in which the flow conditions upstream of the structures were subcritical, and 1:100, in which supercritical flow occurred. At the steeper gradient a fifth type of overflow, a vortex drop with peripheral spill, was also tested. This overflow was not designed until after the 1:100 experiments had started and has therefore not been tested at the 1:500 slope.

DESCRIPTION OF APPARATUS

The apparatus is shown in Fig. 1. It consisted of a 250 ft length of 3 in. dia. pipe, made up of five 10 ft long perspex sections and four 50 ft steel sections, suspended from scaffold frames so that the slope could be varied between 1:1000 and 1:50. A constant-head tank, fed by a pump from a constantly replenished sump (the need to measure salinities precluded re-circulation), supplied the pipe with water by two paths: firstly, the base flow, representing dry weather flow in the sewer, was admitted through a preset valve past a jet through which a concentrated salt solution was injected at a constant rate into the upstream end of the 3 in. pipe; secondly, the fresh-water storm discharge passed through a cam-operated butterfly valve and entered the pipe at the same point. A quick-opening cylinder valve prevented contamination of the downstream end of the storm-water supply before the start of a wave.

Storm waves of trapezoidal shape had been used in the previous work, and this shape was retained for these experiments. The storm discharge increased uniformly for $5/11$ of the wave duration, remained constant for $1/11$, and then fell uniformly for a further $5/11$. The durations chosen for the waves in these tests were 1, 2, 3 and 4 min, representing short, high-intensity storms in the hypothetical prototype.

The model overflows were installed in turn at the downstream end of the pipe. Here two galvanized tanks were used to collect the flows discharged from the structures. These were fitted with water-level recorders, from which a continuous record of the volume of water in each tank, and hence of the discharges entering them, could be obtained. Water passing 'to treatment' was piped into the smaller

tank, while that which spilled was caught in a wooden trough and led into the larger.

The salinities of the base flow, the spilled water and water passed to treatment were measured by previously calibrated electrical conductivity meters. Suspended and floating materials were introduced into the pipe by an electrically driven helical screw-feed injector at a constant rate. The heavier material, representing grit, moved very slowly under baseflow conditions at the 1:500 slope, and was therefore put into the pipe by hand at the start of a run, through holes in the four upstream perspex sections. For the 1:100 tests, when it moved more easily, it was injected by hand throughout each run at the upstream end of the pipe. To ensure that the bakelite particles did not float it was necessary to wet them before injection; thus it was impossible to use the mechanical injector. The solid particles were collected beyond the overflow by passing both flows through sieves before they entered the gauging tanks.

SEDIMENT SIMULATION

In calculating the sizes and specific gravities of particles to represent grit, and floating and suspended solids, the model scale was assumed to be 1:12.

Grit. Using Shields' criterion⁴ for initial movement of bed load, it was found that the largest size of material which would move under the assumed dry weather discharge in a 3 ft pipe at a gradient of 1:500, was 1.5 mm dia. with a specific gravity of 2.65. To simulate this at the model scale, calculation showed that a material with a

specific gravity of 1.45 and a diameter of 0.5 mm was required. Bakelite particles of this size with a specific gravity of about 1.42 were available and were therefore used. It should be noted, however, that the scale relationships for similarity of bed movement and movement in suspension are different. Hence, although the bakelite particles used represented sand of about 1.5 mm dia. when moving along the bed, they behaved as 0.6 mm sand when in suspension.

Coarse suspended solids. In the absence of more reliable information, the size and specific gravity of the coarser fraction of solids to be represented were assumed to be 1 in. dia. and 1.005 respectively. The fall velocity of this material was calculated⁵ to be 0.2 ft/s, and the fall velocity required in the model was therefore $1/\sqrt{12}$ times this value, i.e. 0.058 ft/s. Polystyrene particles with a diameter of about 1.25 mm and a fall velocity of 0.065 ft/s, were used.

Floating solids. It was assumed that the floating solids to be represented had a specific gravity of 0.995 and a diameter of 1 in., giving them a rise velocity of 0.2 ft/s. Polythene particles 2 mm in dia. were used to simulate these solids, and their calculated rise velocity was 0.07 ft/s. This was sufficiently close to the desired value of 0.058 ft/s.

At 1:100 the dry weather flow could move material of a considerably larger size than it would at 1:500. The same size bakelite was used at both slopes, however, as it was felt that the prototype equivalent was representative of gritty material likely to be present in combined sewer systems.

It was found that both the polythene and polystyrene particles were difficult to wet in their original condition,

even if detergent was added to the water. However treatment with concentrated sulphuric acid improved the wetting properties of the polystyrene, while five days' immersion in chromic acid considerably reduced the tendency of the polythene to cling to the water surface.

DESIGN OF OVERFLOWS

General

It was decided to test the overflows with a base flow at a proportional depth* of 0.1, giving a proportional discharge* of about 0.02. The dry weather flow was thus 1/50 of the pipe-full flow, a reasonable value for a sewer, and the peak storm discharge of the input wave was made equal to pipe-full discharge in all cases.

In the design of the overflow structures, the assumed requirements were that first spill should be when the discharge to treatment was 5 times dwf (dry weather flow) and that the flow to treatment should be limited to 6 dwf at maximum inflow. The discharges considered are set

*The proportional depth is the depth of flow expressed as a proportion of the pipe diameter. The proportional discharge is the discharge expressed as a proportion of the pipe-full discharge. Graphs and tables giving the pipe-full discharges and relating proportional depths and discharges from different pipe slopes and roughnesses are given by Ackers 6,7.

out below:

	<u>1:100</u> (cusecs)	<u>1:500</u> (cusecs)
Dry weather flow	0.00245	0.00101
Discharge to treatment at first spill (5 dwf)	0.01225	0.00505
Discharge to treatment at peak incoming discharge (6 dwf)	0.01470	0.00606
Peak incoming discharge (50 dwf)	0.12250	0.0505
Peak spill over weirs (44 dwf)	0.10780	0.0444

The models were intended to be typical examples of overflows currently in use and are not necessarily the best that could have been designed. They were envisaged as 1:12 scale models of structures to be installed on a 3 ft sewer. It was considered desirable that the incoming pipe should never be surcharged, and all the overflows were designed to give a maximum water level below the soffit. Although it was known that the waves introduced at the upstream end of the pipe would attenuate by varying amounts, depending on their duration and on the pipe slope, the maximum discharge at the upstream end was used for design purposes in all cases.

The round-crested type was chosen as being the most convenient type of high coefficient overflow weir for these structures. The discharge coefficient varies with the head over the weir for a given crest radius⁸, reaching a value of about $3.93 \text{ ft}^{\frac{1}{2}}/\text{s}$ at a head/radius ratio of 1.5. In the design calculations, therefore, it was assumed that this coefficient would operate when the discharge over the weir was a maximum. When the corresponding head had been calculated, the radius of curvature was selected to ensure that this would be so.

Scumboards were designed for each overflow and were installed just upstream of the weirs. In each case their distance from the upstream face of the weir was equal to the maximum design head over the weir; the scumboard extended 0.1 times the depth of water below the weir crest and, generally, 1.25 times the maximum head above it. At the steeper slope the tops of the scumboards in the vortex and storage overflows had to be raised to prevent overtopping. In the vortex overflow this was because the depth was considerably greater than the design figure, and in the storage overflow because of a hydraulic jump which moved into the overflow section.

Positive control of flow to treatment in the stilling pond, vortex and storage types of overflow was achieved by using streamlined rectangular orifices. Such an orifice, with contractions fully suppressed, has a discharge coefficient of about 0.95, and facilitated design because its area and height could be varied independently. Other control arrangements, for example a long throttle pipe, are equally applicable. If an alternative control arrangement had been used, it would not have affected the results, provided its discharge characteristics matched those of the orifice actually employed.

Low side-weir

From a study of some existing side-weir structures a design was evolved in which most of the dimensions were related to the diameter of the incoming pipe. The length of the overflow and the weirs was taken as ten times the inlet diameter (30 in. in the model). The diameter of the outgoing pipe to treatment was taken as half that of the incoming one. Because the overflow was to be fitted with scumboards, the width of the channel between the weirs was made 0.5 in. greater than the pipe diameter at each end of the overflow. Thus the trough tapered from 3.5 in.

wide at the upstream end to 2 in. at the downstream end. The slope of the overflow invert and weirs, and of the outlet pipe, was made the same as that of the 3 in. pipe upstream. In determining the height of the weirs, the usual but incorrect assumption was made that the depth of flow in the trough before spill would be the same as that in the 1½ in. pipe downstream under normal flow conditions. A depth of 0.855 in. was found to correspond to the design discharge of 0.00505 cusecs at first spill and the weirs were installed with their crests at this height above the overflow invert.

To determine the maximum head on the weirs, it was assumed that at maximum discharge the surface elevation in the trough would be greatest at the upstream end and that the level would then be half-way between that of the weir crest and the specific energy level upstream¹⁰. This gave a head over the weir of 0.767 in. and a weir crest radius of 0.51 in., rounded down to 0.5 in. for ease of construction. The whole structure, shown in Plate 1(a) and Fig. 2, was made from wood. Because the same proportional depths and discharges would be obtained at both pipe slopes this structure was suitable for use at either slope without modification.

Stilling pond

The stilling pond was similar to one designed for a scheme at Farnworth, Lancashire, where, as is common in many sewerage systems, only a small fall was available between the incoming combined sewer and the outgoing pipe to treatment. This meant that the velocity of flow had to be reduced by increasing the width of the chamber rather than the depth. In theory this is a better way of settling solids, providing that a uniform velocity distribution across the chamber can be obtained, since it is principally the surface area of the pond in relation to discharge and settling velocity that determines its

efficiency. Unlike the low side-weir, the stilling pond was not suitable for use at both pipe slopes unless the orifice was modified. Because of the larger discharges obtained at a slope of 1:100 the design for operation at this slope was undertaken first. The orifice invert was fixed at 1 in. below the invert of the incoming pipe to provide a fall through the chamber and to avoid surcharging the incoming pipe; the maximum water level was fixed at 3.5 in. above this level. The orifice had to pass 6 dwf at this condition: an orifice width of 0.75 in. was chosen, and the corresponding orifice height was calculated to be 0.775 in., giving a nearly square section. The weir height to give first spill at the required discharge was determined to be 1.67 in. Using this height, the length of weir necessary to spill 44 dwf with a head of 0.83 in. (3.50 - 2.67 in.) was calculated to be 18 in. The weir crest radius in this case was $0.83/1.5 = 0.55$ in. For the reduced discharges with a pipe slope of 1:500, the height of the orifice required was 0.3 in. to give the correct first spill discharge: the discharge to treatment then arose to 5.5 dwf at maximum inflow. The orifice was therefore fitted with an adjustable soffit formed of a curved brass plate. The proportions of the chamber were the same as those of the structure on which it was modelled, the floor and overflow weir being made of wood and the walls of perspex. The overflow is shown in Plate 1(b) and Fig. 3.

Because the stilling pond overflow was intended to trap the coarse solids, its efficiency in this respect was estimated. The mean cross-sectional area and the average forward velocity were calculated for the maximum discharge and, using the fall velocity calculated for the polystyrene particles, the efficiency of settling was found from Camp's sediment-removal function¹¹. The calculation was repeated for the polythene particles and in both cases the length of the chamber was found to be just sufficient to give 100% removal in theory.

Vortex with central spill

The vortex overflow was based on a type now in use at Bristol. The analysis used for the design of this structure was similar to that for the stilling pond, except that the orifice invert had to be set lower (1.5 in. below the incoming pipe invert) so that the pipe to treatment could pass under the spiral dry weather flow channel. The calculations for a pipe slope of 1:100 were again carried out first, and the chosen orifice width of 0.75 in. gave an orifice height of 0.71 in., a weir height above the orifice invert of 2.99 in., a weir length of 13.4 in. and a crest radius of 0.675 in. In this case a central circular overflow weir was used, the inside diameter required being 2.92 in. A check was made to confirm that this would not choke at maximum spill.

It was not convenient to provide an adjustable orifice for this overflow, so two interchangeable units were provided, that for controlling flow in the 1:500 experiments being made 0.5 in. square, giving first spill at 5 dwf while regulating the maximum flow to treatment to about 5.5 dwf. The structure is shown in Plate 1(c) and Fig. 4.

Storage overflow

A large part of the pollution in storm water spilled by overflows is reputedly accounted for by the spilling of the first foul flush at the start of the storm wave. The design of the storage-type overflow was based on the containment of this flush in a chamber downstream of the spill weirs. The structure consisted of a long rectangular storage chamber, with an orifice at the downstream end controlling flow to treatment, and a high side-weir overflow at the upstream end. This is similar to one designed at the Hydraulics Research Station for installation at Gillingham¹². The operation of weir and orifice are independent of the storage volume and simply provide the

same control as in the two previous designs.

