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UNPUBLISHED REPORT
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STUDY OF BRIDGE DECK DRAINAGE

by

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April 1981

PREFACE

This research was undertaken under an agreement with the Province of Ontario to investigate the design of bridge deck drainage on behalf of the Ministry of Transportation and Communications.

Messrs. J. E. Gruspier, D. G. Guibord, and J. D. Harris of the Ministry of Transportation and Communications were the technical liaison officers for this project. The author is indebted to them for all the assistance received during the project.

Finally, the author would like to acknowledge the invaluable help of Mr. D. Doede, Mrs. R. Purves, and Mrs. B. Jones of the Hydraulics Division, the National Water Research Institute. Mr. D. Doede and Mrs. R. Purves conducted laboratory experiments, Mrs. B. Jones prepared the text.

ABSTRACT

Hydraulic capacities of commonly used bridge deck drains are not well known and this seriously impedes attempts to rationalize the design of bridge deck drainage. In the report that follows, observed capacities and susceptibility to blockage by debris are given for seven drains.

Experimental drain capacities were used in the development of a new design procedure for bridge deck drainage. This procedure yields the drain spacings and can be applied in an expedient computerized version.

RÉSUMÉ

Il est difficile de rationaliser le drainage du tablier des ponts car on ne connaît pas la capacité hydraulique des installations habituellement employées pour cet usage. On donne dans le présent rapport la capacité hydraulique et la tendance au blocage de 7 drains.

En utilisant des capacités hydrauliques expérimentales, on a mis au point une nouvelle méthode de conception des installations de drainage. Celle-ci permet de connaître la dispostion des drains et peut être transformée en un efficace programme informatisé.

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1.0 INTRODUCTION

Bridge deck drainage has received little attention in the past, because it rarely leads to a structural failure. Consequently, many regard drainage as an inescapable nuisance, rather than a problem. It should be recognized, however, that poor drainage leads to a number of costly problems which could be avoided by a proper design.

Among the problems caused by poor drainage are ponding, uncontrolled drainage discharges, and cold weather problems. Ponding has a variety of causes and adversely affects traffic as well as the bridge structure. Uncontrolled drainage discharges may cause erosion, settlement of pavement slabs, and even structural failure. The runoff water falling on the bridge structure may cause stains and discoloration of exposed faces if it is not collected and disposed of properly. Runoff may also wash off corrosive contaminants and, in contact with structural members, cause deterioration of the bridge structure. Other drainage problems are brought about by cold weather. The freezing of infiltrated water can cause considerable damage.

It is obvious from the above examples that the importance of getting all the liquids off the bridge, in a controlled manner and as soon as possible, can not be overemphasized. From the drainage point of view, there probably cannot be too many drains. Increasing the number of drains reduces the length of travel of water on the bridge and the possibility of drain clogging. Also, should one drain become clogged, other drains can carry the load. When deciding on the spacing of drains, the designer has to consider the cost of drain installations and find a compromise between the cost of drainage and the nuisance or damages arising from underdesigned drainage facilities. It should be borne in mind that the cost of bridge deck drains varies substantially. In open country, a drain can be just an inexpensive piece of pipe installed in the deck. On the other hand, in urban conditions, typical drains are more elaborate structures accompanied by piping.

Recognizing the importance of bridge deck drainage and the lack of rational design procedures, the Ministry of Transportation and Communications commissioned the Hydraulics Division of the National Water Research Institute to conduct a study of bridge deck drainage. The terms of reference of this study (1) may be summarized as follows:

(1) Undertake a laboratory study of hydraulic capacities of commonly used bridge deck drains by means of prototype testing.

(2) Compare hydraulic efficiencies of the tested drains and, if feasible, recommend improved, bicycle safe designs (width up to 300 mm).

(3) Compare the susceptibility to blockage of the tested drains and, if feasible, recommend improvements in grate designs.

(4) Develop a design procedure for spacing of drains using the approach previously recommended for sewer inlets.

2.0 OVERVIEW OF BRIDGE DECK DRAINAGE

U.S. Transportation Research Board recently completed a report on Bridge Drainage Systems, Synthesis of Highway Practice 67 (4). Some of the reported findings were used in the study described here and, consequently, a summary of the U.S. report is presented below.

> Bridges should have adequate cross-slope and grade to allow the water to run quickly to the drains. Where grades permit drains are not used on short bridges but all the water is carried to catch basins at the ends. Bridge drains are sometimes open holes through the deck that can have short pipes to carry the water clear of the beams. More often, however, an inlet box is used to collect the runoff. The spacing and location of drains depend on the amount of rainfall expected, the design of the bridge, the grades, and what is beneath the bridge.
>
> Debris can be controlled by keeping it out of the inlet boxes, accepting and storing it so it cannot go through the system, or transporting it through. Because all debris cannot be kept out, many drainage systems are designed to trap larger debris and let the smaller debris pass. The grates used to screen out larger debris should be hydraulically efficient, strong enough to support traffic, and bicycle safe. If pipes have adequate size, slope, and curvature for debris control, hydraulic considerations seldom limit the flow.
>
> Maintenance at regular intervals is the key to susccess of a drainage system. Because this periodic attention is necessary, the design should make it as easy as possible. Numerous cleanouts should be provided in locations where they are easily and safely accessible. Cleaning equipment ranges from shovels to high-pressure water. Recent innovations include a system for backflushing with high-pressure air and a truck-mounted high-pressure water system.
>
> Disposal of runoff water can be a simple straight drop onto the land or water beneath the bridge or a pipe system to carry the water to a local sewer system. Provisions for controlling or containing spills of hazardous materials are costly and warranted only where the risks are high.
>
> The concensus of current practices indicates that deck cross-slope and grade should be no less than 2 percent and 0.5 percent, respectively; that bridge drains may be holes through the deck, fabricated inlet boxes, or catch basins at the ends of the bridge; that inlet areas should be as large as possible; that pipes should have a minimum diameter of 150 mm, a minimum radius of 450 mm, and a minimum slope of 2 percent (preferably 8 percent); that cleanout plugs and elbows should be easily accessible; that there should be improved communication between designers and maintenance personnel; and, most importantly, that bridge drainage systems should be regularly and carefully inspected and serviced.

3.0 EXPERIMENTAL STUDY OF BRIDGE DECK DRAINS

Experimental investigations of drains dealt with two aspects of drain operation - the hydraulic capacity and the susceptibility to blockage. The results are presented in the same order.

3.1 Hydraulic Capacities of Drains

Bridge deck drains function in the same manner as sewer inlets studied in earlier phases of this project (2, 3). The hydraulic capacity of a drain depends on its geometry, installation, and the characteristics of the road surface flow. On flat grades, the drain capacity may be controlled more by the road discharge (i.e. the supply) than by the drain geometry. Drains are typically installed in a small depression to increase their capacities.

Drain hydraulic capacities can be determined by various methods. Among the approaches used in the past, one could name analytical calculations and scale model studies. Analytical calculations require numerous approximations which may lead to large uncertainties in the calculated results. A typical calculation of the drain capacity is based on an assumption that, for drains of a sufficient length, all the water passing over the width of the drain is intercepted. Consequently, the designer determines the road discharge over the drain and then calculates the drain length as a function of the road flow velocity (4). Such a procedure has two inherent shortcomings - the drain length will vary from case to case and the assumption of a 100 percent flow interception by drain is violated by all the drains tested.

In scale model studies, it is practically impossible to achieve hydraulic similitude because of small flow depths and spacings between the grate bars. It is recognized therefore that actual phototype tests of drains provide the best evaluation of the drain capacity (4).

3.1.1 Experimental facility and test procedures

The road drainage test facility built in the Hydraulics Division Laboratory was used for full-scale testing of bridge deck drains. The details of this facility were given elsewhere (2), and a brief description follows.

The road drainage test facility comprises a water supply tank, a roadway section with a drain, and a water return structure. The water supply system delivers up to 0.250 m^3/s to the roadway. The roadway is about 12 m long and 3.5 m wide. Its grade and crossfall can be varied from 0 to 0.07 and from 0 to 0.06, respectively. About 8 m downstream from the upper end, the tested drains were installed according to the MTC specifications. The flow intercepted by drains was measured by a V-notch weir.

For individual tests, the roadway grade and crossfall were set. The discharge through the facility was increased in a number of steps until either the pavement spread of 3 m was reached, or the drainage channel was filled to the top of the curb. For each test, the road flow rate, depth and spread (upstream of the drain), and the drain discharge were measured.

3.1.2 Experimental programme

The scope of the experimental programme has been established by MTC in consultations with the Hydraulics Division. The total number of experiments was reduced by interpolations and extrapolations of observed data. In all such procedures, the accuracy of ± 10 percent had to be maintained.

Two basic types of drains were tested - the drains adjacent to the barrier (in this case a curb) and the median drains. The experimental programme comprised of close to 100 runs. A summary of the programme is given in Table 1.

3.1.3 Results and discussion of results

A preliminary analysis of experimental results indicated that some drains produced similar discharges. Consequently, average capacities of all the drains were compared to find out if capacities of two or more drains could be described by a common expression. The results of this analysis are listed in Table 2.

It is apparent from the data in Table 2 that drains SS 9-1A and SS 9-4A, and drains SS 9-2A and SS9-2B behaved quite similarly and the corresponding experimental results could be grouped together.

Drains SS 9-1A and SS 9-4A

The dimensions of both drains are shown in Figure 1.

Drain SS 9-1A is formed by a 219 mm OD pipe whose opening is slightly depressed below the projected road surface. The pipe opening is protected by two square bars (20x20 mm) installed parallel to the flow direction. The drain is a bicycle safe design.

TABLE 1 BRIDGE DECK DRAINS

EXPERIMENTAL PROGRAMME

Drain	Curb and Gutter	Grade	Crossfall	Spread*
SS 9 - 1A	E			
SS 9 - 2A	(Installation			
SS 9 - 2B	adjacent to	0.005, 0.001, 0.02	0.02, 0.04, 0.06	0.5 - 3.0 with-
SS 9 - 4A	the curb)	0.04, 0.06, 0.07		out overtopping
SS 9 - 6A				curb
SS 9 - 8A	Median	0.005, 0.01, 0.02		
SS 9 - 8B	Installation	0.04, 0.06	0.02	0.5 - 3.0

* For curb-side installation, the spread was measured from the curb. For median installation, the spread was measured from the waterline to waterline.

TABLE 2 RELATIVE DISCHARGES OF TESTED DRAINS

	DRAIN				
	SS 9-1A	SS 9-2A	SS 9-2B	SS 9-4A	SS 9-6A
Discharge in multiples of the mean discharge of the set	0.62	1.45	1.53	0.64	0.76

In an overall evaluation drain SS 9-1A is rather simple and inexpensive. It has a relatively small opening for intercepting the road flow (the free-opening area is 0.0252 m^2) and, consequently, it has a fairly low capacity. As follows from the comparison of the drains tested, drain SS 9-1A has the lowest capacity among all the drains, followed closely by drain SS 9-4A. In fact, the difference in mean capacities of these two drains was less than 4 percent.

Drain SS 9-4A (see Figure 1) is formed by a 219 mm OD pipe (the same as SS 9-1A) which is inclined at 45° and directs water away from the bridge structure. Such an arrangement appears to have two advantages -the opening area which is an ellipse obtained by cutting the 219 mm OD pipe at 45° is larger, and drainage water is likely to discharge further from the bridge structure.

The free-opening area of drain SS 9-4A is 0.0314 m^2. Although this area is significantly larger than that of drain SS 9-1A, the former drain is rather shallow on the side closer to the road centre and, consequently, does not intercept significantly higher flows than drain SS 9-1A. As stated before, drain SS 9-4A has a capacity of only 4 percent larger than that of drain SS 9-1A.

In an overall evaluation, drain SS 9-4A is relatively inexpensive and has a fairly low capacity. Compared to drain SS 9-1A, drain SS 9-4A is slightly more elaborate, but may direct better runoff from the bridge structure. Unless this feature is found particularly important by MTC, one could discontinue design SS 9-4A and replace it by a similar drain SS 9-1A.

Observed capacities of drains SS 9-1A and SS 9-4A were plotted together in Figure 2. It was noted that drain capacities varied insignificantly with the deck grade and crossfall. Detailed analyses of such variations yielded standard deviations of about ± 12 percent, which was quite close to the expected accuracy of the observed data. Under these circumstances, it was decided to simplify the data analysis by considering all the observed capacities, for both drains and various grades and crossfalls, as a single data set. This set was then approximated by a regression equation in the following form:

$$Q_d = 0.000\,13 + 0.099\,14\,d - 0.118\,49\,d^2 \qquad (1)$$

where Q_d is the drain capacity (discharge) in m^3/s, d is the depth of flow, upstream of the drain, in metres, and the numerical constants were obtained from the regression analysis.

It appears from Figure 2 that, for low flow depths, the observed drain discharges tend to fall below the regression curve. This is caused by the fact that, in the region of low depths and flows, the drain discharge is controlled to a large extent by the road flow which supplies water to the drain. On low grades ($S \leq 0.02$), for which most of the tests were done, the road flows are particularly small and this is then reflected in low observed drain discharges. It is possible to account for the control of the drain discharge by the road flow by simply imposing a constraint $Q_{drain} \leq Q_{road}$. Such a constraint was used in the design procedure described later.

