Surface Water Channels and Outfalls: Recommendations on Design

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HR Wallingford

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Contract

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Summary

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This report describes a research project funded by the Department of Transport which was carried out at HR Wallingford in association with the Transport Research Laboratory (TRL) from September 1992 to December 1994. The objectives of the study were: 1. to review the current situation and research needs concerning the use of surface water channels for road drainage; and 2. to prepare recommendations on outlet design and general recommendations on the use of surface water channels.

To achieve the first of these objectives two questionnaires were produced and several site visits were carried out. The first, and more general, questionnaire was circulated to regional operating units of DOT in England and to corresponding organisations in DOE Northern Ireland, and the Scottish and Welsh Offices. A very positive response was obtained from these organisations: general information was collated from 39 schemes that had already been built or were at various stages between design and construction. The information gathered concerned the geometric characteristics of the surface water channels, the type and gradient of the road, and the type and size of the outfalls. A second, more detailed, questionnaire was prepared and used as a check list during site meetings to obtain qualitative information about the experience of road engineers in designing, constructing and maintaining surface water channels.

In order to achieve the second objective of the study an extensive testing program was carried out involving field measurements of outlet performance in two existing road schemes and laboratory tests of a number of outlet designs. The two sites selected for the field tests were the A20 Folkestone to Dover (Contract 1), in Kent, and the A487 Port Dinorwic Bypass, in Gwynedd, Wales. These sites offered a variety of channel geometries and outlet designs, as well as a wide range of longitudinal slopes, and the tests were carried out with various flow conditions. The most important conclusion from the field measurements concerns the performance of existing outlet designs in schemes where the longitudinal gradient of the road is very steep. In these situations it is likely that considerable flow bypassing of the outlet will occur under channel-full conditions, and therefore an outlet specifically designed for these cases may be required.

The recommendations for the hydraulic design of outfalls were mainly developed from the laboratory tests since these allowed a more systematic way of varying the geometry of the outlets and the flow conditions. It was agreed with DOT to test outlets in symmetrical triangular channels with cross-falls of 1:5 and in a higher capacity channel of trapezoidal cross-section, with

Summary Continued

a design depth of 0.150m, base width of 0.300m and cross-falls of 1:4.5. This trapezoidal channel provides an increase in capacity of 45% in relation to a triangular channel of the same depth and cross-falls of 1:5. Two types of outlet were studied, according to their position along the channel: intermediate outlets, which are located at points part-way along a length of channel, and terminal outlets located at low points. Both in-line and off-line outlet designs were tested for each of the two types of channel. Tests were also carried out to determine the hydraulic performance of a type of outlet which is suitable for very high velocity flows such as those occurring in steep roads. This design consists of a side transition which gently directs the water away from the carriageway onto the verge side and then over a side weir into a lower collecting chamber. The test results, as well as the information obtained from the questionnaires and the site visits, were used to produce design recommendations in a draft Advice Note on Outfall Design. The Advice Note is presented in Appendix III of this report.

The main conclusions drawn from the study and recommendations for further work are presented in the last part of the report. These refer to the design, construction and use of surface water channels and to further work that is required for revision of the existing Advice Note HA 37/88.

List of Symbols

A	Cross-sectional area of the flow
B	Water surface width
B_o	Surface width of flow for channel-full
B_1	Surface width of flow in surcharged channel neglecting the width of surcharge on hard strip or hard shoulder
D	Pipe diameter
E	Width of collecting channel for weir outlet
F	Froude number
F_a	Froude number upstream of disturbance
F_o	Non-dimensional number for channel-full
F_s	Non-dimensional number for surcharged channel
G	Grating width
g	Acceleration due to gravity
h	Water depth of flow
h_p	Water depth of bypass flow
J	Depth of collecting channel for weir outlet
K	Coefficient
L_a	Angled stretch of weir outlet
L_s	Straight stretch of weir outlet
L_w	Total length of weir outlet
n	Manning roughness coefficient
P	Wetted perimeter of channel
Q	Flow rate
Q_i	Intercepted flow
Q_o	Approach flow for channel-full conditions
Q_p	Bypassing flow
Q_s	Approach flow for surcharged conditions
R	Hydraulic radius of channel
S	Longitudinal gradient
V	Mean flow velocity
y_a	Water depth upstream of oblique wave
y_b	Water depth downstream of oblique wave
y_1	Depth of flow from the lower edge of the carriageway
y_2	Depth of the channel from the upper edge of the carriageway
y_3	Overall depth of surcharged channel
Z	Head of water above pipe invert
β	Angle of oblique wave in relation to direction of flow
η	Efficiency
η_D	Efficiency of outlet for gratings with diagonal bar pattern
η_L	Efficiency of outlet for gratings with longitudinal bar pattern
η_o	Efficiency of outlet for channel-full conditions
η_s	Efficiency of outlet for surcharged conditions
θ	Angle of weir outlet
θ_p	Predicted angle of weir outlet