Conservatively, it might have been assumed that it would be necessary to provide storage for the whole of the volume of base flow upstream of the overflow site at the start of the storm. However, there were two factors which diminished the storage volume required. The first factor concerned the flow in the pipe. The assumption that a volume equal to the whole of the base flow in the pipe should be stored would imply that the storm wave travels at an infinite speed through the pipe, entraining all the foul flow in it, but this is clearly not so. The wave travels at a finite speed, about 1.4 times the pipe-full velocity¹³, v_f , and the base flow also continues to flow out of the downstream end to treatment at a velocity of $0.4 v_f$ until the wave reaches the overflow. The foul flow which enters the sewer at the same time as the storm water is assumed to be thoroughly mixed with it and is not regarded as part of the first foul flush. Thus, if the pipe upstream of the overflow has a length L , the wave will traverse the pipe in a time $T = L/1.4v_f$. If the volume of foul flow in it is V , then assuming a uniform cross-sectional area of V/L , the rate of outflow to treatment is $0.4 v_f V/L$. In a time T a volume of $0.4 V/1.4$ will flow out and the volume remaining to be stored is $0.715 V$.

Secondly, consideration of the behaviour of the storage chamber showed that this revised volume could be further reduced because the flow to treatment through the orifice continually increases as the level rises, disposing of some of the first flush before spill starts. This second reduction was obtained from a solution of the differential equation relating the effective storage volume, the rate of rise of discharge at the front of the wave, the geometry of the storage chamber, the base flow discharge, and the first spill discharge. The procedure has been

used in the design of an actual structure¹².

For the 1:500 pipe slope the actual storage volume required after taking these factors into account was 0.467 ft³. The storage chamber was made 96 in. long with a width of 5 in., the invert falling 0.43 in. from inlet to outlet. The dimensions calculated for the orifice were: width 0.5 in., height 0.47 in.; the overflow weirs, 8 in. long, were set 2.47 in. above the orifice invert, with a radius of curvature of 0.5 in. The structure is shown in Plate 1(d) and Fig. 5. For experiments at a pipe slope of 1:100 it was necessary to use longer overflow weirs, 24 in. in total length, and the orifice size was increased to 0.75 in. wide by 0.835 in. high. Although discharges were higher at this slope the depth of dry weather flow was the same as at 1:500. Consequently the volume of base flow in the pipe at the start of the wave, and hence the volume to be stored, was the same.

Vortex with peripheral spill

While the other structures were being tested at a pipe slope of 1:100 the idea of using a vortex drop¹⁴, with a peripheral weir to spill stormwater, was conceived. It was thought that this arrangement might have some advantages over the other types, so a design to give the hydraulic control required was proposed. The characteristics of the vortex drop are such that in the absence of spill the discharge down the central pipe is very nearly proportional to the head in the chamber. Increasing the circulation, i.e. the velocity at entry or the diameter of the chamber, decreases the discharge or, for the same discharge increases the head required. In this type of structure, therefore, the depth in the chamber will first rise in proportion to the incoming discharge, until the weir level is reached. After spill has started, the level in the chamber is controlled by the weir and can rise very little. As the discharge increases still

further, however, it increases the circulation until, at an incoming discharge of about double that at first spill, the flow down the shaft to treatment should in theory stop altogether. Because of these rather unusual discharge characteristics, with the maximum flow to treatment at first spill, it was thought reasonable to arrange that the setting of the overflow should be double the design mean setting of the others and the dimensions were chosen to give first spill at 11 dwf.

The structure was of approximately the same size as the vortex overflow with central spill. The diameter of the chamber was about 12 in., with an outer channel to catch the spill. The central shaft had a diameter of 2.1 in. and the weir, of height 2.33 in., had a round crest of radius 0.4 in. The structure can be seen in Plate 2 and Fig. 6.

PROGRAMME OF TESTS

The tests were divided into two categories: (a) those investigating the discharge of dissolved pollution, and (b) those studying the behaviour of bed load and suspended and floating solids. Each structure was first tested with saline base flow and fresh-water storm waves of 1, 2, 3 and 4 min duration. Runs were duplicated to check repeatability, and in some cases additional tests were made to ensure that the scatter obtained was inherent in the performance of the structure and not due to inaccuracies in measurement. Tests with sediment were then carried out for the same wave durations, again with the same checks on repeatability. Floating solids were studied both with and without scumboards.

In a typical salt test, a sample of the base flow was taken when flow had become steady, and its salinity and that of the water in the two collecting tanks was measured and recorded. The water-level recorders were started, and the base flow was diverted into the appropriate measuring tank. A flood wave of the required duration was then injected at the upstream end of the pipe and, after passing down the pipe and through the overflow, some water spilled into the 'overflow' tank, the rest passing on to the 'treatment' tank. When the base flow discharge, estimated from the slope of the trace on the recorder, and the salinity, measured from occasional samples after the passage of the wave, had returned to their initial values, the recorders were stopped and the salinity of the water in both tanks was measured.

From the recorder charts it was possible to calculate quickly the volume of the storm wave. If this was not within 5% of the theoretical volume injected, the test results were rejected. This indicated faulty setting or operation of the butterfly valve controls, or a fault in the water-level recorders, and a check was carried out before repeating the test.

In the 1:500 slope experiments, sediment tests were carried out with the sediment injector delivering at a rate of about 5 particles/s without salt in the base flow. The two flows were passed through sieves into their respective gauging tanks, the base flow being collected for 5 min after the discharge had returned to its original value.

When the pipe slope was changed to 1:100 a larger butterfly valve was installed to provide the increased flood wave discharges. At this slope the rate of injection of polythene and polystyrene was increased to about 20 particles/s.

During all the sediment tests the salt supply apparatus was used to inject a solution of detergent into the base flow. This ensured the complete wetting of the suspended and floating particles.

In the steady flow experiments conducted at Luton the hydraulic performance of each structure could be measured and observed at any steady discharge. In the experiments reported here, however, the discharge varied with the passage of the wave, but its instantaneous value could be obtained from the slope of the curves on the water-level record. The scale used did not, however, permit the first spill discharge to be evaluated accurately.

ANALYSIS OF RESULTS

Discharge

The recorder charts were analysed for each structure and the duration of each test in turn, to obtain the discharge/time curves shown in Figs. 7 and 8. In each case the upper line shows the total discharge from the overflow, the lower the discharge passed to treatment. The accuracy of the control of discharge to treatment is illustrated by the plateau on the 'discharge-to-treatment' curve: where this is level the control is good. The differences between the total discharge curves for the various overflows at a given wave duration can be accounted for partly by differences in the storage volumes of the overflows and partly by variations in the storage characteristics of the pipe upstream created by the different stage/discharge relationships of the structures.

The discharge curves were analysed to see if the attenuation of the peaks of the waves agreed with that observed for shorter waves in the previous experiments with the same apparatus¹. An expression had been derived¹⁵ to relate the peak wave height, h , to the wave volume, V_1 , the pipe-full discharge, Q_f , the pipe diameter, D , the distance, x , and the base flow depth, h_0 :

$$\frac{h}{D} - \frac{h_0}{D} = \left\{ \frac{1}{2 + x \sqrt{gD^9/15V_1 Q_f}} \right\} 1 + 1.5 h_0/D$$

Assuming for these waves that at peak discharge flow was at normal depth, the maximum discharge at the overflows during the 1:500 tests should have been 0.030 cusecs for a 4 min wave, 0.025 cusecs for a 3 min wave, and 0.019 cusecs for a 2 min wave. These compare satisfactorily with the recorded average values of 0.033, 0.027 and 0.017 cusecs respectively. At 1:100 similar calculations gave values for the maximum discharges at the overflows for 4, 3 2 and 1 min waves of 0.112, 0.109, 0.103 and 0.088 cusecs, compared with the average recorded values of 0.109, 0.108, 0.101 and 0.084 cusecs.

The volume of water spilled by each structure was measured from the recorder charts and is plotted as a proportion of the wave volume in Figs. 9 and 10.

At 1:500 the low double sideweir was the only structure to spill a 1 min wave. The poor control of the flow to treatment exercised by this type of structure has been confirmed by the larger scale experiments at Luton. With longer waves it passed more flow to treatment than did the three with orifice outlet control.

At this slope the stilling pond and vortex overflow, as expected, spilled nearly the same amount as each other, while the storage overflow spilled less by about 0.5 ft³.,

approximately the volume of storage provided. The control with the vortex overflow was not quite as good as with the stilling pond.

At 1:100 the poor control of the low double sideweir was again evident and it spilled less than the other structures (except the peripheral spill vortex with scum-board) at all wave durations. With the higher velocities obtained at this slope the effect of circulation in restricting the discharge over the weir of the vortex overflow with central spill increased and the discharge to treatment rose.

The stilling pond and storage overflows spilled nearly the same volumes at the steeper slope, the difference being considerable less than the volume of storage provided. This could be explained by a relatively small error in the size or setting of one of the orifices.

The vortex overflow with peripheral spill, in spite of its very different discharge characteristics (see Fig. 8), spilled almost exactly the same volume as the stilling pond overflow, thus justifying the rather arbitrary choice of setting of 11 dwf for these test conditions. The installation of a scumboard, however, reduced the circulation in the chamber and it then spilled about the same proportion of the wave volume as the sideweir overflow.

Pollution

(a) General During each test the quantities of pollutant passing to treatment and to spill were measured, and the proportion of the total which passed to spill was calculated. The values for each overflow and wave duration are plotted in Figs. 11(a)-(e) and 12(a)-(e) for salt, polystyrene, bakelite, polythene, and polythene with scumboards, respectively.

The results from the pollution analysis are expressed in a different way in Figs. 13 and 14 in which the average

concentration of each pollutant in the spill is divided by the concentration in the base flow. A value of unity for this ratio would indicate that undiluted base flow had been spilled, while a value of 0.036 would result if no first foul flush had occurred and the pollutant had become uniformly mixed with the storm wave at entry, with no subsequent separation.

In Figs. 13 and 14, the curves showing the concentrations of pollutants lie in approximately the same relative positions for a given wave duration as in Figs. 11 and 12, but the results are now independent of the duration of sampling.

(b) Sub-critical slope At 1:500, the side-weir, stilling pond and vortex overflows gave similar average salt concentration in the spilled water. The storage overflow, however, spilled at a much lower concentration, only slightly greater than the average of 0.036 given by uniform mixing. The whole of the first foul flush of dissolved pollution had therefore been passed to treatment.

In the case of the 4 min wave with the low side-weir overflow at 1:500, an attempt was made to predict the pollution of the spill from the discharge curves, using the calculated volume of the first foul flush of 0.51 ft^3 . The assumption was made that the first foul flush did not mix longitudinally with the following storm flow, and that the subsequent dilution was constant and equal to that obtained by mixing the whole of the incoming storm wave thoroughly with the base flow entering at the same time. It was found from the discharge curve that the foul flush lasted until about 32 s after the arrival of the wave at the overflow (when the flow was 0.85 times its maximum value) and that during this time a volume of 0.261 ft^3 would be spilled at an assumed concentration of unity. The addition of the remaining volume of 3.17 ft^3 overflowed at an assumed concentration of 0.036 gave a

predicted concentration of base flow in spill of 0.109. The average measured value was 0.110.

From Figs. 11 and 13 it can be seen that in the 1:500 tests the overflows with a free surface path past the weirs (the side-weir and storage types) were the most satisfactory with floating solids. However the installation of scumboards considerably improved the performance of all but the vortex overflow, in which strong turbulence carried the polythene under the scumboard. This turbulence originated where the incoming flow joined the flow circulating in the chamber. The stilling pond overflow performed well with a scumboard because, under the conditions tested, floating material was stored behind it for discharge to treatment when the flow subsequently reduced.

These figures also show that the low side-weir spilled the greatest proportion of bakelite and polystyrene at 1:500. The vortex and storage structures spilled somewhat less polystyrene and considerably less bakelite. The stilling pond dealt most effectively with both these types of solids.

(c) Super-critical slope Figs. 12(a) and 14(a) show that at the 1:100 slope, with dissolved pollution, the vortex overflow with central spill had the poorest performance and the peripheral spill vortex without scumboard the next poorest. There was not a great deal to choose between the other types, although at the shortest wave duration the storage overflow was the most effective.

The time delay between the arrival of the wave at the overflow and the start of spill was very short in the vortex with central spill. The overall concentration in the spill may be calculated from the known volume of flush, excluding the proportion that passes to treatment in this period, plus a uniform mixture of stormwater and base-flow, at a concentration of 0.036. Values approximating to

those shown in Fig. 14(a) are then obtained. The delay in the case of the sideweir, however, was nil at this slope, so that there must be some different explanation for its fairly good performance. This is found in its large initial discharge to treatment. Although it may spill prematurely, it spills only a relatively small amount of the first foul flush because of its poor control of flow to treatment.

In the peripheral overflow the effect of the higher velocities at the front of the storm wave was to decrease the flow to treatment up to first spill and hence the anticipated benefit of a high initial setting was lost. Nevertheless, the pollution spilled was less than that from the vortex overflow with central spill. A possible explanation of this rather surprising result is that the volume of the peripheral overflow was greater than that of the central spill overflow and also that it could allow diluted water to spill at the perimeter before the foul flush had been completely discharged.