The tendencies described here for drains SS 9-1A and SS 9-4A were also observed for other drains and taken into consideration the same way as described above.

The main advantage of having a single drain capacity curve, described by a regression equation, will become apparent in the section on the design of bridge deck drainage.

Drains SS 9-2A and SS 9-2B

Drains SS 9-2A and SS 9-2B are shown in Figure 3. Both drains have a rectangular opening 1000x230 mm connected to a drain pipe of 219 mm OD. The two drains differ only in the arrangement of the grate bars - the type 2A has two longitudinal bars and the type 2B has ten crossbars. Only the latter type appears to be bicycle safe.

Drains SS 9-2A and SS 9-2B have the highest capacities among all the drains tested. This follows from their large area of free openings - 0.2100 m^2 for the type 2A and 0.2024 m^2 for the type 2B. On the average, these two drains intercept more than twice the flow intercepted by other drains tested.

The capacities of both drains differed very little - by less than 5 percent. Surprisingly, the type 2B with crossbars yielded the higher capacity. An explanation for this deviation from the earlier results obtained for sewer inlets (2) was found in an inefficient operation of the drain with longitudinal bars. The flow enters this drain at a fairly high velocity and partly deflects upwards at the downstream end and leaves the box again. Note that at the downstream end, the drain box is less than 60 mm deep. In the case of the drain with crossbars, the flow velocity gradually decreases as the flow passes over the crossbars and, consequently, a larger part of the flow is intercepted.

Since the average capacities of both drains differed insignificantly, the observed data for both drains were grouped together and plotted in Figure 4. Detailed analyses of capacity variations for various grades and crossfalls indicated that these variations were fairly small, rarely exceeding ± 15 percent. To simplify the design procedure, all the data for various grades and crossfalls were grouped together and approximated by the following regression equation:

$$Q_d = 0.000\,39 + 0.268\,d - 0.440\,d^2 \tag{2}$$

where Q_d is the drain capacity (discharge) in m^3/s, d is the depth of flow, upstream of the drain, in metres, and the numerical constants provided the best fit. The selection of the above regression equation followed from a condition that the regression curve should pass through or close to the origin. As discussed earlier, the observed points fall below the curve in the region of low depth where the drain capacity is controlled by the road flow discharge. This can be accounted for by imposing the constraint $Q_d \leq Q_{road}$.

In an overall evaluation, drains SS 9-2A and SS 9-2B have the highest capacities among the tested curb drains, but will be more expensive to manufacture and install. When comparing types 2A and 2B, the latter type has a slight advantage, because it is bicycle safe and has a slightly higher capacity. Where large drain spacings are desirable, drains SS 9-2A and SS 9-2B are the clear choice among the drains tested.

Drain SS 9-6A

Drain SS 9-6A is a special design recommended for use with CSA G40.21-M-350A Steel Girders. The drain is formed by a 200 mm square tube fitted with two longitudinal bars in the opening (see Figure 5). It appears to be a relatively simple and inexpensive drain design with a limited capacity. The area of free openings is 0.0320 m^2, slightly larger than that mentioned earlier for circular drains. The drain appears to be bicycle safe.

Observed drain discharges were plotted in Figure 6. It appears that for practical purposes, all the observations for various grades and crossfalls could be approximated by a single regression equation.

$$Q_d = 0.000\,18 + 0.131\,d - 0.189\,d^2 \tag{3}$$

where Q_d is the drain capacity (discharge) in m^3/s, d is the depth of flow, upstream of the drain, in metres, and the numerical constants were obtained from the regression analysis.

In an overall evaluation, drain SS 9-6A which is a special design used with certain bridge structures is a simple, inexpensive design of limited capacity.

Median Drains SS 9-8A and SS 9-8B

Drains SS 9-8A and SS 9-8B are the only median drains studied. Basically, these two drains are identical to the earlier discussed types SS 9-2A and SS 9-2B, except for the installation (see Figure 7). The former drains are installed in the median, the latter adjacent to the curb. A median installation allows a much better inflow of runoff to the drain than the curb installation and this is fully reflected in the observed drain capacities.

The only difference between the type 8A and 8B is in the arrangement of the grate bars. The former type has longitudinal bars, the latter has crossbars.

Observed capacities of drains SS 9-8A and SS 9-8B were analysed in the same way as described for other drains - the data obtained for both types and various grades were grouped together and approximated by the following regression equation:

$$Q = 0.000\,9 + 0.0053\,d - 0.000\,147\,d^2 \qquad (4)$$

where d is the depth of flow upstream of the drain.

In an overall evaluation, the median drains SS 9-8A and SS 9-8B have the highest capacities among all the tested drains.

3.2 Susceptibility to Blockage

The hydraulic capacity of bridge deck drains can be substantially reduced if drain openings become partly or fully blocked with debris. A reduced drain capacity then may lead to ponding and the resulting adverse effects on traffic and increased maintenance costs. It is, therefore, of interest to evaluate the susceptibility to blockage of the drains tested and to look for possible improvements.

3.2.1 Experimental technique

In the comparative tests of drain blockage, two types of debris were used - straw and plastic sheets (about 0.2 x 0.2 m). The tests were done for two road flow rates - the low flow of about 0.005 m^3/s and the high flow of 0.025 m^3/s. The road grade was 0.02 and the crossfall was 0.02.

In the experiments with straw, a known quantity of dry straw was floated on the water surface well upstream of the drain. The straw trapped on the drain grate was collected, dried and weighted to determine the amount trapped.

In the tests with plastic sheets, five plastic sheets (0.2 x 0.2 m) were placed on the water surface upstream of the drain and the number of sheets trapped by the drain grate was recorded.

Both types of experiments were repeated twice to verify the repeatability of results. Average values of trapped quantities of the debris are shown in Figure 8.

3.2.2 Results and discussion of results

Several tendencies can be inferred from the data in Figure 8. It is quite apparent that, among the drains tested, drains SS 9-2A and SS 9-8A are the least susceptible to blockage. On the average, these drains trapped less than 16 percent of the incoming debris. The remaining debris either passed through the drain, for low flows, or bypassed the drain for high flows. It was also noted that drains SS 9-2A and SS 9-8A performed much better than similar designs with crossbars (SS 9-2B and SS 9-8B) which trapped about three times more debris.

With the exception of drains SS 9-2A and SS 9-8A, the susceptibility to blockage of the tested drains depended on the road flow rate. At low flows and spreads, a large fraction of the road flow was intercepted and this resulted in higher amounts of trapped debris. At high flows, most of the flow bypassed the drains and so did the debris.

Finally, the mean percentages of debris trapped, representing the susceptibility to blockage, are listed in Table 3.

3.3 Evaluation of Tested Drains

Basic characteristics, i.e. the hydraulic capacity and susceptibility to blockage, of the drains tested are given in Table 4. It follows from this table that, where high drain capacities are required in curb installations, drains SS 9-2A and SS 9-2B should be used. When comparing these two drains, only the type 2B is bicycle safe, but it is more susceptible to blockage by debris. The remaining three drains, used in curb installations, should be used in locations where closely spaced, inexpensive drains are acceptable.

Both median drains have fairly high capacities. The type with longitudinal bars, SS 9-8A, is less susceptible to blockage than the crossbar design (SS 9-8B).

TABLE 3 **DRAIN SUSCEPTIBILITY TO BLOCKAGE**

	DRAIN						
	SS 9-1A	SS 9-2A	SS 9-2B	SS 9-4A	SS 9-6A	SS 9-8A	SS 9-8B
Mean Percentage of Debris Trapped	50	16	47	35	69	17	49
Susceptibility to Blockage (Ascending Order)	6	1	4	3	7	2	5

TABLE 4 **CHARACTERISTICS OF BRIDGE DECK DRAINS**

	DRAIN						
	SS 9-1A	SS 9-2A	SS 9-2B	SS 9-4A	SS 9-6A	SS 9-8A	SS 9-8B
Relative Costs	Low	High	High	Low	Low	High	High
Relative Capacity	Low	Medium-High	Medium-High	Low	Low	High	High
Bicycle Safe	Yes	No	Yes	Yes	Yes	No	Yes
Susceptibility to Blockage	Medium	Low	Medium	Low-Medium	High	Low	Medium
Special Features	-	-	-	Directs discharge away from the structure	To be used with CSA steel girders	Median drain	Median drain

4.0 NEW DRAIN DESIGNS

One of the study objectives was to evaluate the presently used drain designs and, if feasible, to propose new designs of better characteristics.

Regarding drain capacities, the drains tested have relatively low capacities with the exception of drains SS 9-2A and SS 9-2B. Improvements in drain capacities would be desirable and could be achieved by increasing the drain width from the existing values 0.205 m-0.230 m to 0.300 m. It is understood that such a modification is acceptable to MTC. In the absence of experimental data for new designs, one can only estimate the capacities of newly proposed designs.

A tentative proposal for new drains derived from drains SS 9-1A, SS 9-4A and SS 9-6A is shown in Figure 9. The width of these designs is 300 mm. To make these designs bicycle safe, additional bars had to be added to the original grate. It is estimated that these new drains would have capacities about 50 percent higher than the old designs. Note that these increased capacities would still be below those of the existing drains SS 9-2A and SS 9-2B.

Drains SS 9-2A and SS 9-2B could be also modified by increasing their width from 230 mm to 300 mm and by increasing the depth of the drain box. A tentative proposal of the modified design is shown in Figure 10. It is expected that the modified designs would have capacities about 30 percent higher than the existing ones.

Regarding the susceptibility to blockage, there appears to be little room for further improvements of the existing drain grates. Comparative tests of drains SS 9-2A and SS 9-2B indicated that longitudinal bars were less susceptible to blockage than the crossbars. With the exception of drain SS 9-2B, all the drains have longitudinal bars, fairly widely spaced, and further improvements do not appear to be feasible.

Where bicycle safety is important, drains with crossbars (or diagonal bars) may have to be used, recognizing that this will lead to an increased probability of drain blockage.

5.0 DESIGN PROCEDURE FOR BRIDGE DECK DRAINAGE

The design of bridge deck drainage encompassses a large number of considerations including hydrological and hydraulic aspects, the type of the structure, location, operation and maintenance. The design procedure which follows concentrates on the hydrological and hydraulic aspects.

A survey of bridge deck drainage design practices indicated that these practices are largely based on experience and empiricism (4). Very few agencies have detailed procedures for determining the drain spacing. Two such procedures are briefly reviewed below.

5.1 Review of Selected Design Practices

5.1.1 Idaho Procedure (4)

Having selected a drain position, the runoff peak flow Q (cfs) at this point is calculated from the Rational Method as follows:

$$Q = 0.000\,023\, A\, i \qquad (5)$$

where A is the deck area in ft^2, i is the rainfall intensity in in/hr specified for six state districts. Using the calculated Q and a nomograph for flow in triangular channels (see the Appendix), the flow spread is calculated. For a standard drain, it is assumed that all the road flow in a strip 1.5 ft wide (1.75 ft wide for grades smaller than 0.02), adjacent to the curb, enters the drain. The remaining flow, as determined from the earlier mentioned nomograph, bypasses the drain. If this bypass is larger than 0.033 ft^3/s (0.001 m^3/s), another drain is located not less than 10 ft downstream from the first one. For most bridges, the maximum carryover flow from the last drain should not exceed 0.033 ft^3/s (about 0.001 m^3/s).

The following deck drain locations are suggested - over medians, water or slope paving, in low points, in flat areas, and in areas with minimum carryover.

5.1.2 California Procedure (4)

The runoff peak flow is computed from the Rational Method. Where precise values of rainfall intensity are not available, a five-minute rainfall intensity of 5 in/hr is assumed. The runoff coefficient is taken as C=1.0.

Drainage restrictions are such that no water is permitted to flow over joints, or the paving notch. The width of flow may vary, but should not encroach on the travelled way. Where the total flow interception is required, the drain width determines the maximum spread. The length of the drain grate varies depending on the flow velocity. Consequently, for a selected drain width, one determines the grate length from an empirical formula. The flow bypassing a given drain is added to the runoff quantity draining to the next following drain.

In summary, both procedures are fairly simplistic. The runoff peak flow is calculated from the Rational Method using typical values of the rainfall intensity. Actual drain capacities are not known, it is assumed that drains fully intercept road flows of a certain width. Typically, the drain positions are selected first and their inflow checked. If required, additional drains are placed shortly downstream.

5.2 Proposed Design Procedure

The newly proposed procedure has a number of distinct advantages over the procedures reviewed above. These advantages stem from the use of actual drain capacities, direct determination of drain spacings, and the computerization of the whole procedure. The computerized version simplifies the design and gives the designer a better appreciation of the drainage system behaviour. The general applicability of this procedure can be established only through extensive testing in actual design work.

A detailed description of the proposed design procedure follows.

5.2.1 General description

A notation sketch for the design procedure is given in Figure 11.