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PART A SCOPE OF STUDY

A.1 GENERAL INTRODUCTION

Since 1989, the Highways Agency of the Department of Transport (DOT) has promoted the use of purpose-built surface water channels as an alternative to filter drains or kerb-and-gully systems for dealing with rainfall run-off from roads. In principle the channels offer several advantages over the two older methods of drainage. Firstly, they are very accessible so that any blockages are visible and can be quickly dealt with. Secondly, the channels enable the surface water to be kept quite separate from the system draining the sub-base allowing the smaller seepage flows to be collected by narrow fin drains along the edge of the road. The separation of the two systems also prevents the possibility of surface water flowing back into the sub-base, as can happen if filter drains become surcharged. Thirdly, the channels do not interrupt the continuity of the road construction, unlike kerb gullies which can produce cracks in the pavement through which surface water can enter the sub-base. Fourthly, surface channels generally have a much higher flow capacity than the shallow triangular gutters formed by kerbs and the cross-fall of the road. As a result, the distance between outlets in surface channels can be much greater than with conventional kerb-and-gully systems.

A method of determining the capacity of surface channels and the required spacing between outlets was provided by the publication of DOT Advice Note HA 37/88. The method is based on kinematic wave theory and takes account of flow variations with time during a storm and the interrelation between rainfall intensity, storm duration and frequency of occurrence. Surface channels are most suited to slip-form construction in mass concrete and recommendations on their geometry and use are given in Highway Construction Details Series B and DOT Advice Note HA 37/88. When the first schemes with channels were constructed (starting with the A21 Pembury Bypass in 1988), many were designed and built with triangular channels having a maximum depth of 150mm and a side-slope of 1:5 (vertical:horizontal) on the carriageway side and 1:1 on the verge side; in the central reserve, the channels were required to have symmetrical side-slopes of 1:5. In 1991, Amendment No. 1 to HA 37/88 was published which specified that symmetrical side-slopes of 1:5 or flatter should be used both in the verge and the central reserve; local variations in side-slope to a maximum of 1:4 were permitted in special cases.

Information on the flow capacity of British Standard gully gratings was published in Contractor Report 2 (1984) but this document applies only to kerb-and-gully situations where the triangular gutter has a near-vertical kerb and the cross-fall of the road is not steeper than 1:20 (or in some cases 1:15). Conditions in surface water channels are more severe because the depth and velocities of flow can be significantly higher than in normal kerb-side gutters. In the absence of suitable capacity data, road engineers have produced a wide range of outlet designs for the surface water channels that have so far been constructed. In some cases, the gully gratings have been installed along the centreline of the channel ("in-line" type) and in others they have been offset towards the verge ("off-line" type). The number of gratings considered necessary at each outlet has also varied from scheme to scheme.

The Highways Engineering Division (now the Highways Agency, of the Department of Transport) therefore identified that a need existed for an Advice Note giving recommendations on the design of outfalls from surface water channels. It was anticipated that it would first be necessary to review the various designs that have already been built and collate the experience that has been accumulated by road engineers in the use of surface water channels. However, it was also appreciated that detailed laboratory testing and field measurements would be needed in order to develop suitable outfall designs and produce methods for determining their hydraulic performance. A contract for the research project was awarded by DOT to HR Wallingford, in association with the Transport Research Laboratory (TRL), in September 1992. The primary division of work involved HR (as Contractor) carrying out the hydraulic testing of the outfalls and the development of the design methods for the Advice Note, with TRL (as Sub-Contractor) providing specialist inputs concerning construction and safety aspects and the review of existing experience.