The performance of the stilling pond was about the same as that of the sideweir, but for different reasons. An appreciable delay occurred before spill began, as a result of the greater volume. This enabled part of the first flush to pass to treatment in spite of the good control exercised by the orifice outlet.

In the storage overflow the beneficial effect of storage is only apparent in the results for the 1 min and, to some extent, the 2 min waves. The steep front of the wave was reflected from the downstream end of the overflow and travelled back up the storage chamber as a hydraulic jump, until it reached the weir section. With the longer waves spill started very nearly as quickly as it had done in the stilling pond and this, taken with the fact that some of the water which was spilled had been brought back to the chamber entrance with the reflected wave is

is sufficient to explain the difference in performance between the tests at this slope and at 1:500.

It should be remembered, however, that the theoretical minimum pollution of spill by salt is represented by a concentration of 0.036 in Figs. 13(a) and 14(a), a figure which is approached by all except the vortex overflow for the 1:100 slope at the longest wave duration.

With floating solids (polythene) the sideweir and storage overflows again performed best at 1:100. The installation of scumboards improved the performance of all except the storage overflow but the vortex with central spill and stilling pond overflows were still the least satisfactory.

The best performance with coarse suspended solids (polystyrene) was obtained with the storage overflow, there being little to choose between the other four types. With bakelite, as might have been expected, the vortex overflow with peripheral spill was the best. The vortex with central spill was perhaps the most efficient of the other types, although there was really very little difference in their performance.

CONCLUSIONS

These experiments under non-steady flow conditions provide a comparison of four types of overflow structure with respect to their hydraulic characteristics and their handling of pollutants. They highlight the importance of the first foul flush.

(a) Subcritical conditions, pipe slope 1:500

1. The stilling pond and storage overflows with orifice outlets provided good control of the flow to treatment. The vortex overflow tested did not provide such an accurate limitation of flow to treatment and other experiments suggest that this was because proper allowance had not been made in the design for the effects of circulation. The control of discharge to treatment by the side-weir overflow was the least satisfactory; it spilled prematurely but passed too much to treatment at high flows.
2. The three structures without special provision for storage gave very similar results in terms of dissolved pollution. The storage overflow was much the best in this respect.
3. The side-weir, storage and stilling pond overflows were better than the vortex overflow in handling floating material, providing that they were fitted with scumboards. Without scumboards, the storage overflow gave the best results, with the side-weir marginally better than the other two types.
4. With suspended and heavy solids, the behaviour seemed to depend on the settling efficiency and the velocity of flow near the overflow weirs. The two structures using side-weir overflows, the low side-weir in particular and the storage overflow to a lesser extent, were worse than the stilling pond. Surprisingly, the storage overflow did not come out as well as expected in this respect, perhaps because these solids tended to lag behind the general fluid movement, and hence were not concentrated in the first flush as was the dissolved and floating pollution.
5. The reduction in the pollution of the spill obtained by retaining the first flush in the storage overflow makes this the most efficient structure in terms of the overall pollution spilled. Although it spills slightly less of

the volume of each wave, the additional flow to treatment consists almost entirely of undiluted foul flow.

6. Of the non-storage overflows, the experiments cannot be said to have established conclusively that one of the types tested is better in all respects than the others, although on balance the stilling pond with scumboard is to be preferred.

(b) Supercritical conditions, pipe slope 1:100

7. The best control of discharge was still achieved by the stilling pond and storage overflows. The vortex overflow with central spill was affected even more by the circulation in the chamber and both this and the sideweir again passed too much flow to treatment under supercritical conditions. The vortex overflow with peripheral spill, which was only tested under supercritical conditions, did not control discharge to treatment satisfactorily.

Although it spilled the same volume as the overflows with good control the greater part of the spill occurred at the more highly polluted front of the wave, the maximum flow to treatment occurring after the arrival of the dilute stormwater. This was the reverse of what would be expected under sub-critical conditions.

8. The storage overflow reduced the spill of dissolved pollution only with the two shorter waves. During the longer waves the stilling pond and sideweir gave comparable performances, with the peripheral overflow a little worse. The central spill overflow gave the poorest performance.

9. The storage, sideweir and peripheral spill overflows gave the best performance with floating solids both with and without scumboards.

10. With suspended solids the storage overflow was better than the other four. The polystyrene did not lag behind the wave at this slope, and thus was swept into the storage

chamber as a part of the first foul flush. The stilling pond was marginally the least effective, and this is probably accounted for by the presence of a hydraulic jump at or near the upstream end of the structure throughout most of the period of spill. The turbulence caused made separation by settlement extremely unlikely.

11. The spill of heavy solids did not vary greatly between the original four types of overflow and this may well be due to their increased mobility at this slope. The vortex with peripheral spill dealt better with heavy solids than any of the other structures. The flow of water near the bed of the chamber, and even in the channel upstream, was biased strongly towards the central orifice. Material moving along the bed was therefore almost inevitably passed to treatment.

12. At this slope, with no provision for reducing the velocity of approach to the structures, it appears that the most effective overflow in keeping down the overall spill of pollution was the sideweir, followed very closely by the storage overflow. However, the apparent merit of the sideweir is to a large extent spurious, being due to the fact that it passes too much flow to treatment. The difference between the curves in Figs. 12 and 14 illustrates the effect of this: the "concentration" curves are a better guide to true efficiency than the "proportion" ones.

(c) General

13. In comparison with real sewage, the polluting solids used in these model experiments may well represent only the upper and lower extremes of the range of gross solids which cause objectionable signs of pollution if spilled. However, evidence from other sources indicates that the dissolved and fine suspended matter forms by far the major part of real pollution.

14. One general conclusion which must be emphasised is that, if overflows are to achieve hydraulic separation, they must be preceded by a section of subcritical sewer: they are unlikely to be effective on supercritical sewers.

15. Many sewerage systems are at supercritical gradients, and the research indicates that low side-weirs are the most effective at limiting the overall pollution discharged under such conditions. This does not stem from hydraulic efficiency, but from their characteristic of passing a greater flow to treatment than overflows with efficient throttles.

16. The general performance of the low side-weir seems to be independent of the slope of the system. The other types of overflow deteriorate with increasing slope.

17. Provided a subcritical approach can be arranged, the storage of the first flush beyond the point of overflow has considerable merit. In the absence of storage, the stilling pond is the most effective of the types tested, although not outstandingly so.

18. The designs of overflow considered in this investigation should not be regarded as the best of their respective types. A general increase in size, coupled with detailed modifications, would undoubtedly improve the separating characteristics of all except the side-weir. Some changes in the shape of the vortex overflow could improve the velocity distribution and reduce the secondary currents, increasing the retention of floating and suspended material. The replacement of the high side-weir overflow on the storage structure by the type of weir used in the stilling pond with a low approach velocity could make it, under subcritical conditions, superior in every way to the other types.

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FIGURES

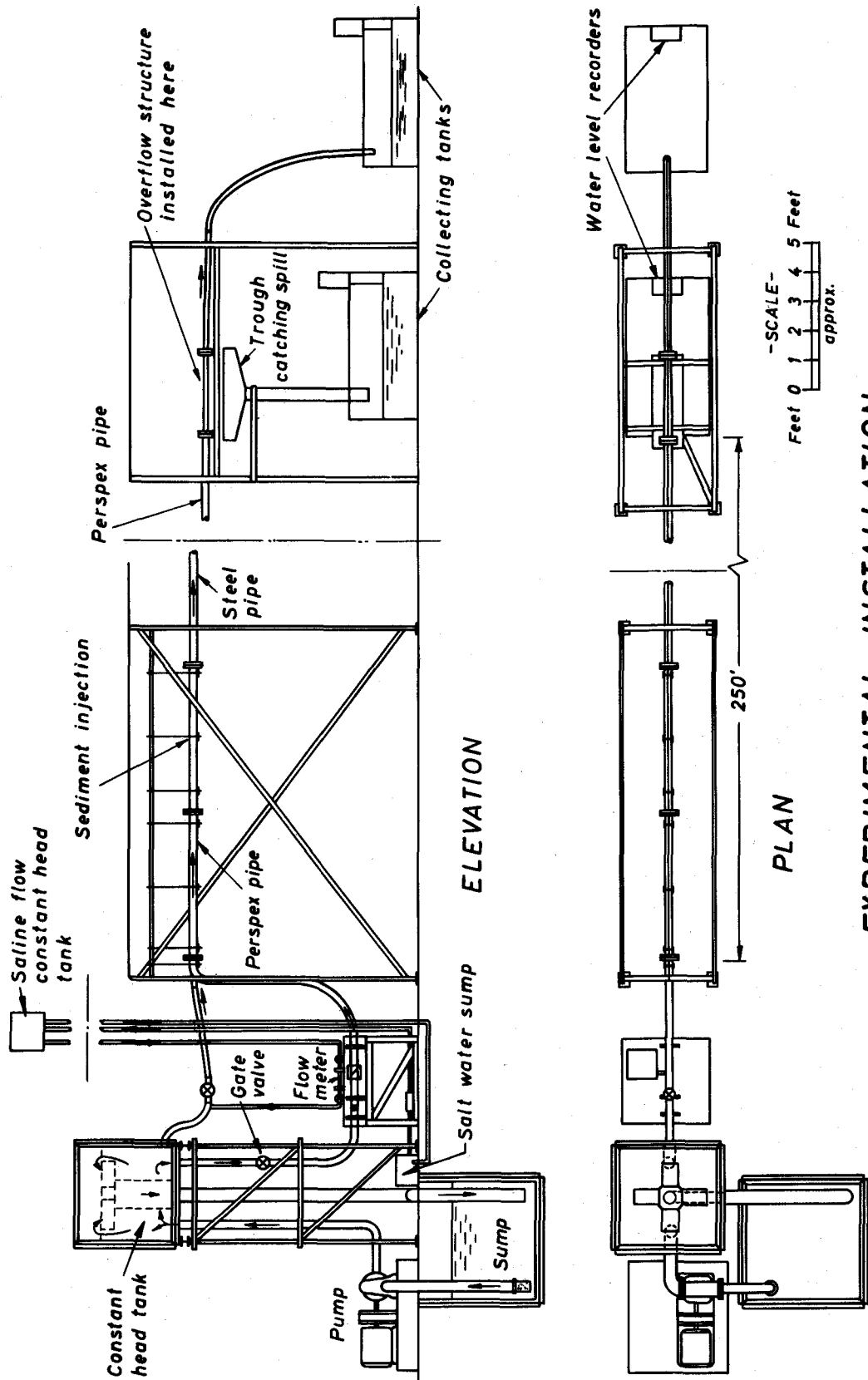
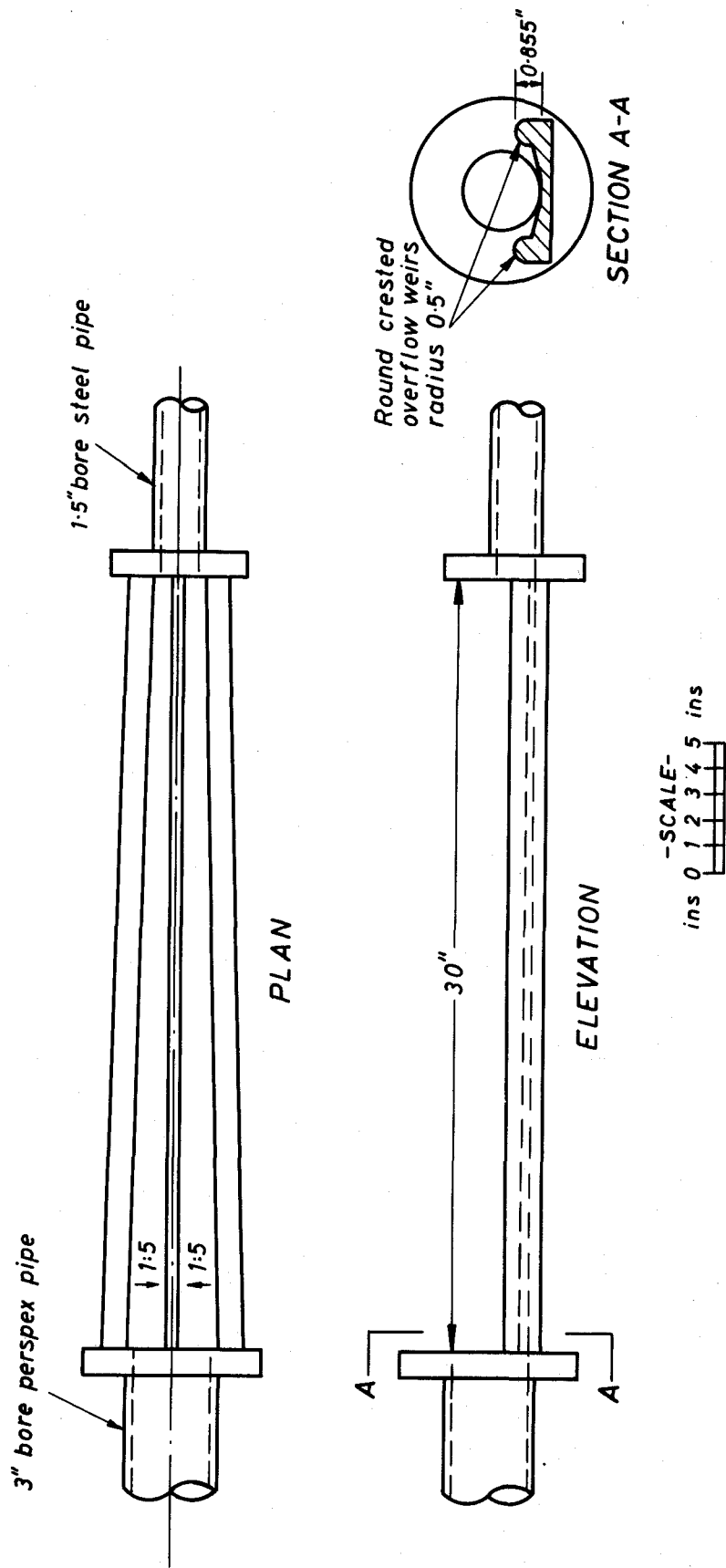