For design of bridge deck drainage, the following information is given:

		Dimension
Description of the bridge structure and location (important for runoff disposal)		
Limiting factors or criteria - e. g. such as no flow over the joints or the pavement notch.		
The roadway width	W_p	m
The shoulder width	W_s	m
The curb type		
The total drained width	$W = W_p + W_s$	m

The total drained length	L_t	m
The grade (longitudinal slope)	S	m/m
The crossfall	S_x	m/m
The type (or types) of drains to be used		-
The type of pavement and the corresponding Manning's roughness	n	-
The spread	T	m
The runoff coefficient	C	-
The concentration time	t_c	min
The return period of the design rainfall	R	years
The average design rainfall intensity for the concentration time t_c and the return period R (this value is read from a rainfall atlas).	i_{t_c}	mm/hr

From the above data, one can calculate:

The runoff peak flow (as a function of the length of the drained area)	Q_P	m^3/s
The maximum road flow capacity corresponding to spread T	Q_{gT}	m^3/s

Finally, the design is completed by calculating the following two parameters:

The spacing for the first drain	L_1	m
The spacing for the consecutive drains	L_c	m

5.2.2 Runoff peak calculation

Assuming the applicability of the Rational Method, the runoff peak may be expressed as

$$Q_p = 0.000\,000\,278 \text{ C i WL} \qquad (6)$$

where Q_p is the runoff peak in m^3/s, C is the dimensionless runoff coefficient (typically taken as C=0.95), i the average ten-year rainfall intensity of ten-or 15-minute duration (depending on the type of roadway) in mm/hr, and W and L are the width and length of a rectangular drainage area in metres, respectively. For other additional details, see an earlier progress report on this project (2).

5.2.3 Road flow calculation

The flow on the road surface with a crossfall may be described as an open-channel flow in a triangular channel. A modified Manning's equation for such a flow was presented in an earlier report (2) in the following form:

$$Q_r = \frac{0.375}{n\,S_x} S^{1/2} d^{8/3} \tag{7}$$

where Q_r is the road flow (in a triangular channel), n is the Manning's coefficient of roughness of the channel, S_x is the road (deck) crossfall, S is the grade, and d is the flow depth at the curb. Alternatively, one may use the nomograph given in the Appendix.

5.2.4 Calculation of drain spacing

The calculation of drain spacing is based on the condition that the road flow spread, immediately upstream of the drain, will reach the design value. Such an approach is identical to that proposed earlier for sewer inlets (2).

First drain spacing - To obtain the spacing for the first drain on a continuous grade, the drain is assumed to be located at a point where the runoff peak flow, from the drained area, just reached the road flow Q_{rT} corresponding to the design spread:

$$Q_p = Q_{rT} \tag{8}$$

After substituting from Equations 6 and 7 into Equation 8, one obtains the following expression for the first spacing:

$$L_1 = \frac{1.35 \times 10^6\, S^{1/2}\, d^{8/3}}{C\,i\,W\,n\,S_x} \tag{9}$$

Equation 9 can be further rearranged by substituting $d = TS_x$

$$L_1 = \frac{1.35 \times 10^6\, S^{1/2} T^{8/3} S_x^{\ 5/3}}{C\,i\,W\,n} \tag{10}$$

where d is the depth of flow at the curb, T is the spread, and S_x is the crossfall.

It is of interest to note that the first spacing does not depend on drain characteristics. It is controlled by the characteristics of the road flow.

At the first drain, some fraction of the road flow is intercepted by the drain and the remainder, referred to as a carryover flow Q_c, bypasses the drain. This condition can be expressed as

$$Q_{rT} = Q_{dT} + Q_c \tag{11}$$

where Q_{dT} is the drain discharge for the spread T.

Consecutive drain spacing - Past the first drain, the flow on the road starts to increase because of runoff contributions. The next drain is again installed at the point where the road flow reaches the design spread, i.e. $Q_r = Q_{rT}$. After expressing the road flow as $Q_r = Q_c + Q_p$ and substituting for Q_{rT} from Equation 11, one obtains

$$Q_p = Q_{dT} \tag{12}$$

By substituting for Q_p from Equation 6 and solving for L_c, the consecutive spacing, one obtains the following final expression:

$$L_c = \frac{3.6 \times 10^6\, Q_{dT}}{C\, i\, W} \tag{13}$$

The drain discharge Q_{dT}, which corresponds to the design spread T, can be determined from the earlier given regression equations (Equation 1-4). In fact, one could substitute those equations into Equation 13 with the following results:

$$\text{Drain } \begin{matrix}\text{SS 9-1A}\\ \text{SS 9-4A}\end{matrix} \quad L_c = \frac{3.6 \times 10^6 (0.000\,13 + 0.099\,1\, TS_x - 0.118\,5\, T^2 S_x^2)}{C\, i\, W} \tag{14a}$$

$$\begin{matrix}\text{SS 9-2A}\\ \text{SS 9-2B}\end{matrix} \quad L_c = \frac{3.6 \times 10^6 (0.000\,39 + 0.268\, TS_x - 0.440\, T^2 S_x^2)}{C\, i\, W} \tag{14b}$$

$$\text{SS 9-6A} \quad L_c = \frac{3.6 \times 10^6 (0.000\,18 + 0.131\, TS_x - 0.189\, T^2 S_x^2)}{C\, i\, W} \tag{14c}$$

SS 9-8A
SS 9-8B

$$L_c = \frac{3.6 \times 10^6 (0.0009 + 0.0053\, TS_x - 0.000147\, T^2 S_x^2)}{C\, i\, W} \quad (14d)$$

The total number of drains N can be determined from an expression

$$N = 1 + (L_T - L_1) / L_c \quad (15)$$

where L_T is the total length to be drained.

Typically, the calculated N will not be an integer. By rounding off N down, there will be a length of the deck, L_r, contributing runoff past the last drain. Thus the flow at the end of the drained area consists of the carryover past the last drain plus the runoff generated over the length L_r

$$Q_{outflow} = Q_c + Q_{pLr} \quad (16)$$

where Q_c can be calculated from Equation 11 and $Q_p L_r$ can be calculated from Equation 6 by substituting L_r. After making these substitutions, one obtains the following expression:

$$Q_{outflow} = \frac{0.375\, S^{1/2}\, T^{8/3}\, S_x^{5/3}}{n} + 0.000\,000\,278\, C\, i\, W\, L_r - Q_{dT} \quad (17)$$

The designer then decides on the disposal of the outflow. If a full interception is required, more than one drain may be needed to achieve it.

Finally, the designer checks the calculated locations of drains and makes adjustments required to dispose of the drained water in suitable places.

The design procedure described above can be easily computerized using a programmable calculator. An example of a design program written for the Hewlett-Packard Calculator Model 9825 is given in the Appendix.

An example of a drain spacing design using the proposed procedure follows.

5.3 <u>Design Example</u>

Given Data: $S=0.02$; $S_x=0.02$; $n=0.013$; $W=9.5$ m; $L_T=1{,}000$ m; $T=1.5$m; Drain SS 9-2A; $t_c=15$ min; $i_{15}=150$ mm/hr; $C=0.95$; the deck runoff should be fully intercepted.

Find: The total number of drains.

Step 1 - Determine the first drain spacing from Equation 10.

$$L_1 = \frac{1.35 \times 10^6 \times 0.02^{1/2} \times 1.5^{8/3} \times 0.02^{5/3}}{0.95 \times 150 \times 9.5 \times 0.013} = 47.1 \text{ m}$$

Step 2 - Determine the drain discharge corresponding to the spread of 1.5 m. The corresponding flow depth d can be calculated as

$$d = TS_x = 1.5 \times 0.02 = 0.03 \text{ m}$$

The drain discharge is then calcualted from Equation 2 as

$$Q_d = 0.000\,39 + 0.268 \times 0.03 - 0.44 \times 0.03^2 = 0.008\,0 \text{ m}^3/\text{s}$$

Check if the condition $Q_d \leq Q_{rt}$ is satisfied. For that purpose, calcualte Q_{rT} from equation 7 as

$$Q_{rT} = \frac{0.375}{0.013 \times 0.02} \; 0.02^{1/2} \; 0.03^{8/3} = 0.0177 \text{ m}^3/\text{s}$$

Thus

$$Q_d = 0.008 < 0.0177 = Q_{rT}.$$

Step 3 - Calculate the consecutive spacings from Equation 13

$$L_c = \frac{3.6 \times 10^6 \times 0.008}{0.95 \times 150 \times 9.5} = 21.4 \text{ m}$$

Alternatively, steps 2 and 3 could have been combined by using a more tedious equation (14b) with the same result

$$L_c = \frac{3.6 \times 10^6 \,(0.000\,39 + 0.268 \times 1.5 \times 0.02 - 0.440 \times 1.5^2 \times 0.02^2)}{0.95 \times 150 \times 9.5} = 21.4 \text{ m}$$

Step 4 - Calculate the number of drains needed from Equation 15

$$N = 1 + (L_T - L_1)/L_C = 1 + (1000 - 47.1)/21.4 = 45.5$$

Assume N=45 and calculate the drained length as

$$L_{Td} = L_1 + 44 \times L_C = 47.1 + 44 \times 21.4 = 988.7 \text{ m}$$

Step 5 Calculate the outflow at the end of the bridge.
Determine the length to be drained past the last drain.

$$L_r = L_T - L_{Td} = 1000 - 988.7 = 11.3 \text{ m}$$

Substitute into Equation 17

$$Q_{outflow} = \frac{0.375 \times 0.02^{1/2} \times 1.5^{8/3} \times 0.02^{5/3}}{0.013} + 0.000\,000\,278 \times 0.95 \times 150$$

$$\times\, 9.5 \times 11.3 - 0.08 = 0.0140 \text{ m}^3/\text{s}$$

Step 6 Determine the number of drains required to intercept the outflow.

Since Q_{dT}=0.0080 m^3/s (for T=1.5 m), two drains are required to intercept the outflow. Thus the total number of drains is 45+2=47.

Step 7 Finalize the design by checking the drain locations for suitable disposal of drained water.

5.4 Other Design Considerations

Apart from the calculation of drain spacings, there is a number of other design considerations. Drains should not be located over highway or railroad travelway because piping would be required. Locations over water, slope paving, or medians (with a suitable landing spot for water) are ideal. Higher denisty of drains than calculated may be required when placing drains in low points and flat areas, or when trying to avoid streams of water crossing the travelled roadway on bridges with superelevation and at the end of the centre divider curb, which is terminated short of the end of the bridge with superelevation.

The blockage of drains by debris may disrupt the operation of the bridge deck drainage. The problem of inlet blockage was discussed in an ealier report (2). The report suggested that inlet blockage by debris and the resulting reduced capacity should be considered only in those cases where it could lead to serious damages and disruption of traffic. A good example of such a situation is the inlet in a sag. Similar recommendations apply to the bridge deck drains. It is believed that the problem of drain blockage can be prevented through regular inspection and maintenance procedures.

6.0 FUTURE RESEARCH

During the course of the Bridge Deck Drainage study, two problems requiring further attention have been identified. Both problems could be addressed immediately using the existing experimental apparatus.

6.1 Capacity of New Large Drains

As suggested by MTC, the currently used bridge deck drains could be replaced by a new series of drains with widths up to 300 mm. Such a change would be desirable, because it would increase drain capacities and spacings, and thereby reduce the drainage costs. It is recommended, therefore, to manufacture wooden prototypes of the new drains and test them in the laboratory. In particular, the new drains derived from the existing designs SS 9-1A, SS 9–2A, and SS 9-6A deserve further attention.

6.2 Capacity of Drains in a Sag

The capacity of the new (or old) drains in a sag could be readily established using the existing apparatus. More detailed testing would be required for circular drains (e.g. SS 9-1A) whose capacities in a sag cannot be readily estimated from formulas derived earlier for rectangular inlets.

7.0 SUMMARY OF FINDINGS

Hydraulic capacities of seven bridge deck drains currently used by MTC were established by a full-scale testing in the hydraulics laboratory. Drain capacities depended on the drain geometry and on the road flow conditions.

Among the five drains adjacent to the curb, only the types SS 9-2A and SS 9-2B had satisfactory capacities typically ranging from 0.006 to 0.013 m^3/s for the depths of flow from 0.02 to 0.05 m. The corresponding capacities of the remaining three drains varied from 0.002 to 0.006 m^3/s.

The observed drain capacities varied primarily with the varying depth of flow, the effects of the road grade and crossfall were practically negligible.

Among the drains tested, the type SS 9-2A which has widely-spaced longitudinal bars was by far the least susceptible to blockage by debris. Unfortunately, this drain is not bicycle safe.

Two median drains, SS 9-8A and SS 9-8B, were also tested. These drains are quite similar to the types SS 9-2A and SS 9-2B discussed earlier and produced similar results.

A new design method for spacing of bridge deck drains has been developed. This method is based on the design spread and its main advantages follow from the use of observed drain capacities, direct calculation of drain spacings, and computerization of the design procedure.

8.0 REFERENCES

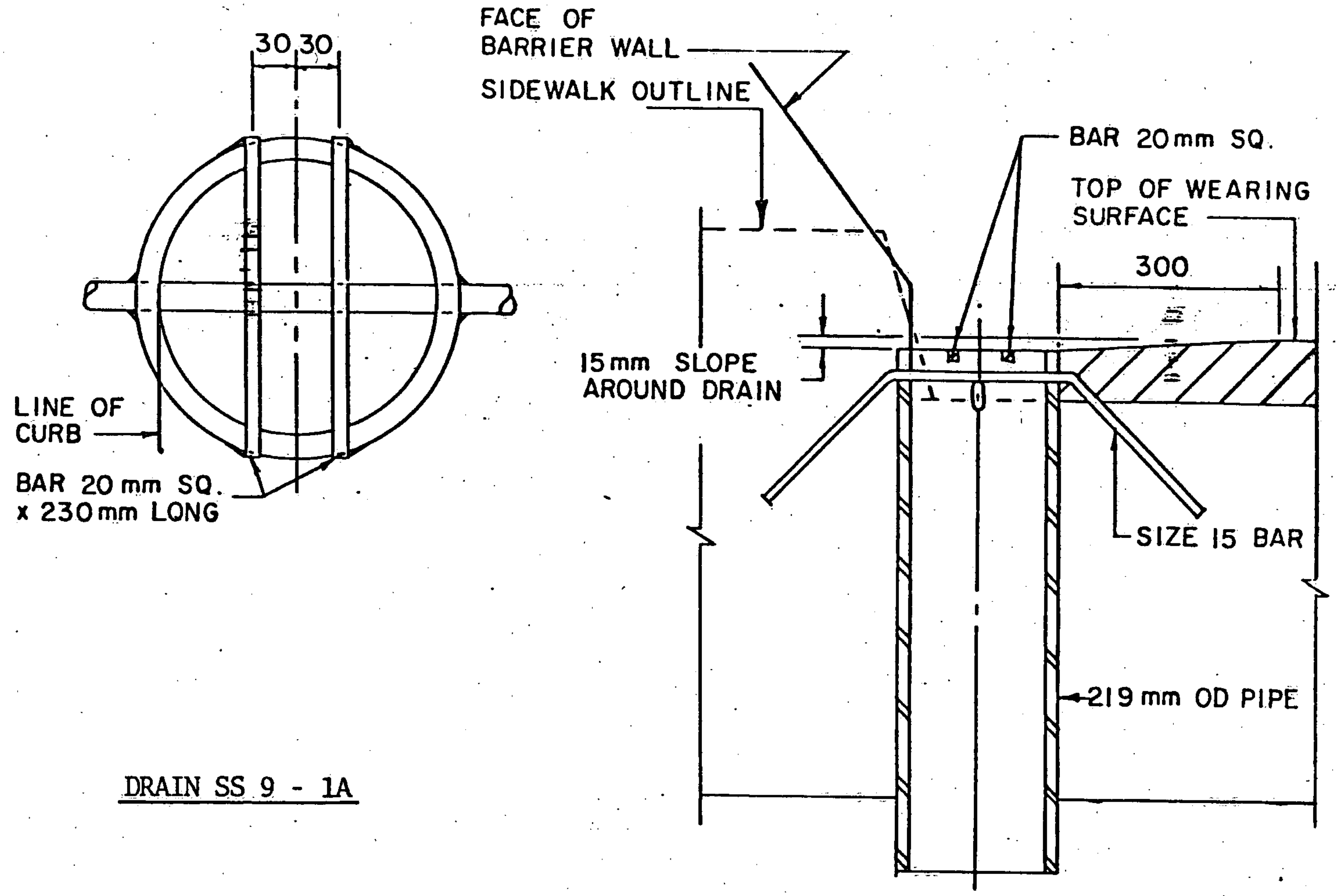
1. Gruspier, J. E., 1980. "Bridge Deck Drains: Research Needs Statement, Project 23211". Ministry of Transportation and Communications, Toronto.

2. Maršalek, J., 1979. "Sewer Inlets Study". A report to the Ministry of Transportation and Communications, July.

3. Marsalek, J., 1980. "Sewer Inlets Study: Laboratory Investigations of Selected Inlets". A report to the Ministry of Transportation and Communications, July.

4. Transportation Research Board, 1979. "Bridge Drainage Systems". National Cooperative Highway Research Program, Synthesis of Highway Practice 67, National Research Council, Washington, D. C., December.

APPENDIX



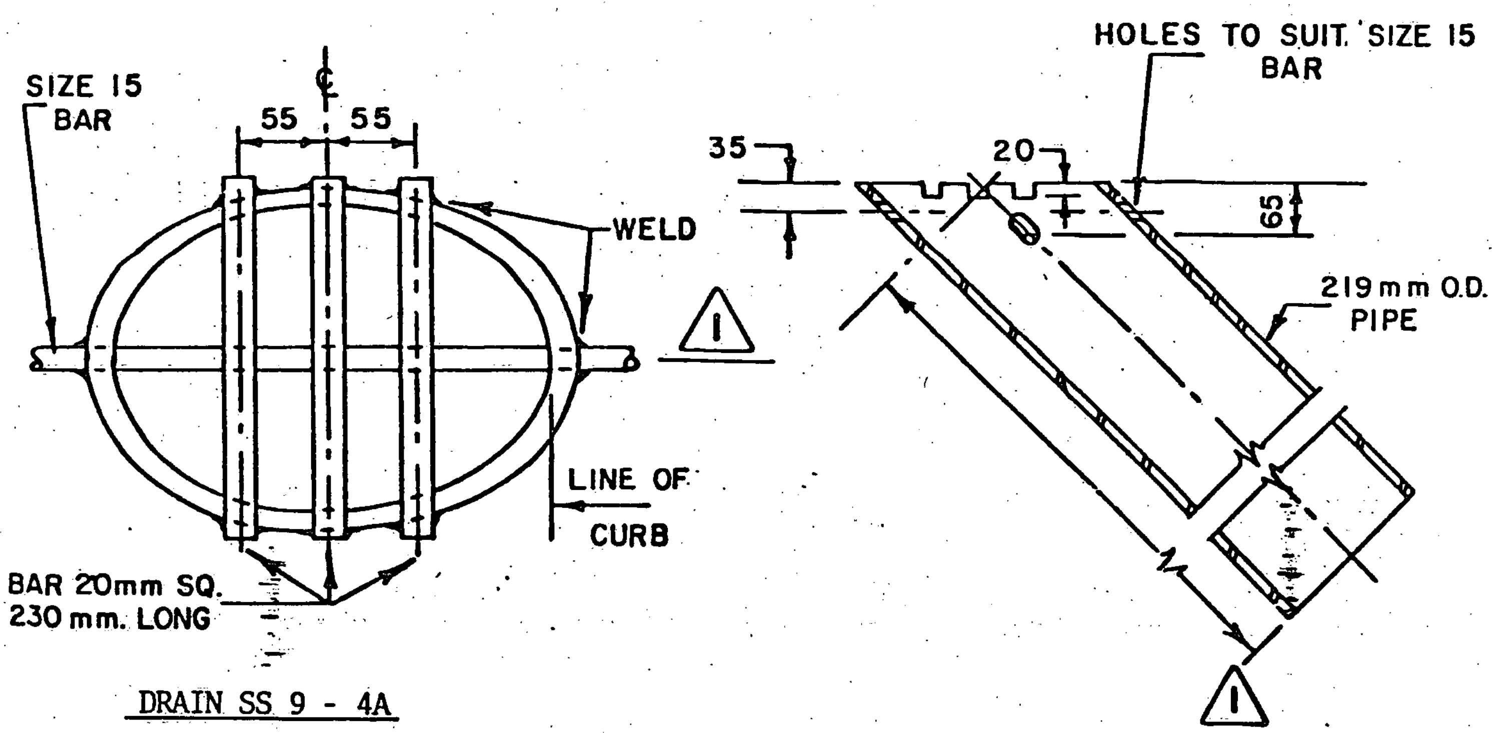


Fig.1. DRAINS SS 9 - 1A AND SS 9 - 4A

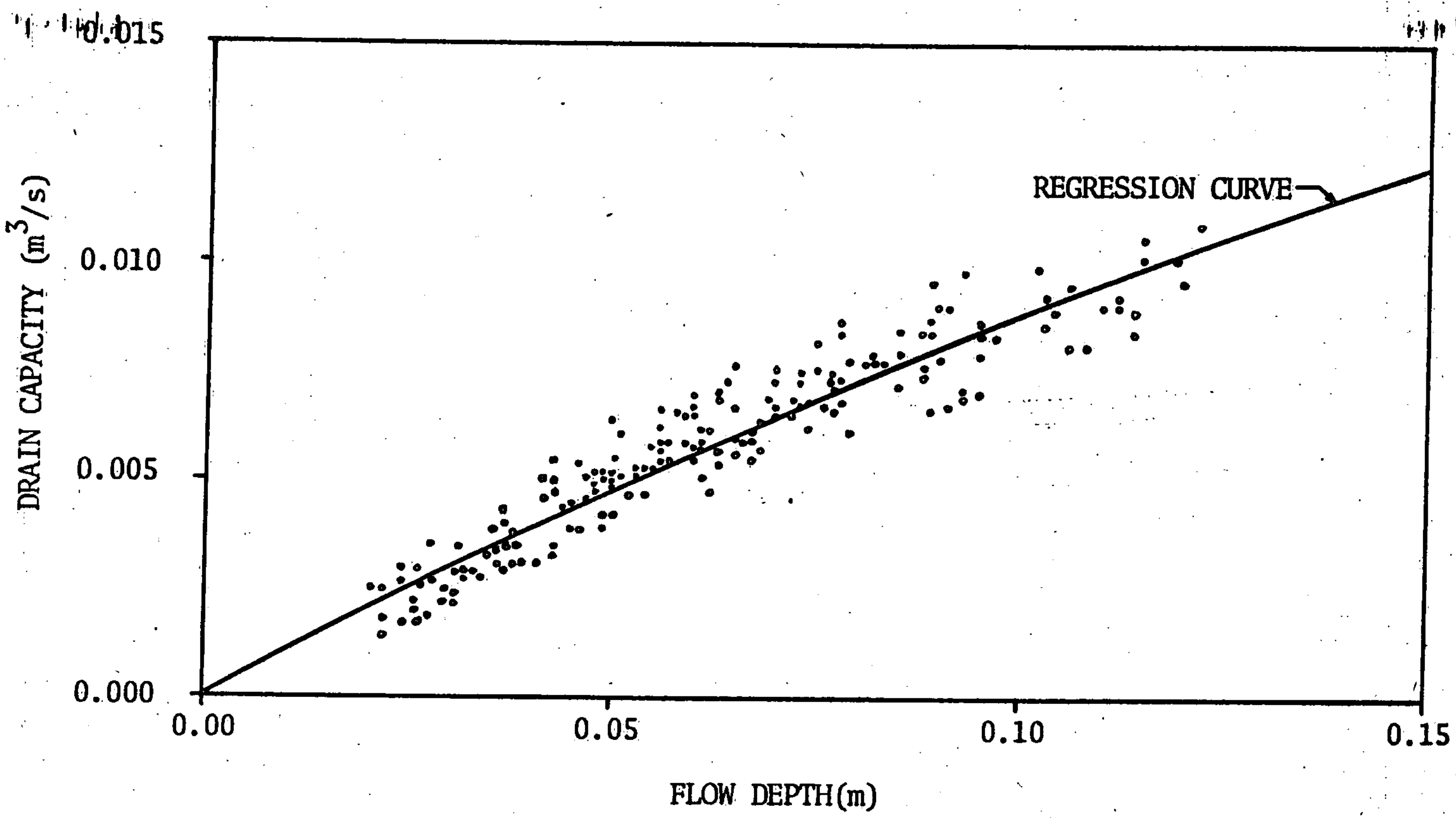
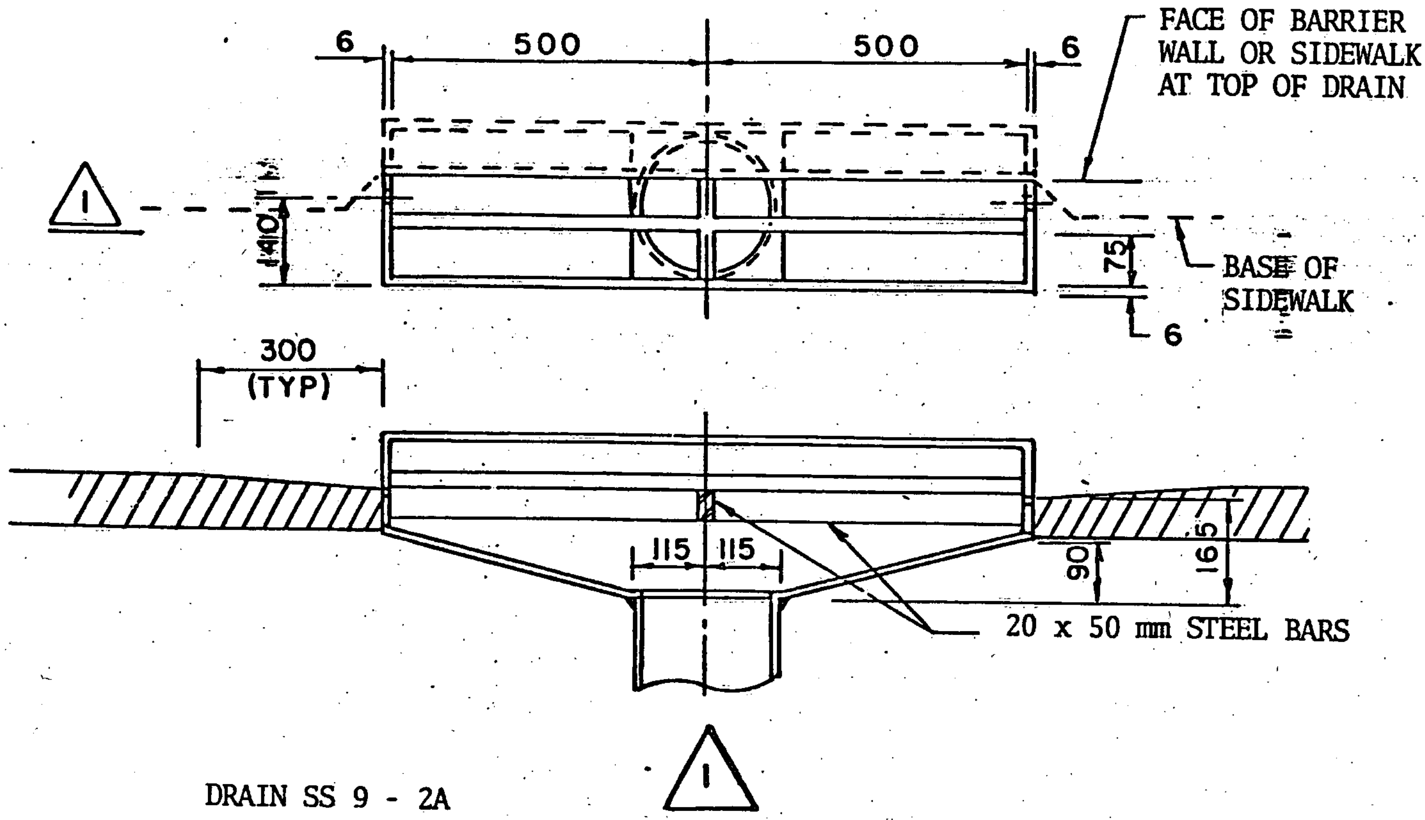
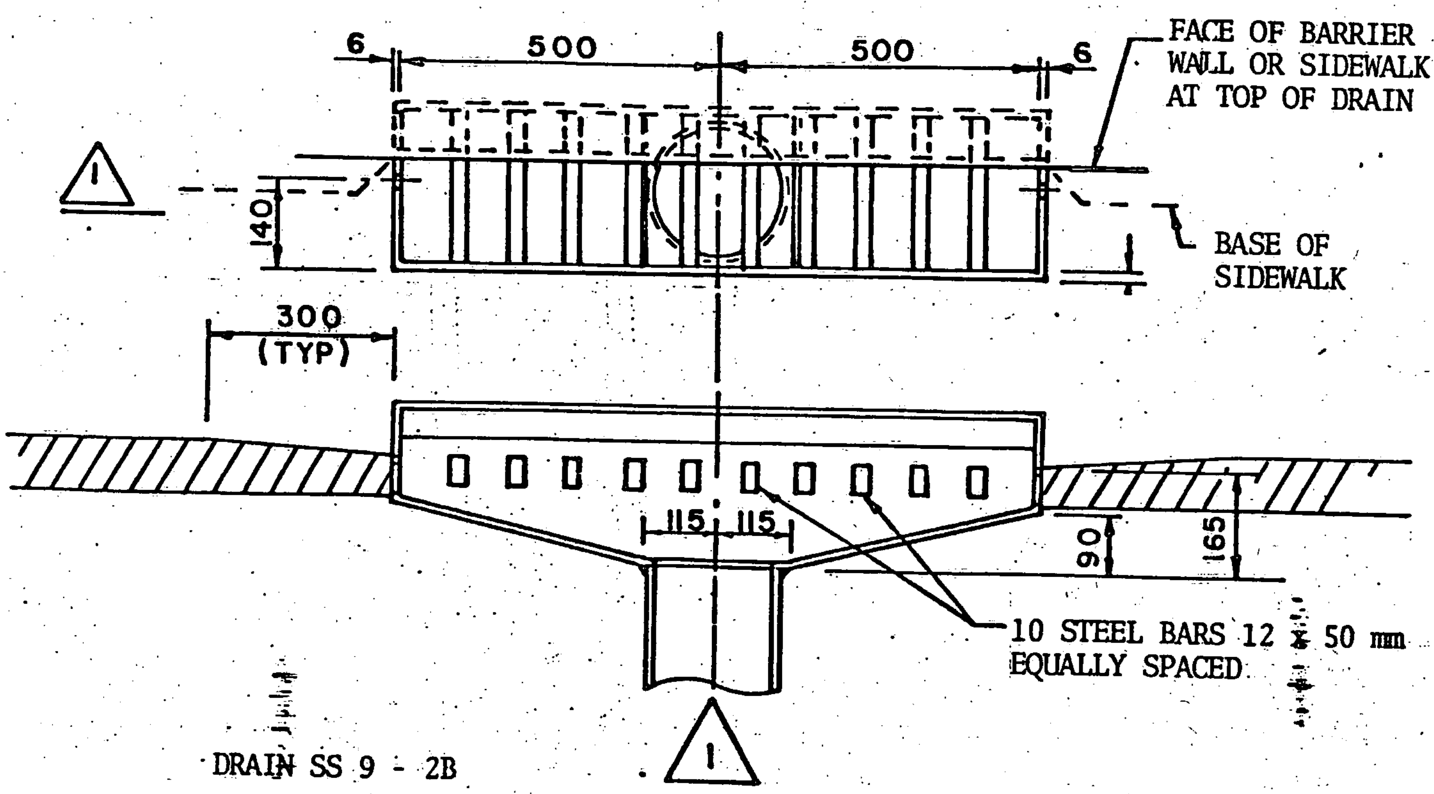