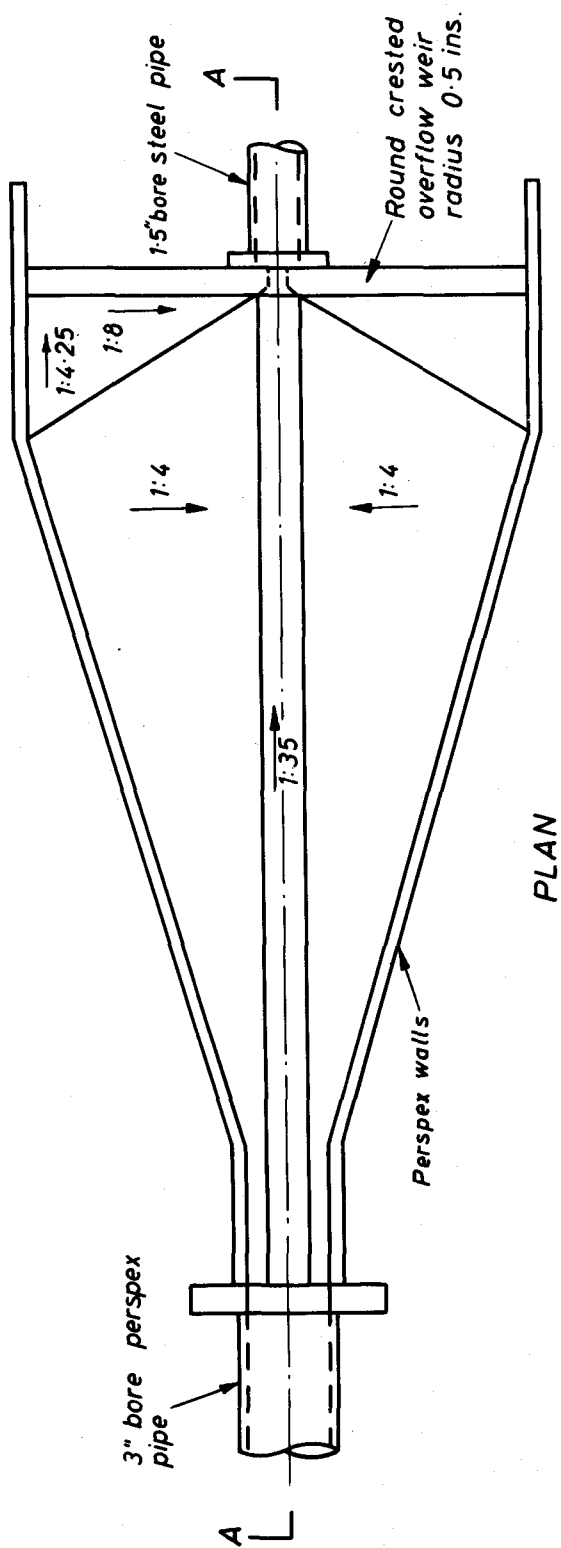
FIG 1

EXPERIMENTAL INSTALLATION

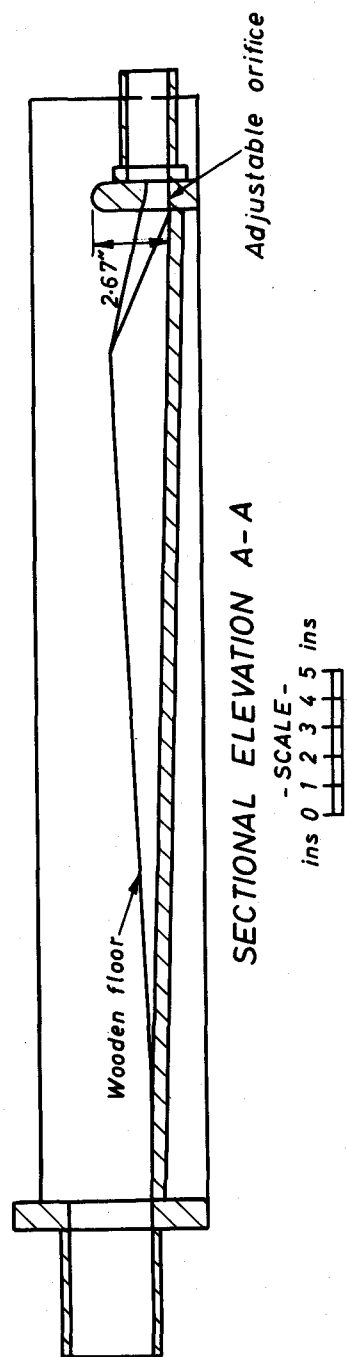


LOW DOUBLE SIDEWEIR OVERFLOW

FIG 2



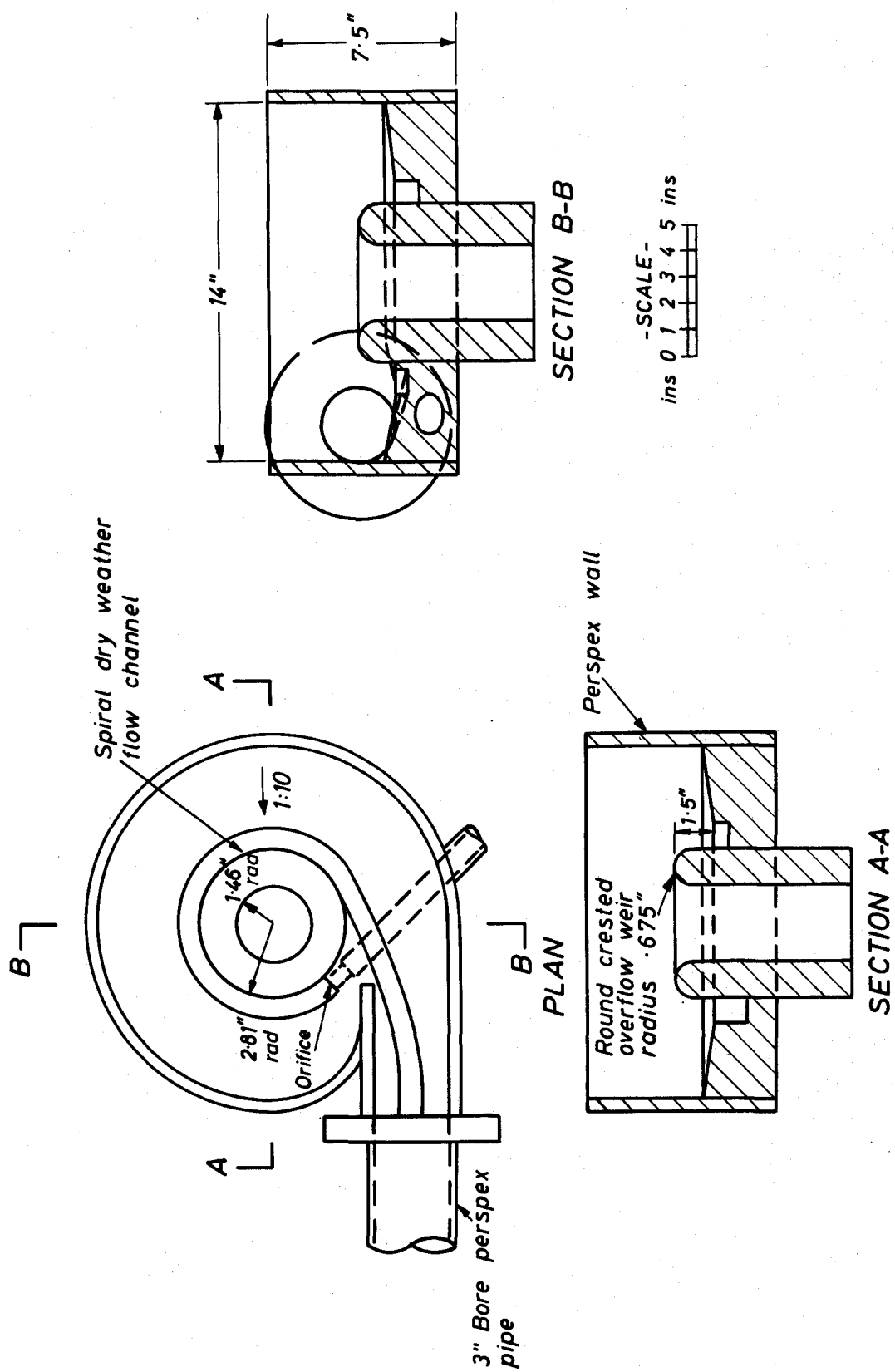
PLAN



SECTIONAL ELEVATION A-A

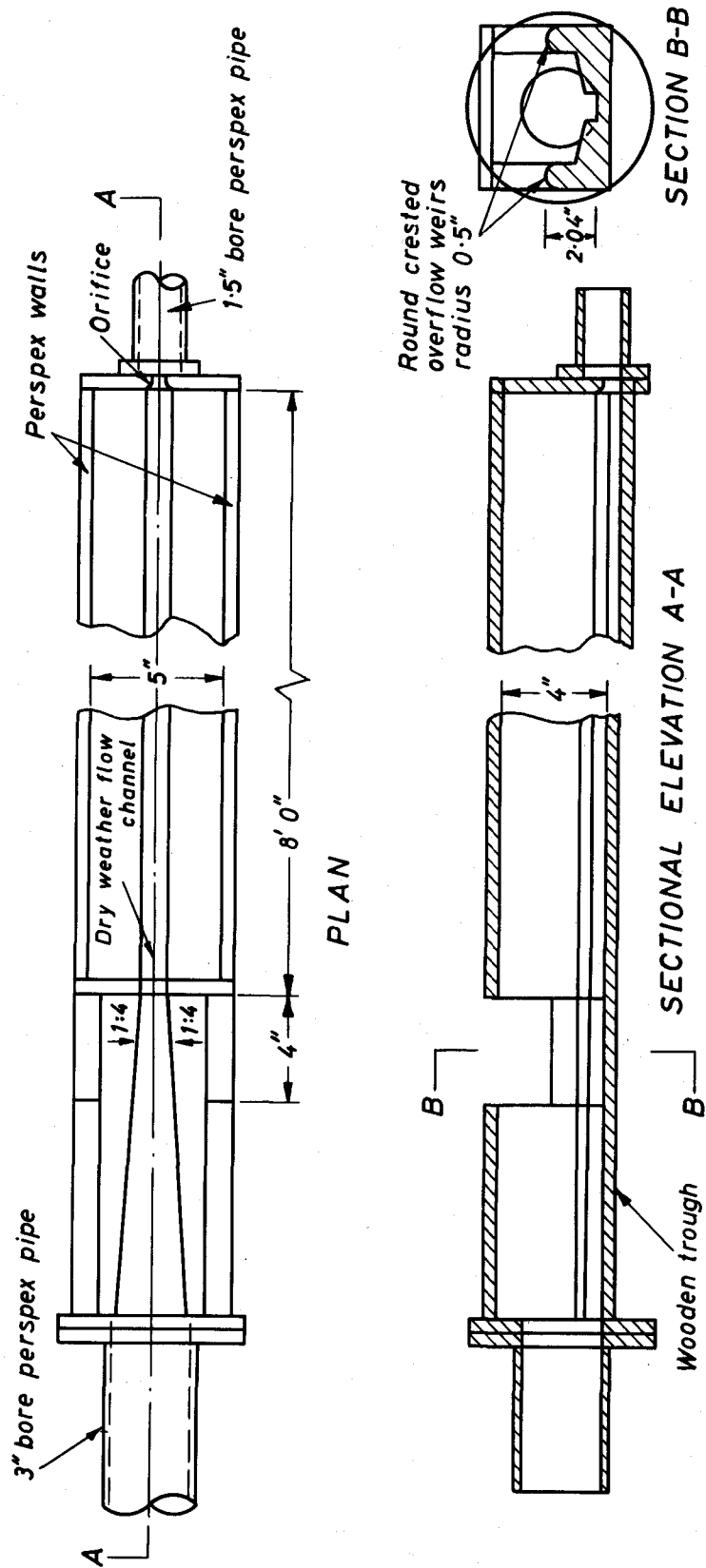
STILLING POND OVERFLOW

FIG 3



VORTEX OVERFLOW

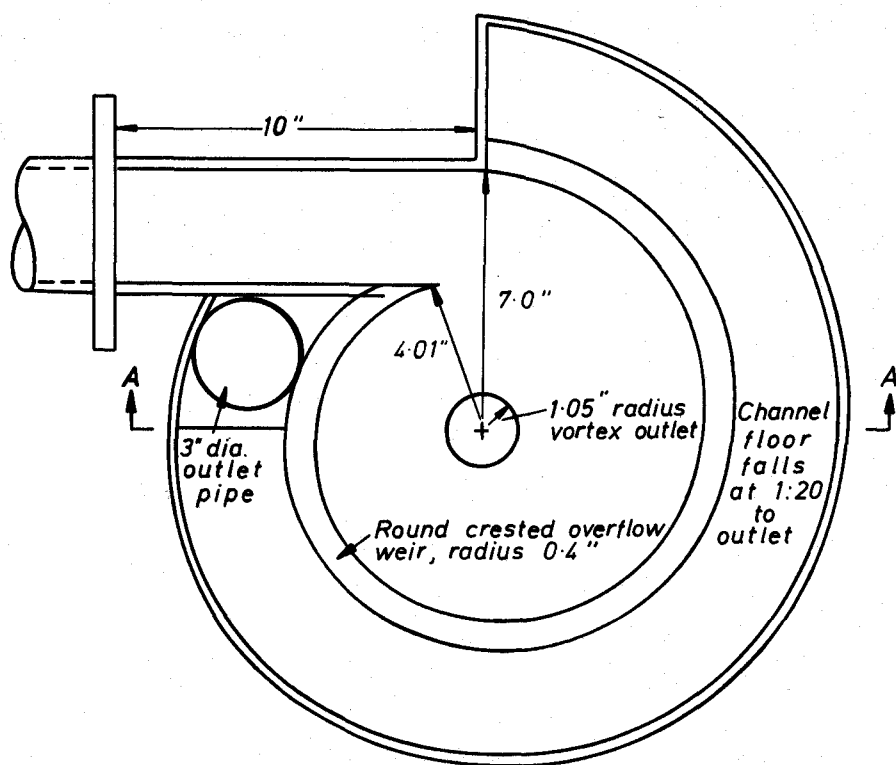
FIG 4



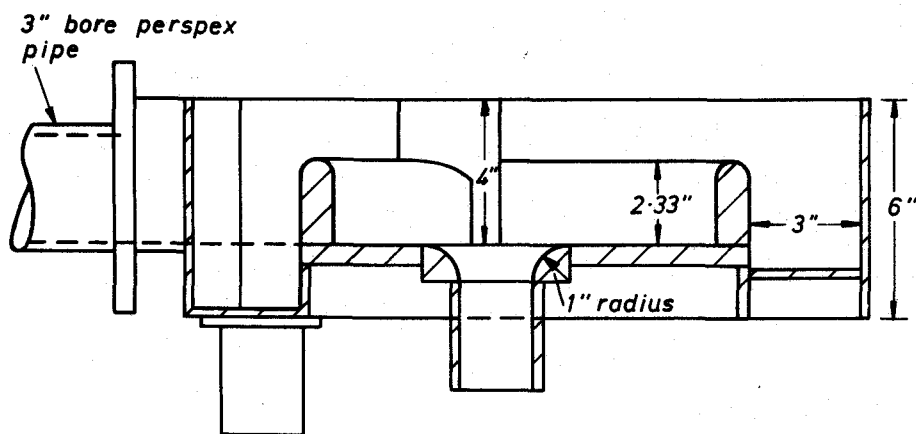
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STORAGE OVERFLOW

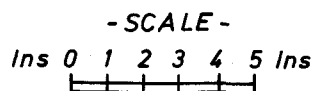
FIG 5



PLAN

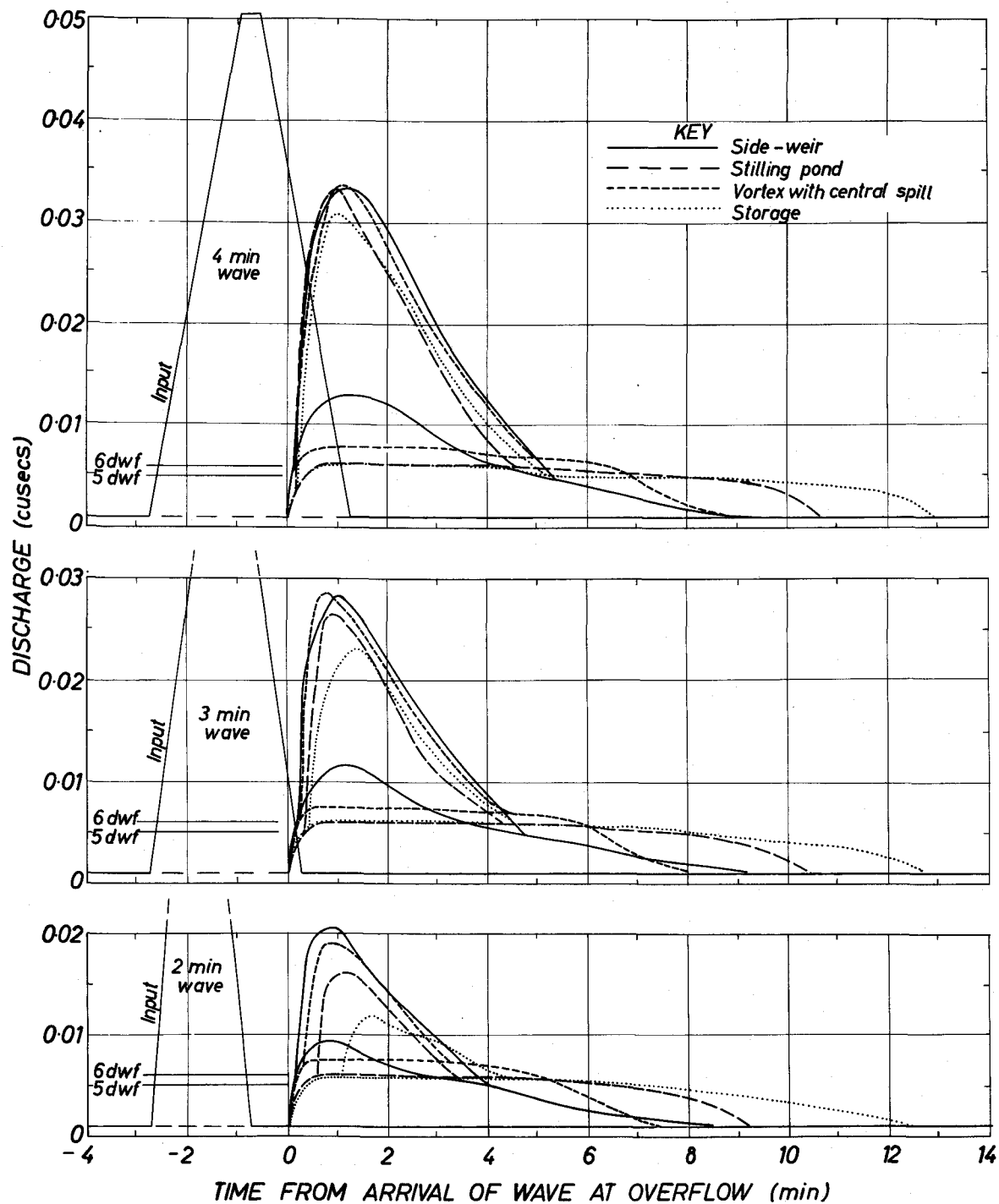


SECTION A-A



VORTEX OVERFLOW WITH PERIPHERAL SPILL

FIG 6



DISCHARGE -TIME CURVES
(PIPE SLOPE 1:500)