Fig.2. HYDRAULIC CAPACITY CURVE FOR DRAINS SS 9 - 1A AND SS 9 - 4A



DRAIN SS 9 - 2A



DRAIN SS 9 - 2B

Fig.3. DRAINS SS 9 - 2A and SS 9 - 2B

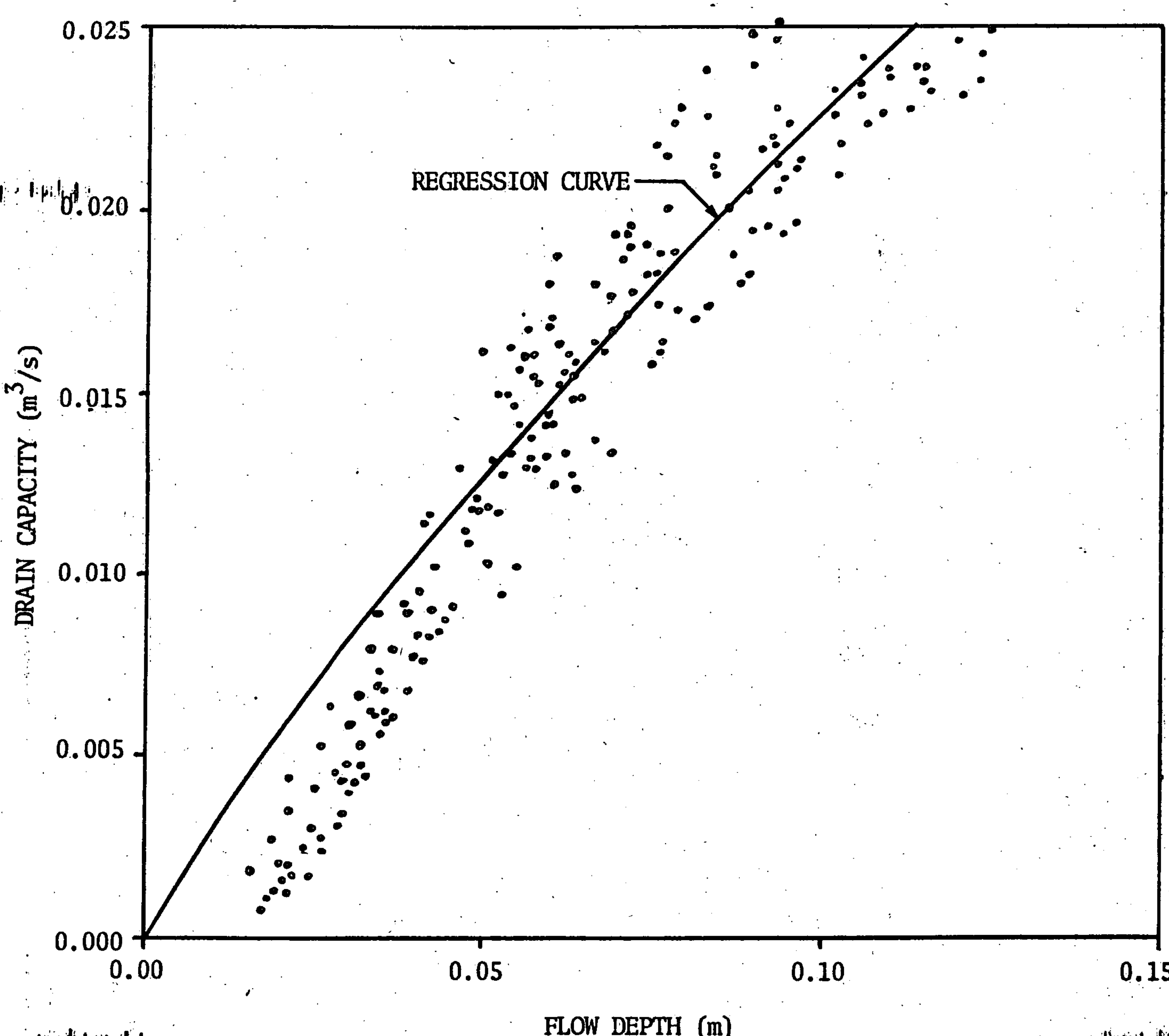
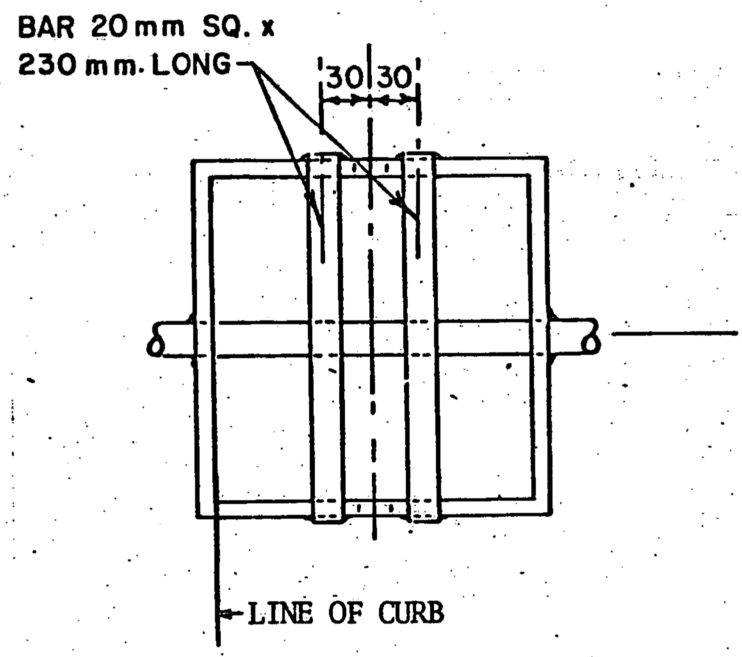