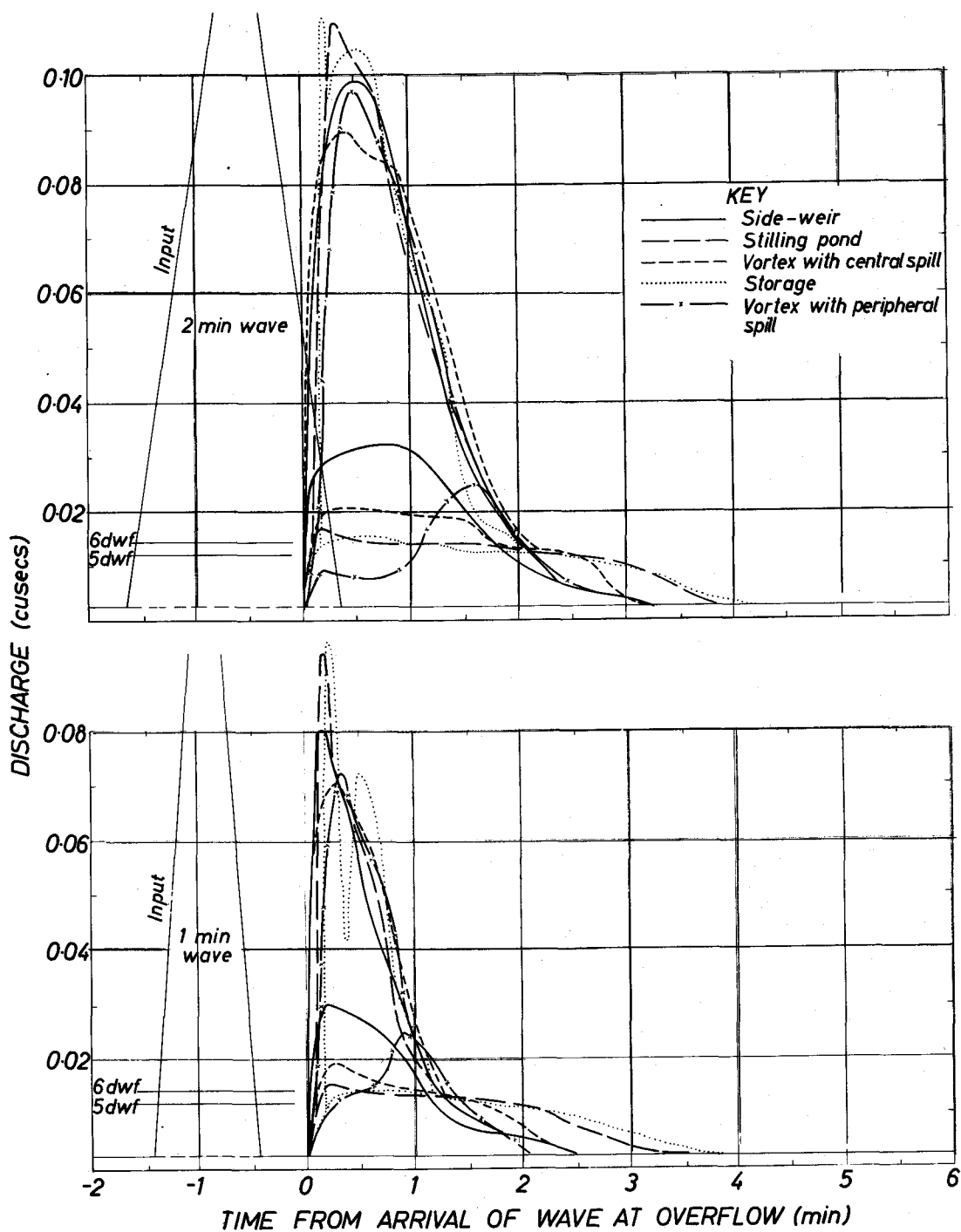
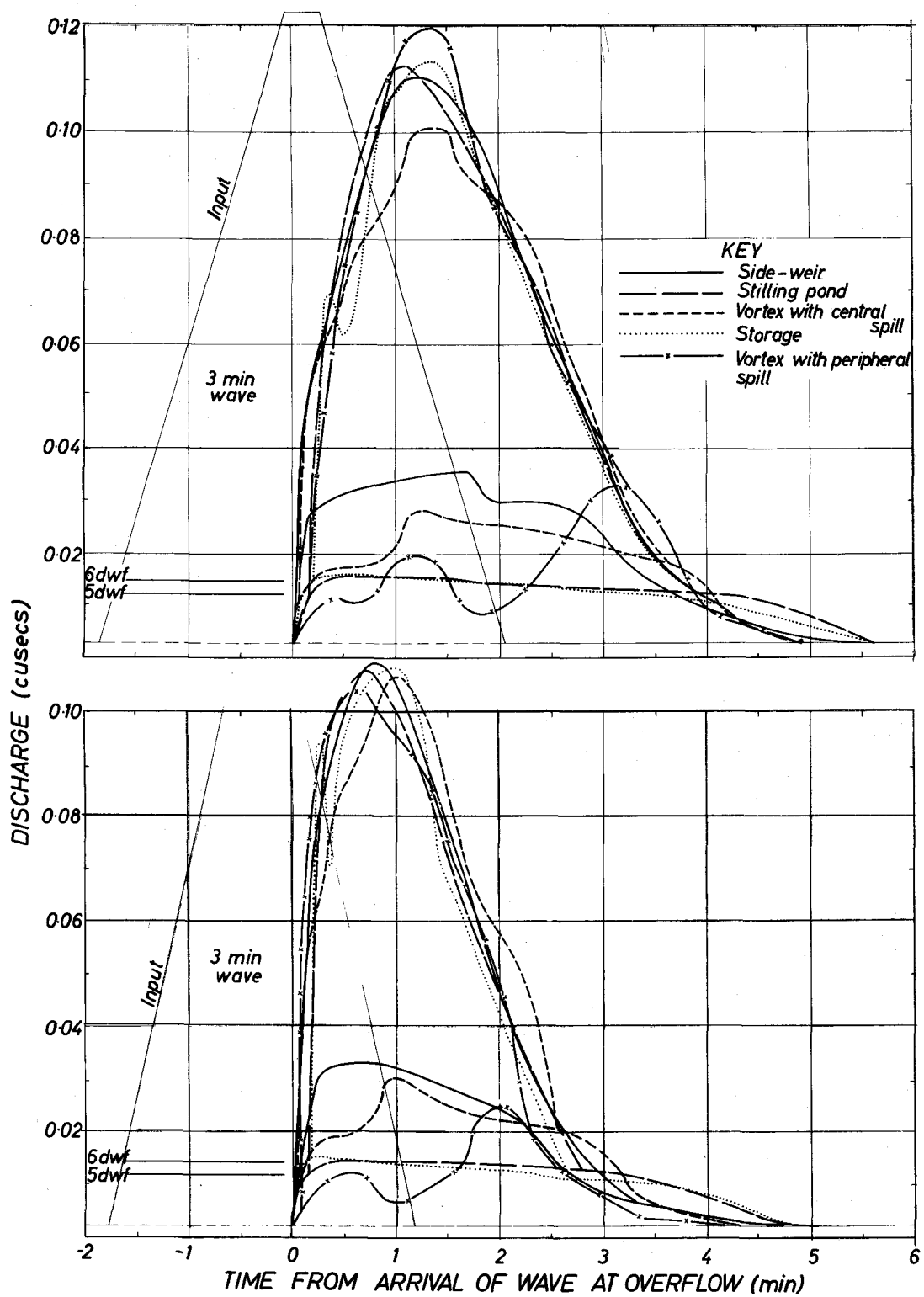
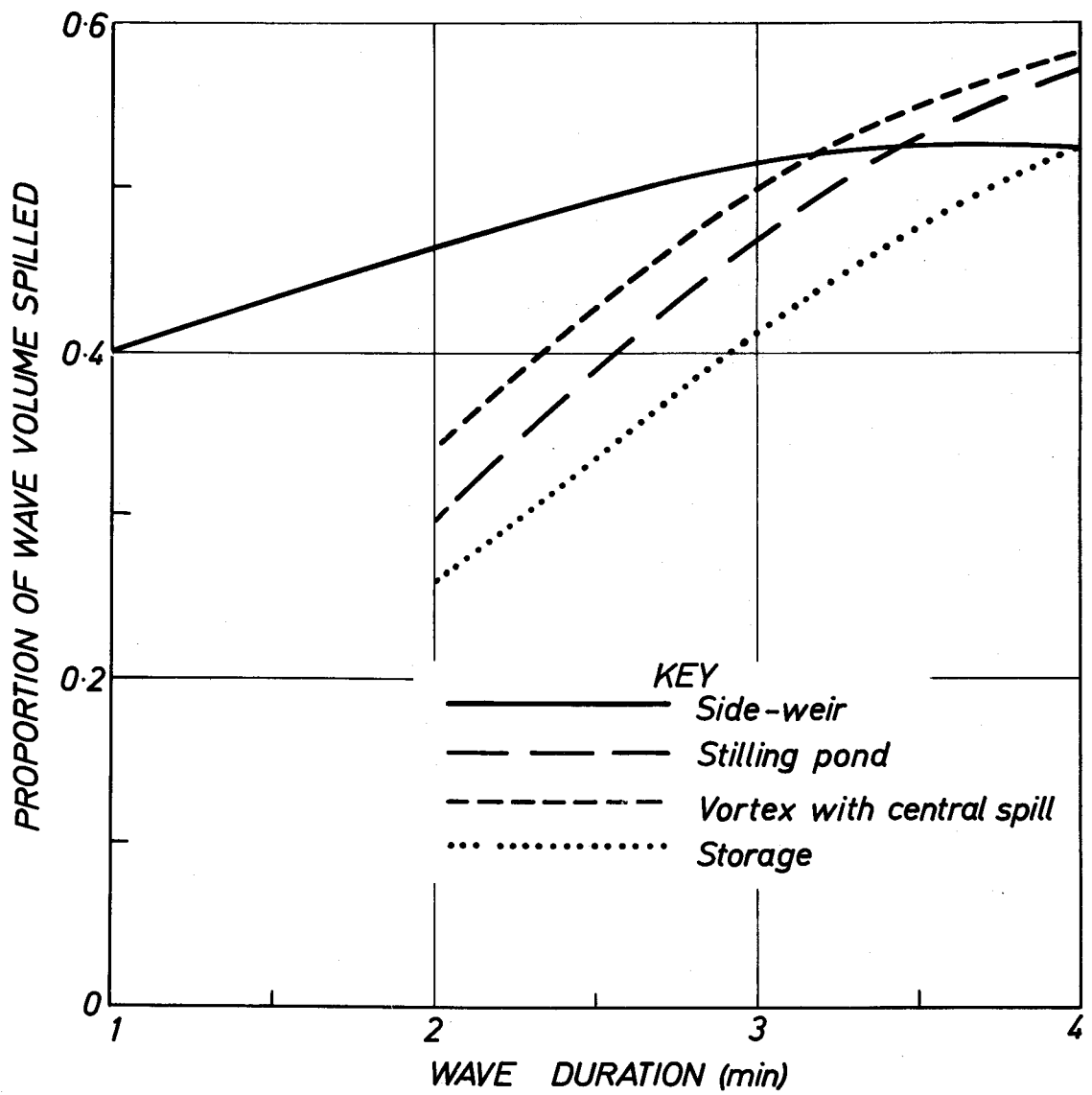


FIG 8a

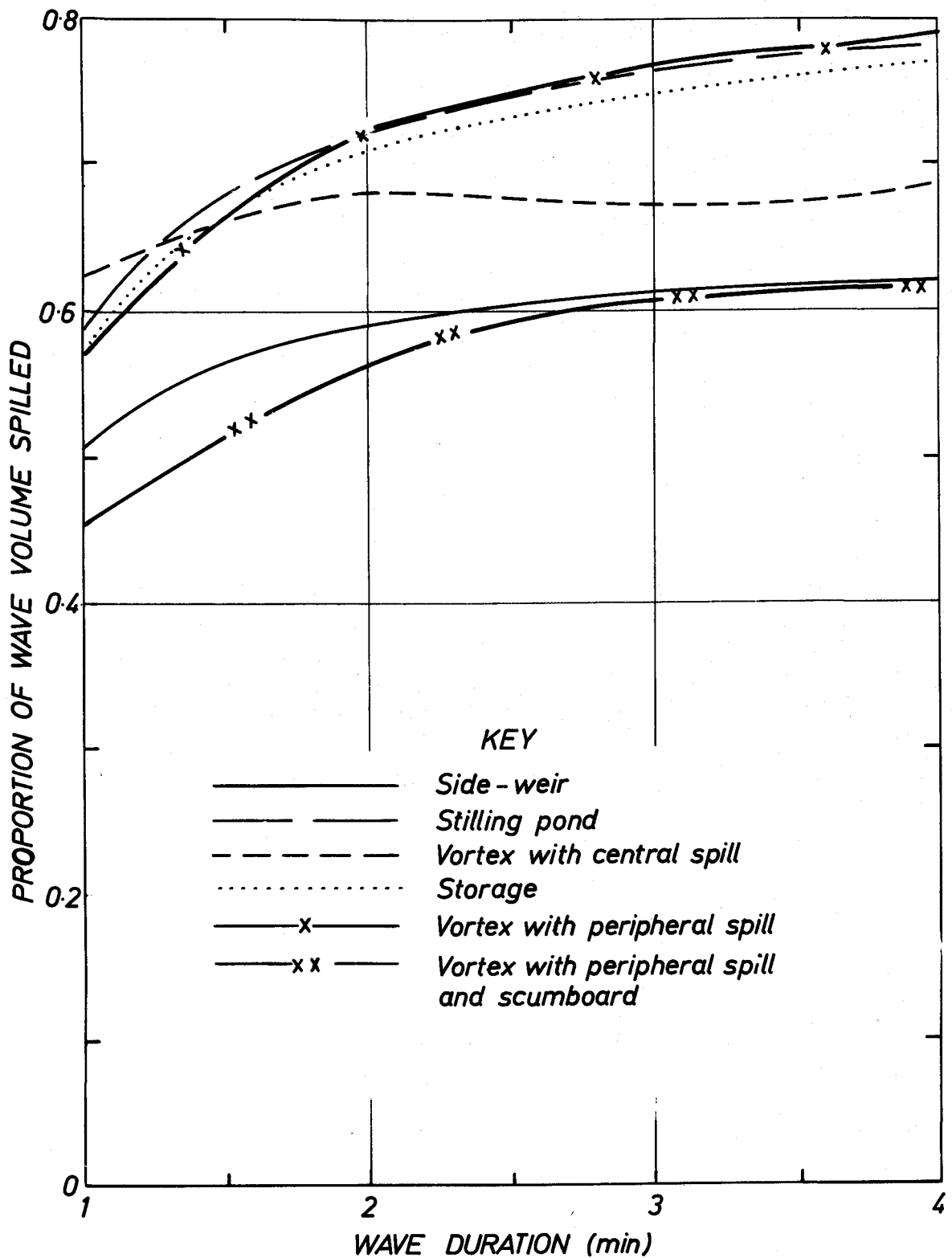


DISCHARGE - TIME CURVES
(PIPE SLOPE 1:100)



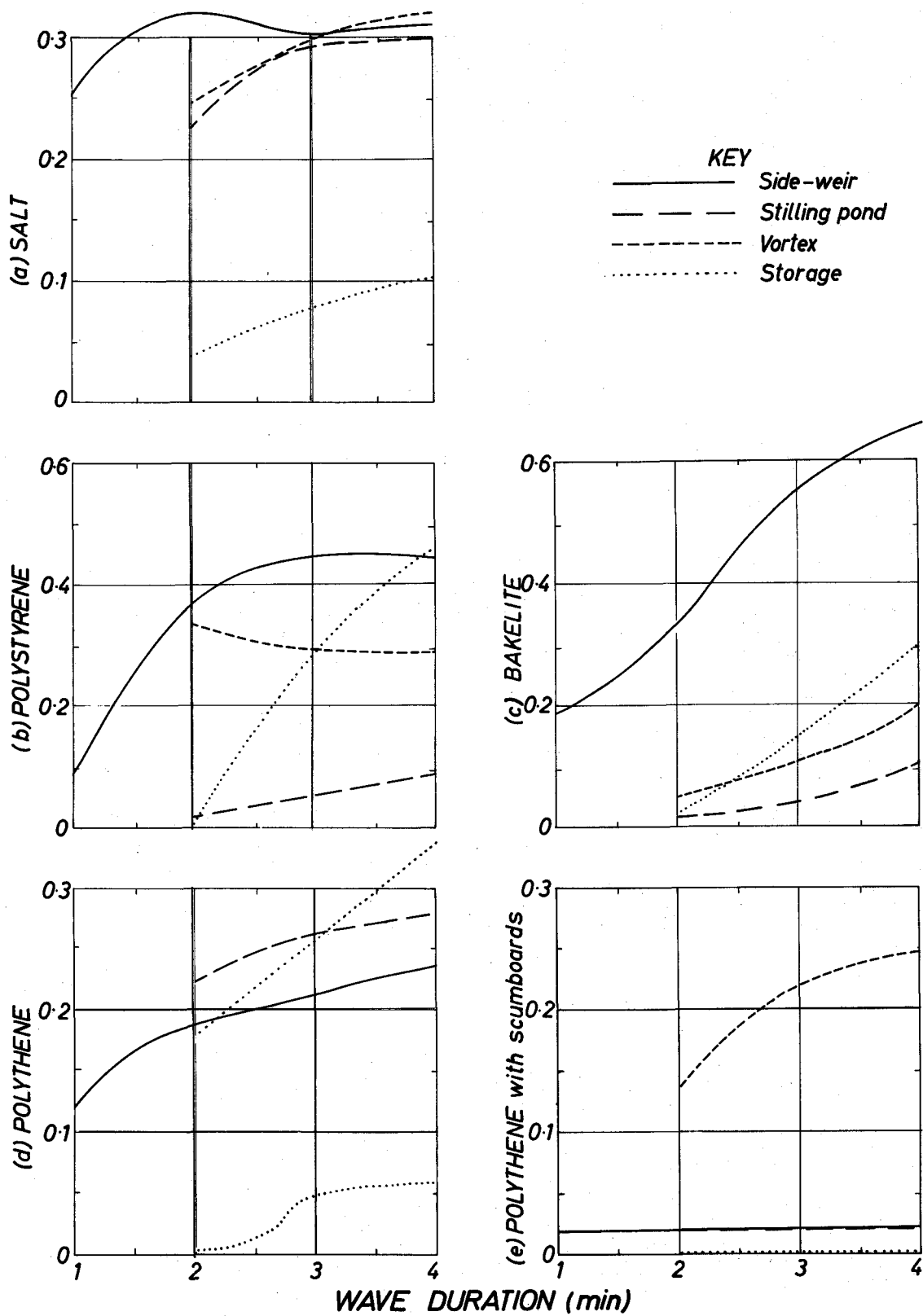
PROPORTION OF WAVE VOLUME SPILLED
(PIPE SLOPE 1:500)

FIG 9

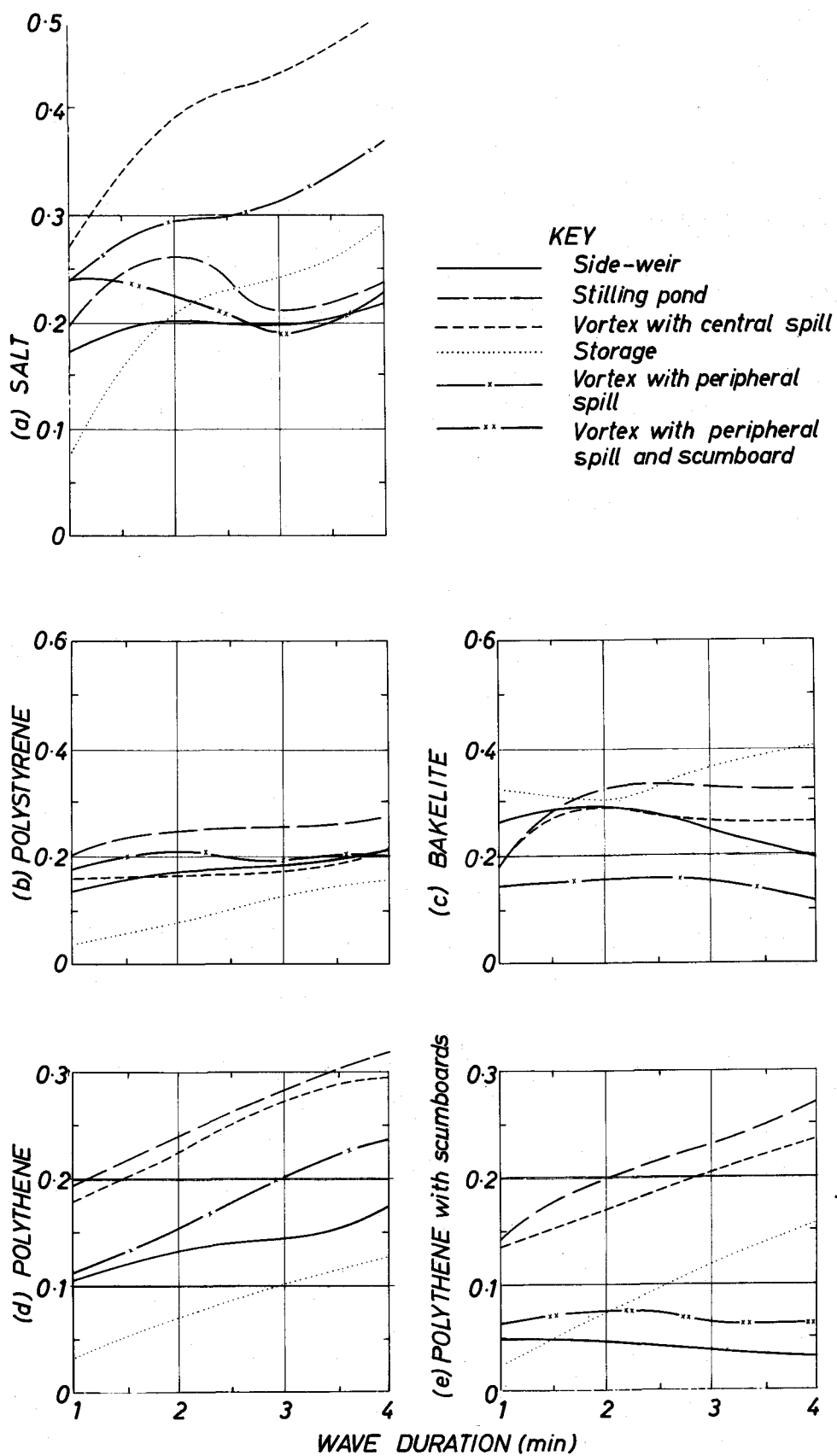


PROPORTION OF WAVE VOLUME SPILLED
(PIPE SLOPE 1:100)

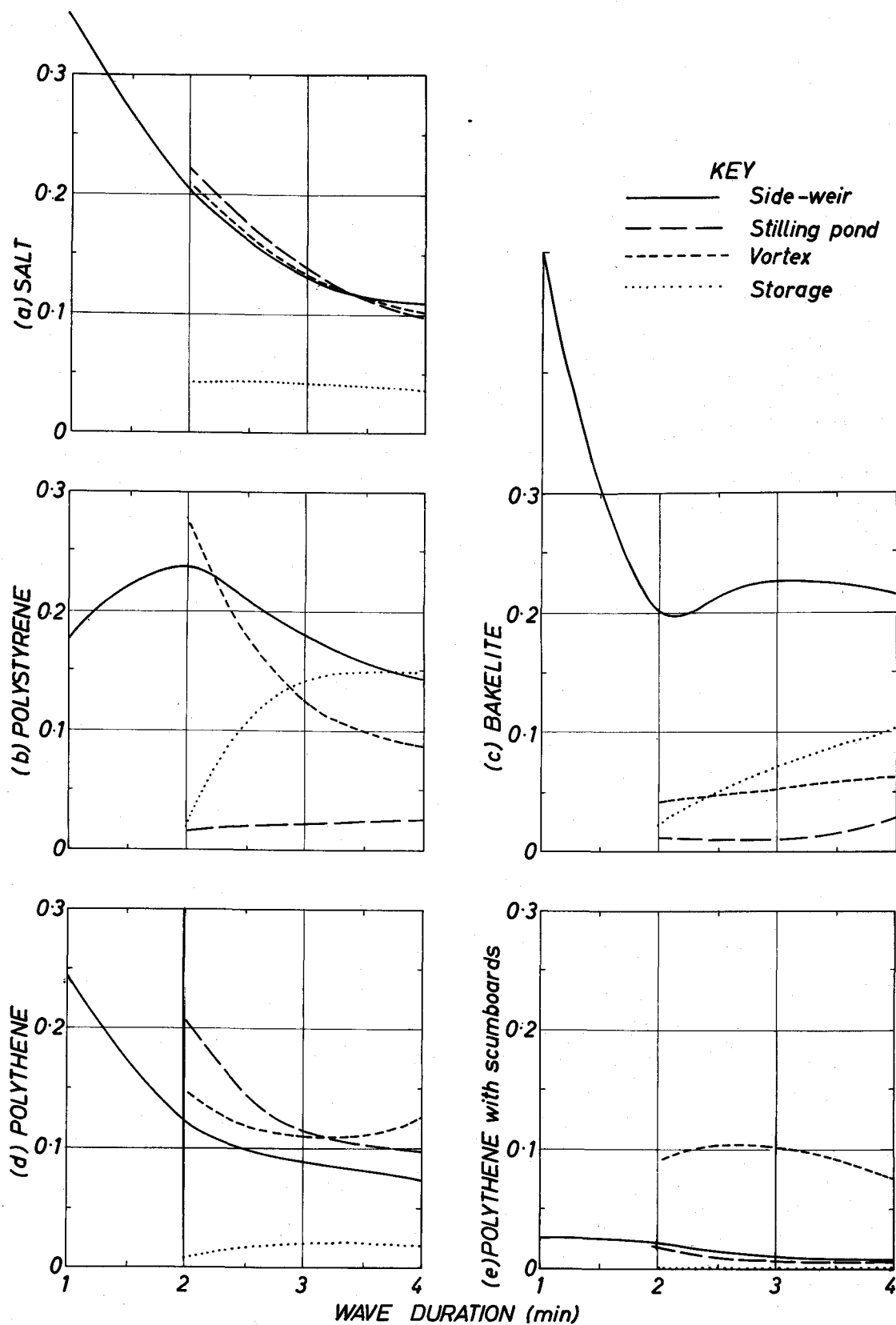
FIG 10



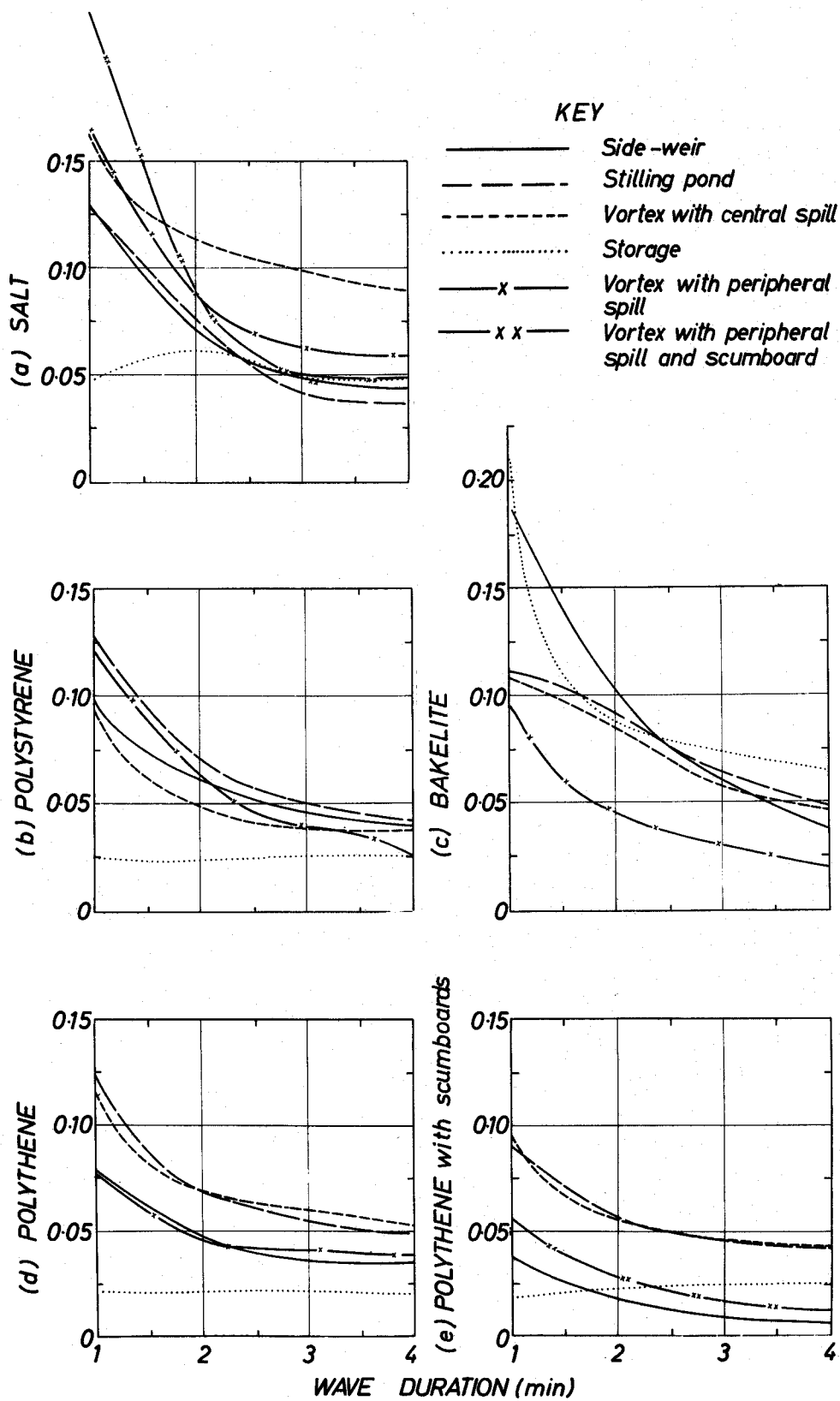
**PROPORTIONS OF POLLUTANTS SPILLED
PIPE SLOPE (1:500)**



PROPORTIONS OF POLLUTANTS SPILLED
(PIPE SLOPE 1:100)



AVERAGE CONCENTRATIONS OF POLLUTANTS IN SPILL AS
PROPORTIONS OF BASE-FLOW CONCENTRATION
(PIPE SLOPE 1:500)