Fig.4. HYDRAULIC CAPACITY CURVE FOR DRAINS SS 9 -2A AND SS 9 - 2B



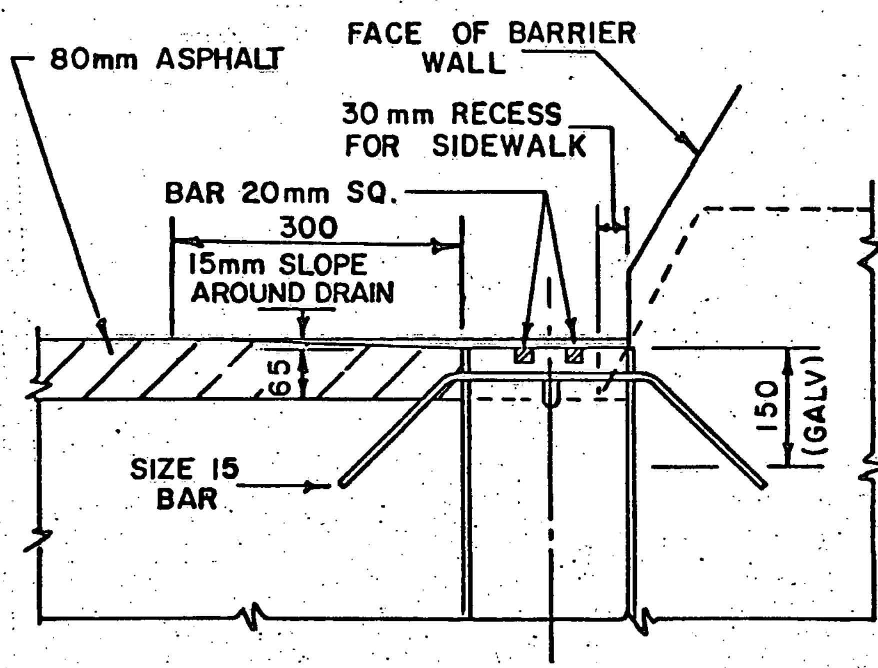


Fig.5. DRAIN SS 9 - 6A

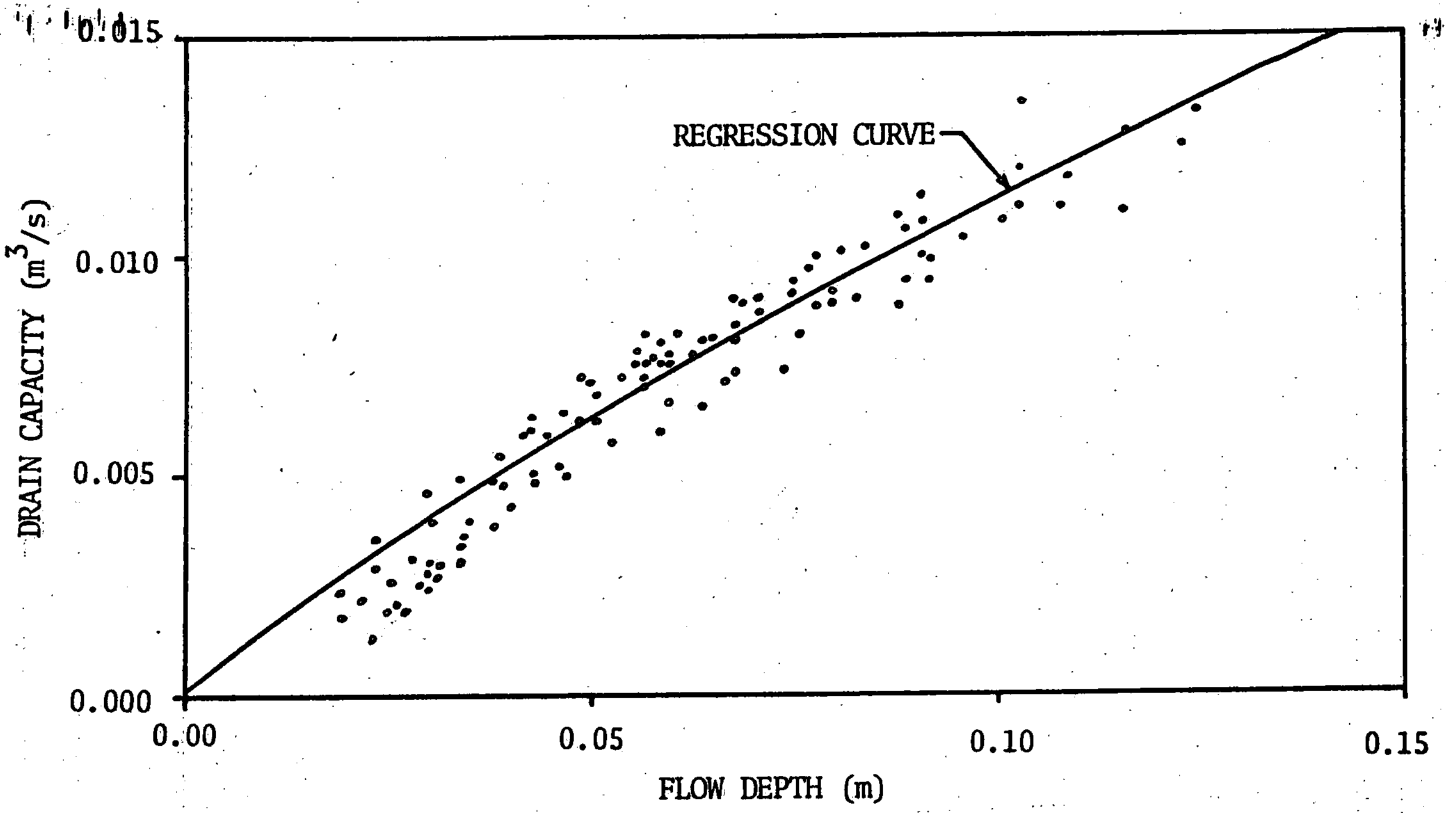
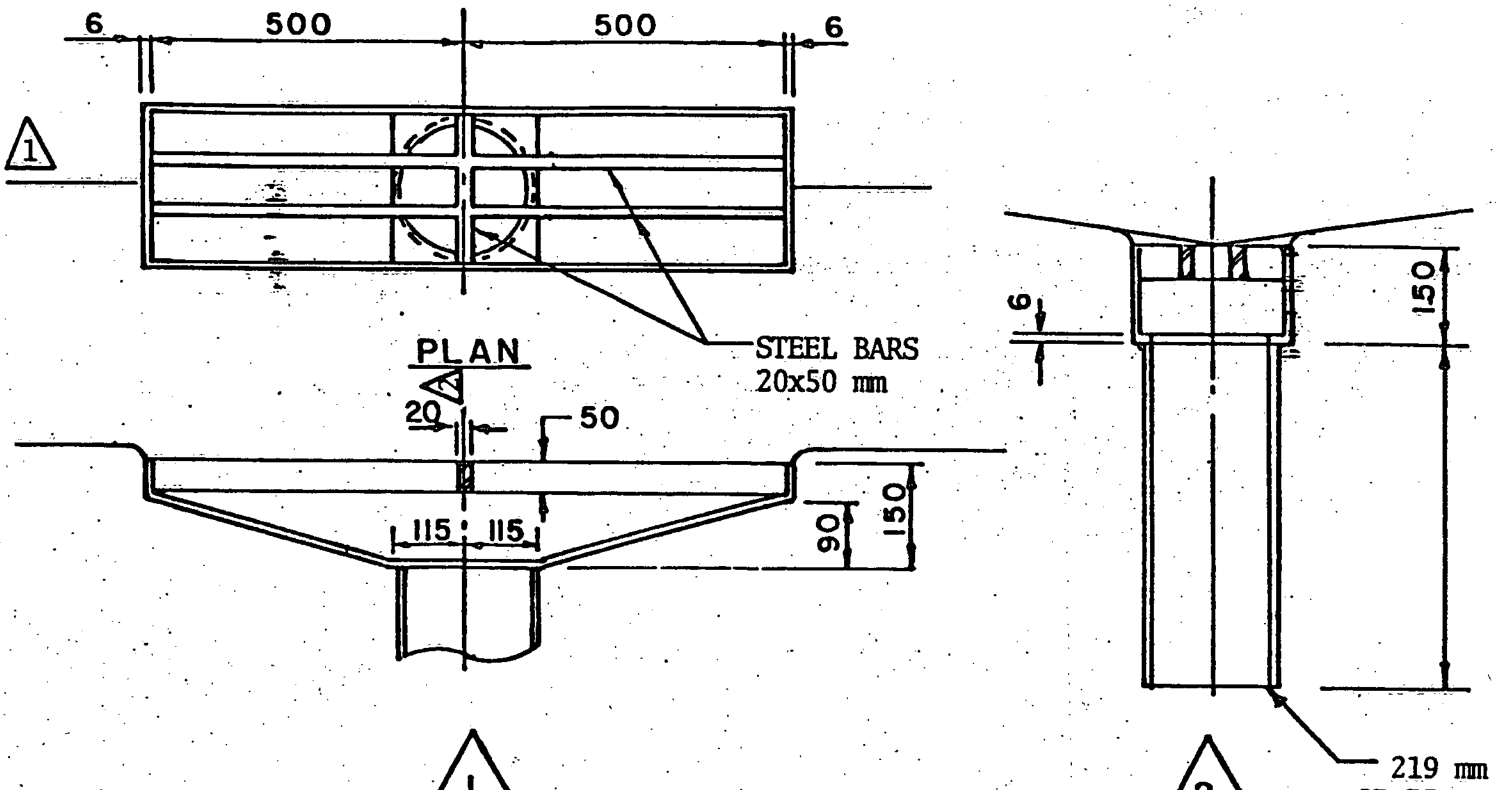
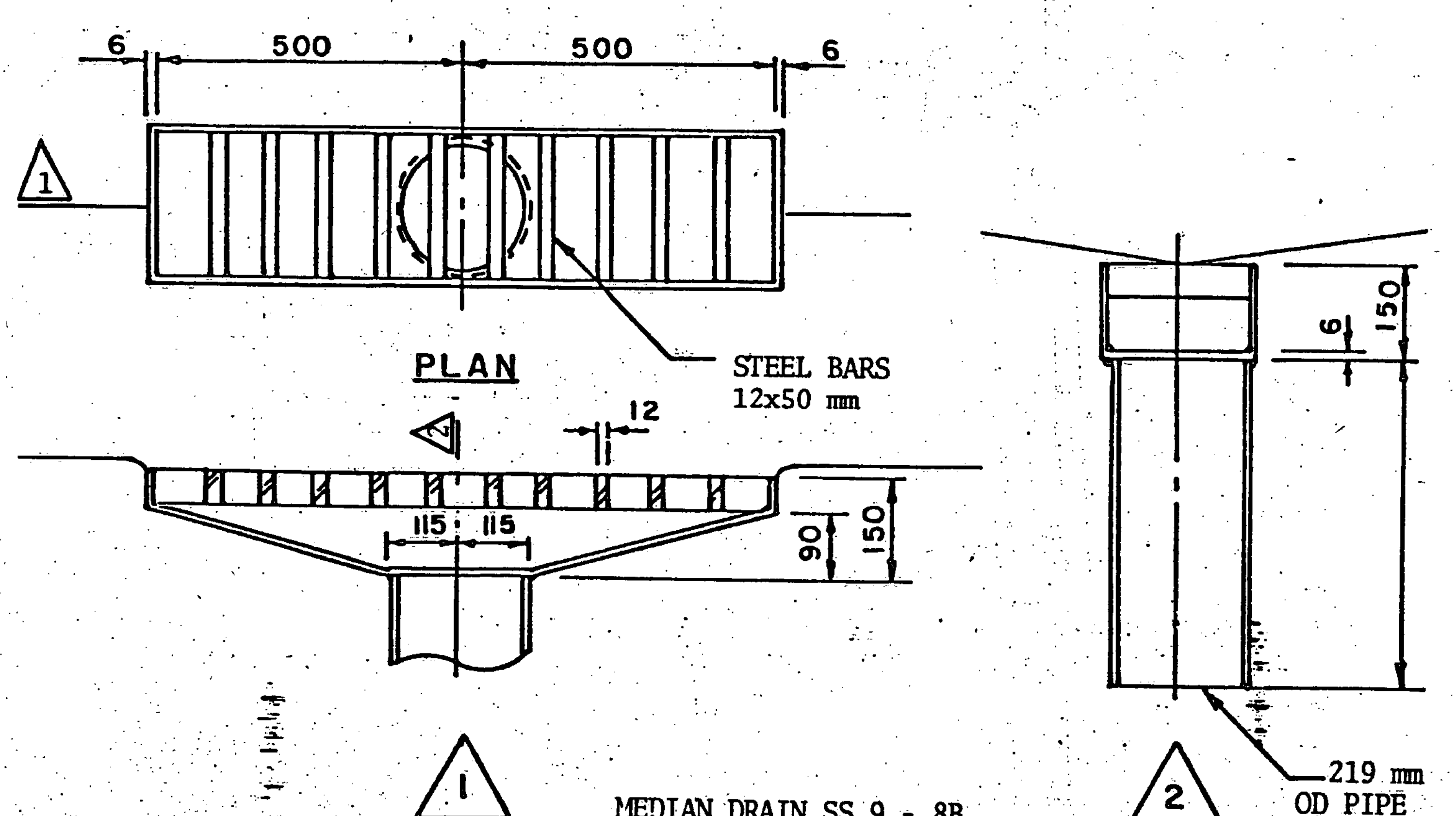