**AVERAGE CONCENTRATIONS OF POLLUTANTS IN SPILL AS PROPORTIONS OF BASE FLOW CONCENTRATION
(PIPE SLOPE 1:100)**

PLATES

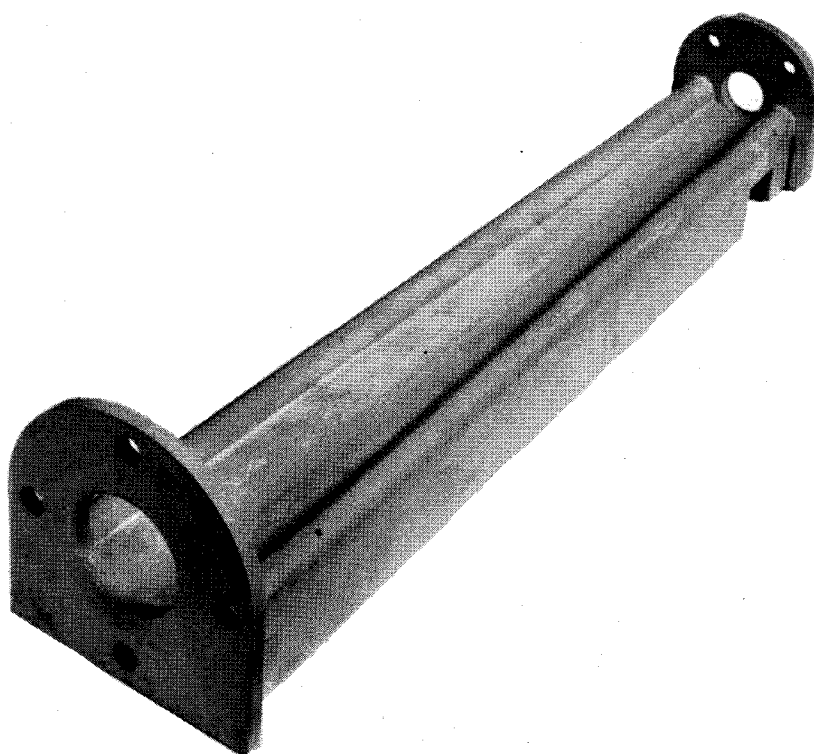


PLATE 1(a) Stormwater overflows for sewers
a low double side-weir

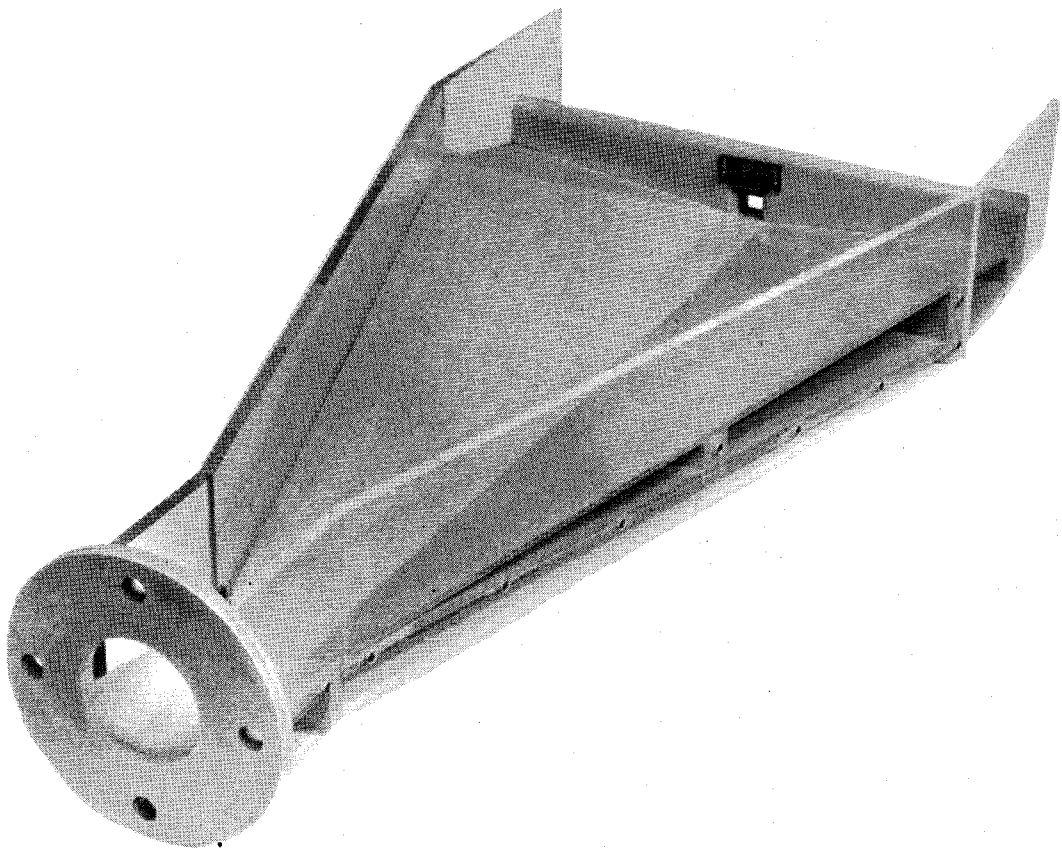


PLATE 1(b) Stormwater overflows for sewers
a stilling pond

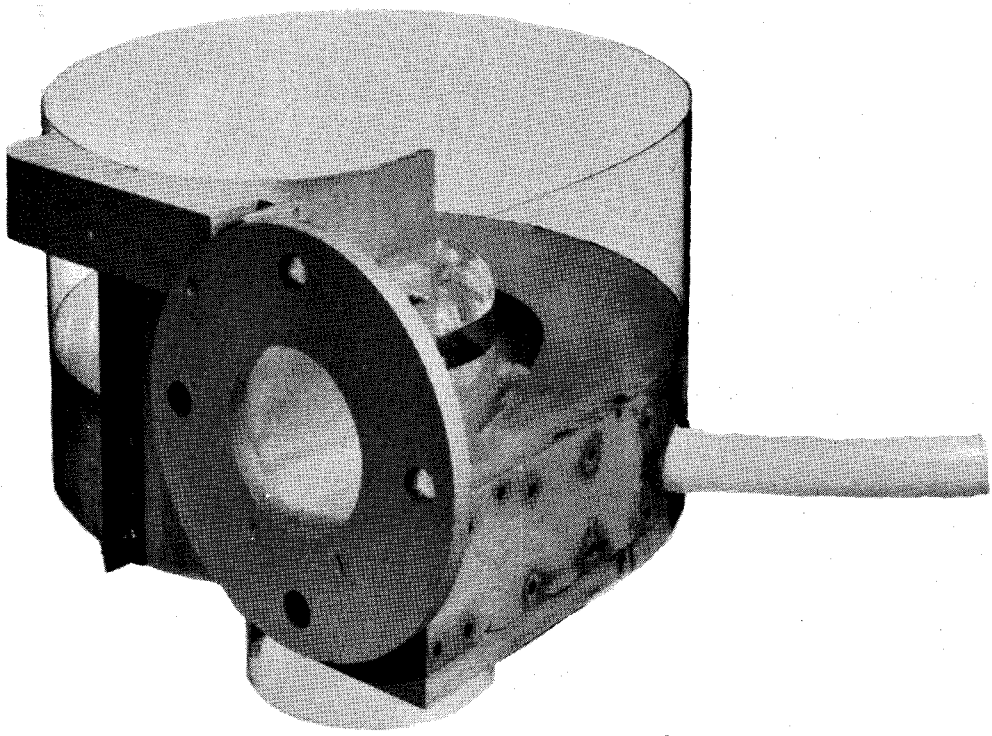


PLATE 1(c) Stormwater overflows for sewers
a vortex

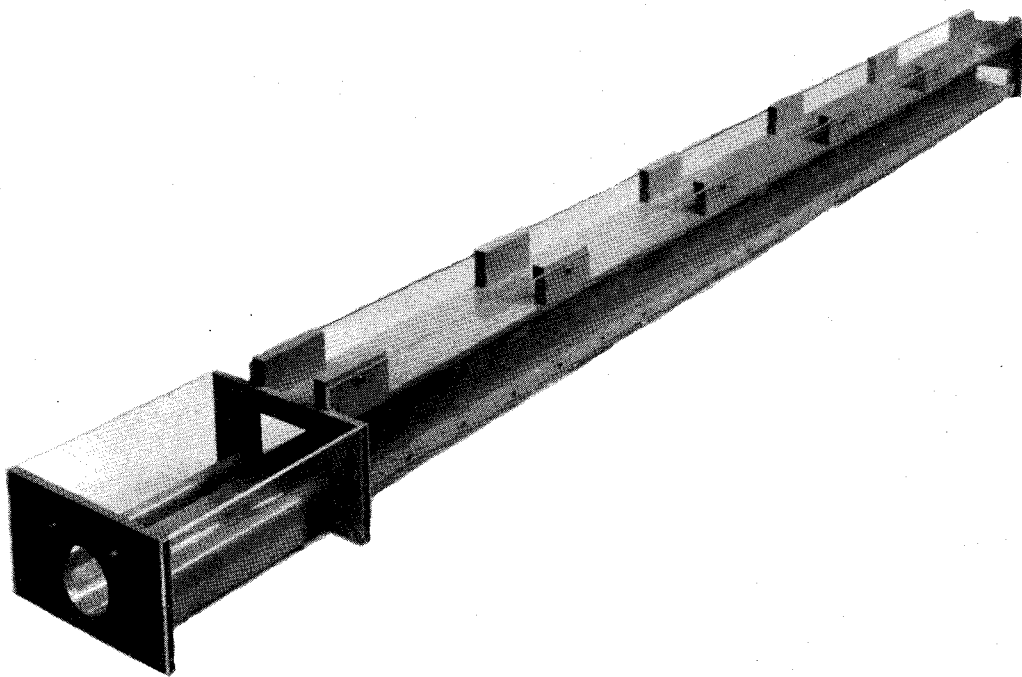


PLATE 1(d) Stormwater overflows for sewers
a storage overflow

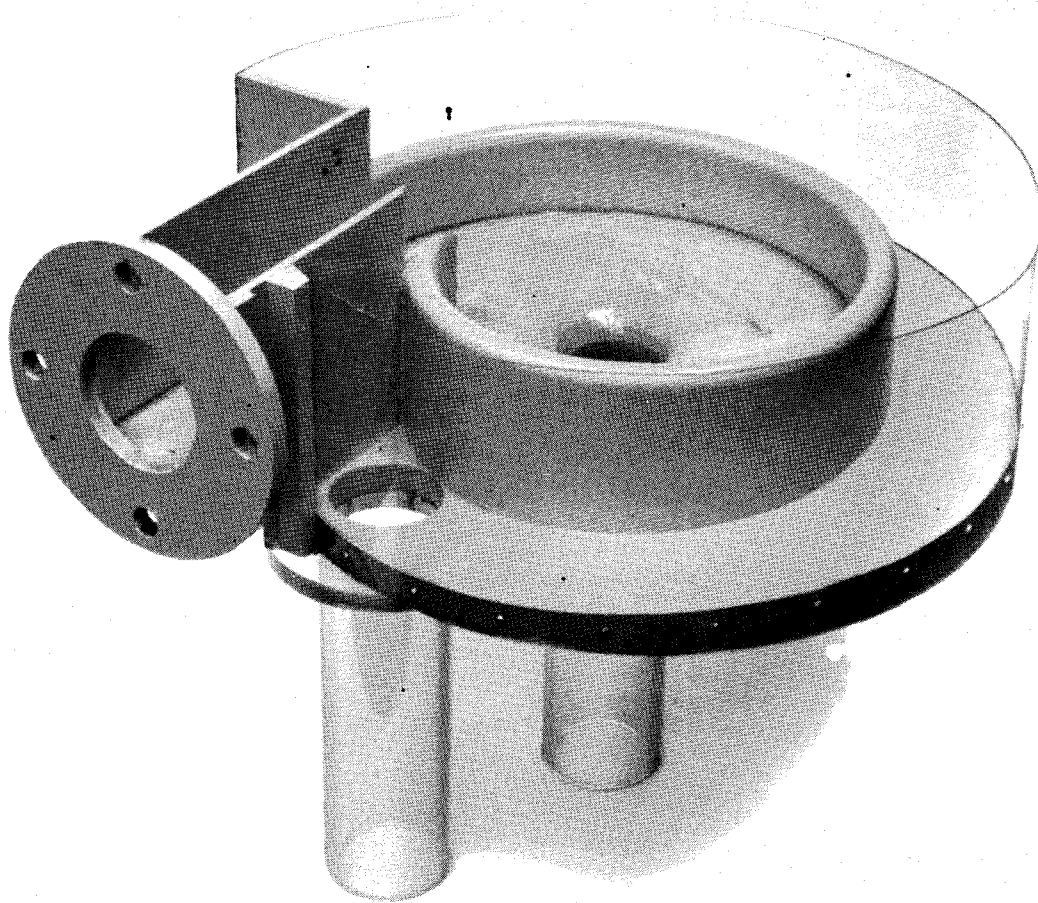


PLATE 2 Vortex overflow with peripheral spill