Fig.6. HYDRAULIC CAPACITY CURVE FOR DRAIN SS 9 - 6A



MEDIAN DRAIN SS 9 - 8A



MEDIAN DRAIN SS 9 - 8B

Fig.7. MEDIAN DRAINS SS 9 - 8A AND SS 9 - 8B

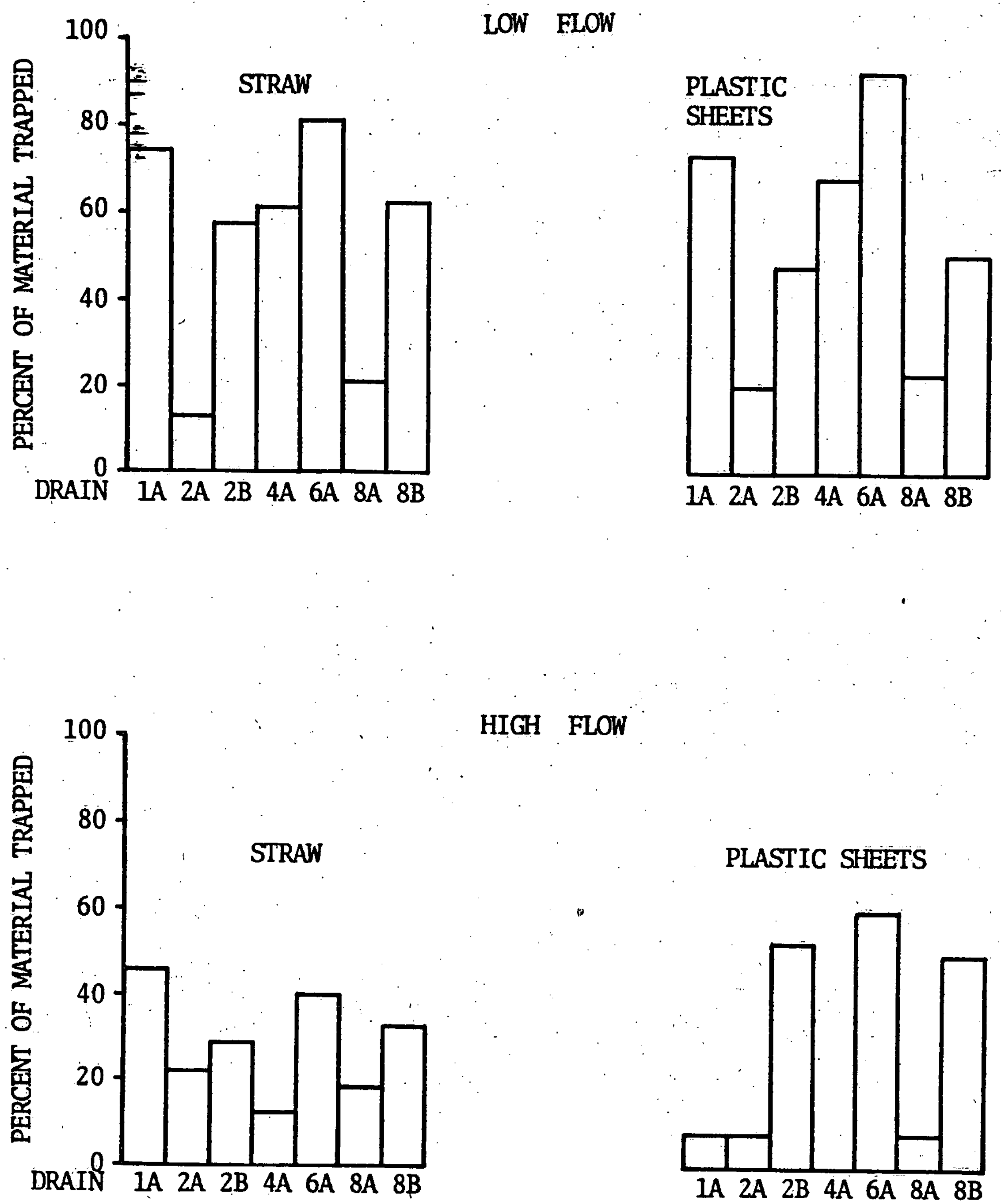


Fig.8. TRAPPING OF FLOATING DEBRIS BY DRAINS TESTED

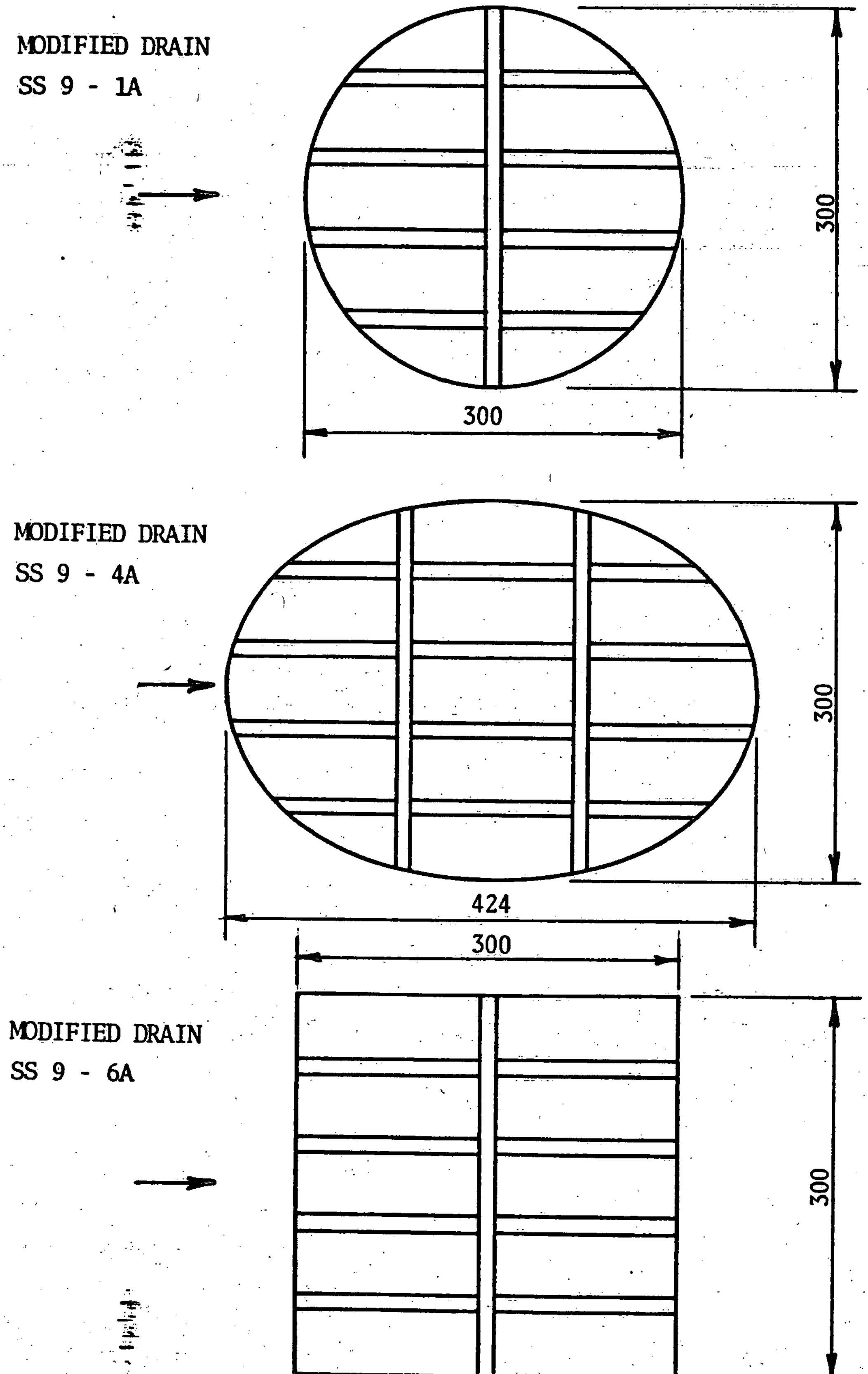
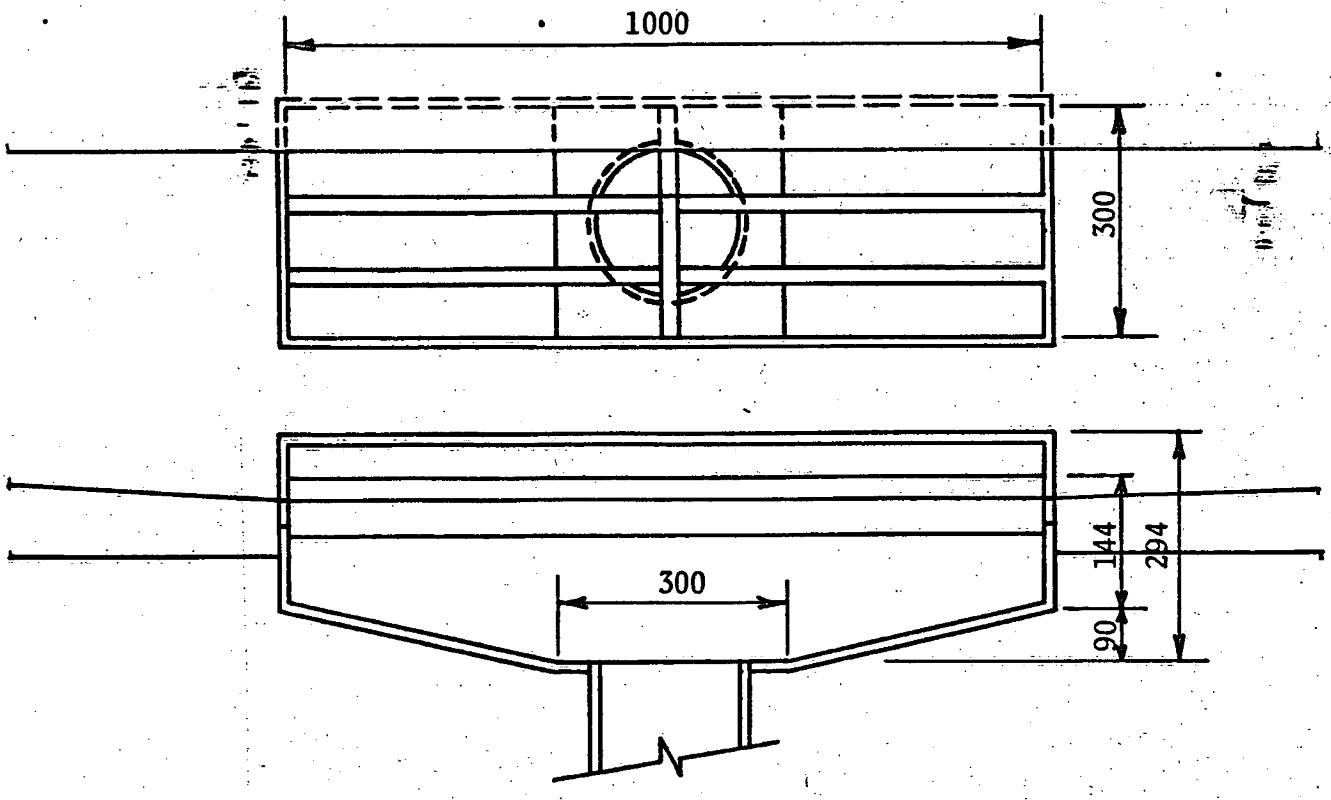


Fig.9. NEW DRAIN DESIGNS DERIVED FROM DRAINS SS 9 -1A, SS 9 - 4A, AND SS 9 -6A

MODIFIED
DRAIN SS 9 - 2A



MODIFIED DRAIN
SS 9 - 2B

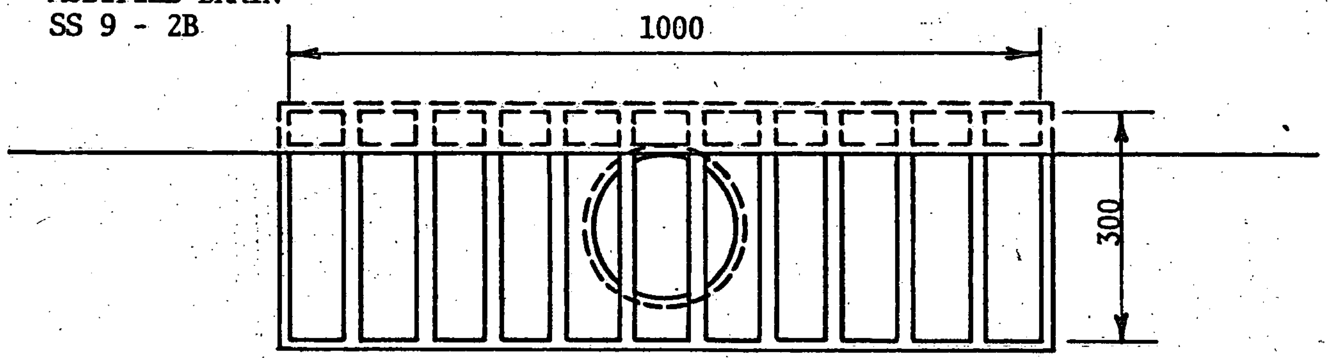


Fig.10. NEW DRAIN DESIGNS DERIVED FROM DRAINS SS 9 - 2A AND SS 9 - 2B

L_1 = First Drain Spacing

L_c = Consecutive Drains Spacing

W = Drained Area Width

T = Pavement Spread

L_T = Total Deck Length

L_{dT} = Deck Length Measured to the Last Drain

Q_r = Road Flow

L_r = Remaining Length Past the Last Drain

Q_d = Drain Discharge (Capacity)

Q_c = Carryover Flow

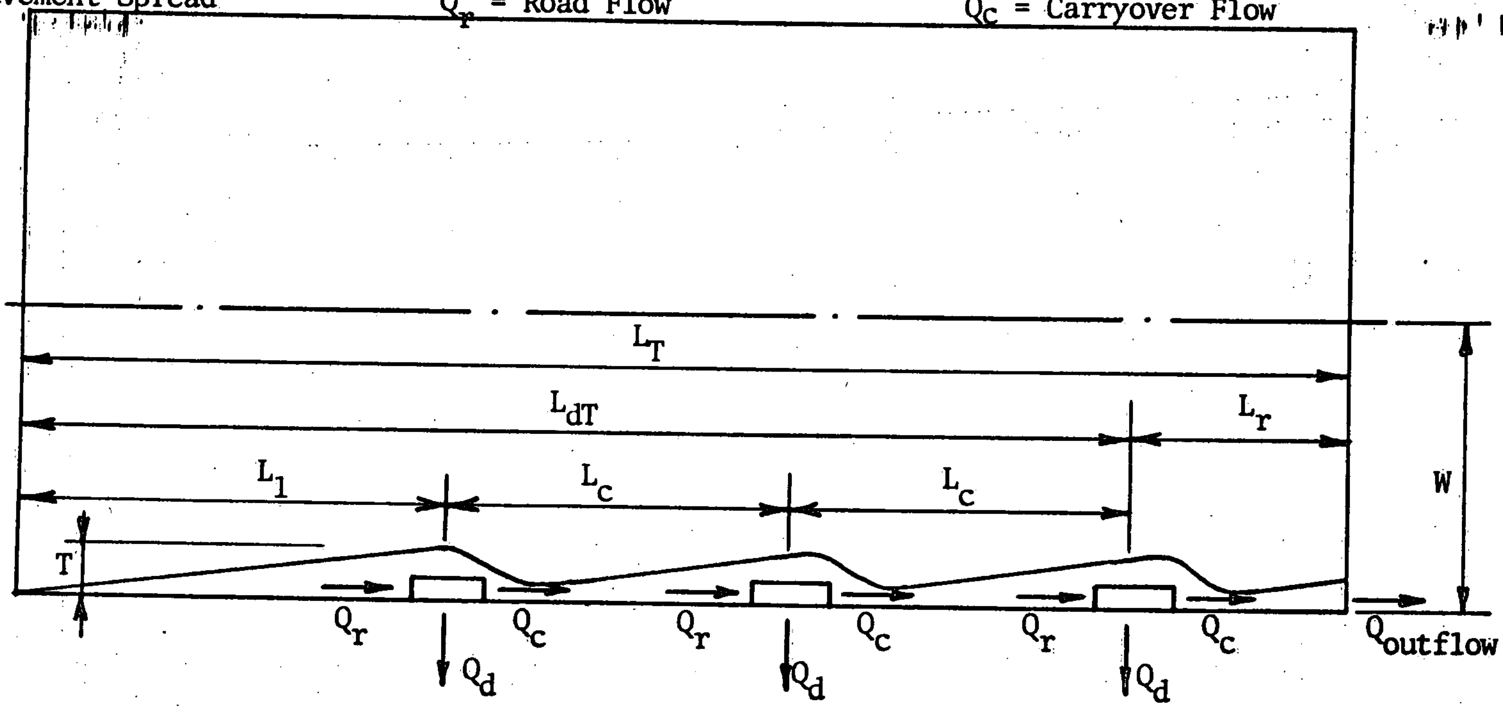


Fig.11. NOTATION SKETCH

PROGRAM FOR DESIGN OF BRIDGE DECK DRAINAGE BASED ON A DESIGN SPREAD

```
0: fxd 4
1: prt "    *****
 ***)*    "
2: spc
3: enp "Dr.Area
 Length[m]?",L
4: enp "Dr.Area
 Width[m]?",W
5: enp "Design
 Spread[m]?",T
6: enp "Crosfall
 ?",F
7: enp "Grade?",
 G
8: enp "Mannings
  n?",N
9: enp "Runf.Coe
 ff?",C
10: enp "Rainf.I
 nt.[mm/hr]?",I
11: enp "Drain
 No.?",D
12: enp "Drain
 Clogging[%]?",R
13: spc
14: prt "     ***
 *    "
```

Enter Data

L = length of the area to be drained

W = width of the area to be drained

T = design spread

F = crossfall

G = grade

N = Manning's n

C = runoff coefficient

I = design rainfall intensity(typically t=15 min.,10-year)

D = drain type; 1=SS 9-1A, 2=SS 9-2A, 3=SS 9-2B, 4=SS 9-4A, 5=SS 9-6A

R = percent of drain blocked by debris(=percent of capacity reduction)

```
15: F*T→r1
16: if D=2;gto
 21
17: if D=3;gto
 21
18: if D=5;gto
 23
```

r1 = flow depth in a triangular channel corresponding to spread T and crossfall F

depending on the drain type,directs control to an appropriate equation

```
19: .00013+.0991
 *r1-.1185*r1↑2→
 Q
20: gto 24
```

$Q_{d\ 1,4}$ = f(r1) drain capacity,eq.(1);drains SS 9-1A and 4A

```
21: .00039+.2684
 *r1-.4396*r1↑2→
 Q
22: gto 24
```

$Q_{d\ 2,3}$ = f(r1) drain capacity,eq.(2);drains SS 9-2A and 2B

```
23: .00018+.131*
 r1-.1894*r1↑2→Q
```

$Q_{d\ 5}$ = f(r1) drain capacity,eq.(3);drain SS 9-6A

```
24: .375*√G*r1↑2
 .667/(N*F)→r3
25: if Q>r3;r3→Q
26: r3-Q→P
27: (1-R/100)*
 Q→Q
```

r3 =road flow rate (triangular channel,grade G,crossfall F)
If drain capacity exceeds the road flow,set equal to road flow
P = carryover flow = road flow - drain capacity
Calculates reduced drain capacity for various degree of blockage

```
28: 1350000*√G*
 T↑2.667*F↑1.667
 /(C*I*W*N)→r11
```

r11 = L_1 = first drain spacing,eq.(10)

```
29: 3597000*Q/
 (C*I*W)→r12
```

r12 = L_c = consecutive drains spacing,eq.(13)

```
30: 0→J
31: J+1→J
```

J = a counter

```
32: r11+J*r12→r13
```

Keeps adding drains till the total length to be drained is just exceeded

`33: if r13<L; gto 31`	
`34: J-1→K`	
`35: r11+K*r12→r14`	r14 = L_{dT} = Deck length measured to the last drain
`36: L-r14→r15`	r15 = L_r = Remaining length past the last drain
`37: .000000278* C*I*W*r15→r16`	r16 = Q_{pLr} = runoff from the remaining length
`38: r16+P→r17`	r17 = $Q_{outflow}$
`39: if D=2;gto 45`	Directs control for printout
`40: if D=3;gto 47`	
`41: if D=4;gto 49`	
`42: if D=5;gto 51`	
	Printout
`43: prt "Drain No.SS9-1A"`	Identifies the drain type
`44: gto 52`	
`45: prt "Drain No.SS9-2A"`	
`46: gto 52`	
`47: prt "Drain No.SS9-2B"`	
`48: gto 52`	
`49: prt "Drain No.SS9-4A"`	
`50: gto 52`	
`51: prt "Drain No.SS9-6A"`	
`52: prt "Qroad[m3/s]",r3`	r3 = Road flow rate
`53: fxd 2`	
`54: prt "Max.Spread[m]",T`	T = design spread
`55: fxd 3`	
`56: prt "Max.Depth[m] ",r1`	r1 = flow depth at the curb (corresponds to spread T)
`57: fxd 4`	
`58: prt "Qdrain[m3/s]",Q`	Q_d = drain capacity (for spread T)
`59: prt "Qcarryov.[m3/s]",P`	P = Qc = carryover flow rate
`60: spc 1`	
`61: fxd 1`	
`62: prt "1st Spacing[m]",r11`	r11 = L_1 = first drain spacing
`63: prt "2nd Spacing[m]",r12`	r12 = L_c = consecutive drains spacing
`64: fxd 0`	
`65: prt "Number of Drains",J`	J = total number of drains
`66: fxd 1`	
`67: spc 1`	
`68: prt "Drained L[m]=",r14`	r14 = L_{dT} = deck length measured to the last drain
`69: prt "Remainder[m]",r15`	r15 = L_r = remaining length past the last drain

```
70: fxd 4
71: prt "Rem.Run
 off[m3/s]",r16
72: prt "Outflow
 [m3/s]",r17
73: spc
74: prt "   ***
 *******
75: stp
76: end
*953
```

r16 = Q_{pLr} = runoff from the remaining length

r17 = $Q_{outflow}$

END

PROGRAM FOR BRIDGE DECK DRAINAGE DESIGN

SAMPLE RUN

INPUT DATA

```
**          **

Dr.Area Length[m
]?
1000
Dr.Area Width[m]
?
9
Design Spread[m]
?
1.5
Crosfall?
.02
Grade?
.01
Mannings n?
.013
Runf.Coeff?
.95
Rainf.Int.[mm/hr
]?
100
Drain No.?
2
Drain Clogging[%
]?
0
```

OUTPUT

```
Drain No.SS9-2A
Qroad[m3/s]
          0.0125
Max.Spread[m]
            1.50
Max.Depth[m]
           0.030
Qdrain[m3/s]
          0.0080
Qcarryov.[m3/s]
          0.0045

1st Spacing[m]
            52.7
2nd Spacing[m]
            33.9
Number of Drains
              28

Drained L[m]=
           966.7
Remainder[m]
            33.3
Rem.Runoff[m3/s]
          0.0079
Outflow[m3/s]
          0.0124

     **********
```

EQUATION

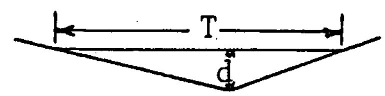
$$Q = 0.375 \frac{Z}{n} S^{1/2} d^{8/3}$$

Q = triangular channel discharge
Z = reciprocal of crossfall
n = Manning's roughness coefficient
S = channel slope; d = flow depth

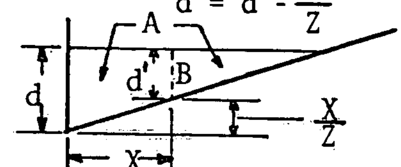
INSTRUCTIONS

1. Connect Z/n ratio with slope S; intersect with turning line connect with depth d to obtain discharge Q

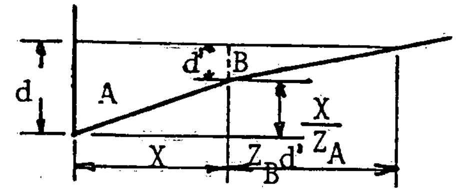
2. For shallow V-shaped channel shown below use Nomograph with $Z=T/d$

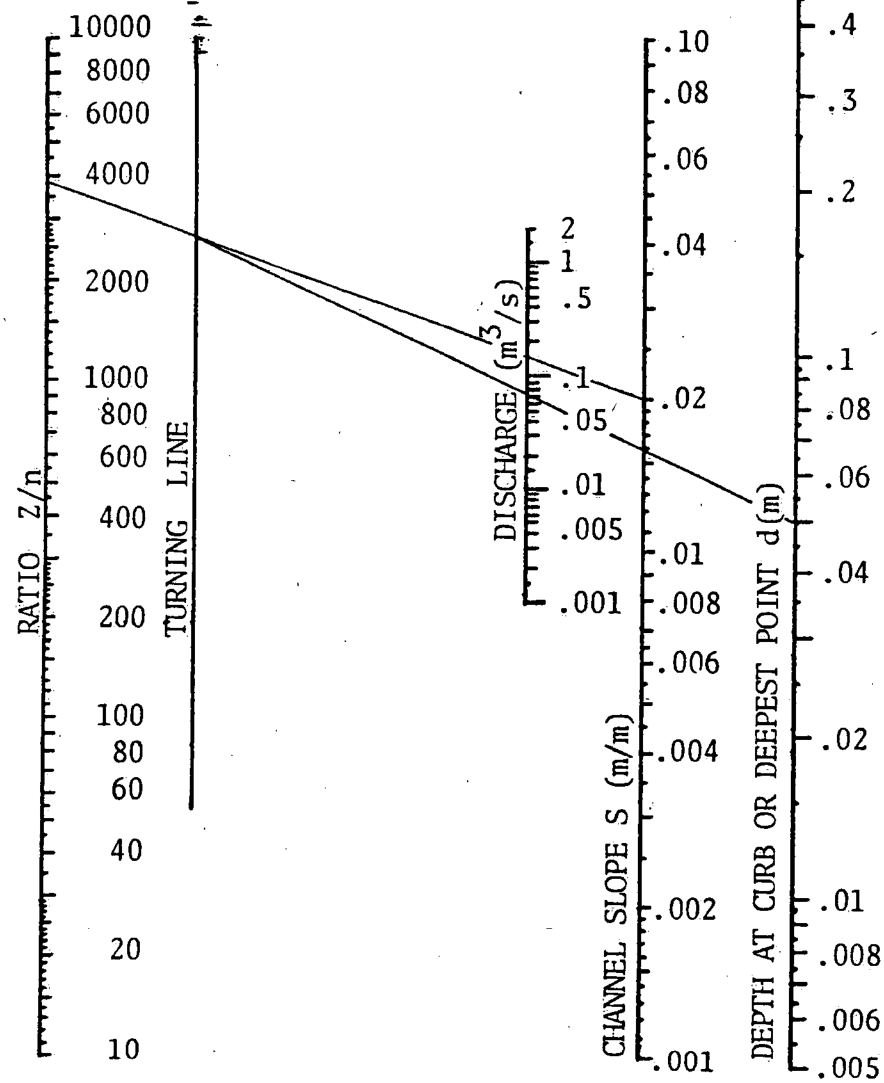


3. To determine discharge Q_x in portion of channel having width x : determine depth d for total discharge in the entire section A. Then use Nomograph to determine Q_B for depth $d' = d - \frac{X}{Z}$



4. To determine approximate discharge in composite section : Follow instructions 3 to obtain discharge in Section A at assumed depth d; obtain Q_B for slope ratio Z_B and depth d', then $Q_T = Q_A + Q_B$





Example

Given: S = 0.02, Z = 50, n = 0.013, Z/n = 3846, d = 0.05 m
Find : Q = 0.070 m^3/s

NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS
(Based on chart by U.S. Federal Highway Administration)

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