This final report of the project is divided into four main parts. Part A comprises this general introduction and details of the technical specification for the work. Part B describes the results of two surveys and several site visits that were carried out to collect data and experience on existing schemes with surface water channels. Part C describes the experimental work on outfall designs that was carried out in the laboratory at HR Wallingford and in the field on two road schemes. The text also explains how the data were analysed and used to produce the design recommendations contained in the draft Advice Note, a copy of which is included as Appendix III. Part D draws together the main results from Parts B and C and identifies what changes are necessary or desirable in other design documents relating to the use of surface water drainage channels.

A.2 TECHNICAL SPECIFICATION

The agreed technical specification for the project resulted from a combination of the original DOT Work Specification (Appendix A in the contract documents) and certain alternative items which were proposed in the HR/TRL tender submission and accepted by DOT on 15 September 1992. A limited extension to the contract was also agreed on 23 July 1994 to cover the development of an additional type of outfall for use with steep channels.

The objectives of the research were to:

- review the current situation and research needs concerning the use of surface water channels.
- prepare recommendations on outlet design together with general recommendations on the use of surface water channels.

The items of work and the methods needed to achieve these objectives can be summarised as follows:

(1) Review of existing schemes

Collect and collate information on channel geometry in use or at the design or construction stages, and make recommendations on the applicability of the channel drainage system, describing the advantages and disadvantages during design, construction and maintenance.

The work method proposed by HR/TRL involved use of two separate questionnaires and site visits to at least five schemes. The first questionnaire was needed to obtain general geometric data about as many existing schemes with surface water channels as possible. The second questionnaire was required during the site visits to record more qualitative information about the experience of road engineers in designing, constructing and maintaining the channels.

(2) **Testing of outfall designs**
Consider a wide range of outfall designs and measure the performance of existing outfalls on the road network and, if necessary, using specialist laboratory equipment.

The work method proposed by HR/TRL emphasised the use of laboratory tests because of the need to carry out accurate measurements under a wide range of controlled conditions. Allowance was initially made for testing three outfall designs with each of the two agreed channel geometries; an outfall design for steep channels was added later. Also included was field testing of two outfalls from existing road schemes in order to provide data for comparison with the laboratory results.

(3) **Design recommendations**
Analyse results and produce Advice Note giving guidance on predicting performance of various outfall designs. Collect and collate experience on the best use of channels and produce Final Report to advise DOT on the best way forward.

The HR/TRL proposal was in accordance with these required outputs from the project.

PART B EXPERIENCE IN USE OF SURFACE WATER CHANNELS

B.1 INTRODUCTION

It was explained in Section A.2 that one of the objectives of the project was to collate information and experience on the use of surface water channels in existing road schemes. The information was needed in order to identify possible improvements in design and construction and to produce guidance on the applicability of channels to different types of drainage problem. Data about the various designs of outfall already in use were also needed when planning the laboratory and field tests.

The collection of the information about existing schemes therefore formed the first stage of the project and was carried out by means of two questionnaires and several site visits. Details held centrally by DOT on a number of projects with surface water channels were also made available.

The purpose of the first questionnaire was to obtain basic quantitative data about factors such as the size and shape of the channels, the slopes of the roads and the typical distances between outlets. Based on advice from the Highways Engineering Division of DOT, it was decided to circulate the questionnaire to the nine regional Operating Units in England and to the corresponding organizations in DOE for Northern Ireland, the Scottish Office and the Welsh Office. Each Operating Unit was requested to provide separate details for each of the schemes in its area with surface water channels that had either been built or were at any stage between design and construction. The Operating Units in turn passed the questionnaires to relevant organisations such as County Council highway departments and Consulting Engineers who held the required information for particular schemes. The main batch of questionnaires was sent out in December 1992 after approval by DOT and replies were received up until about July 1993. The questionnaire is described in Section B.2 where the results are also discussed.

It was realised that it would be difficult to obtain more qualitative information about experience in the use of surface water channels through the use of another impersonally-addressed questionnaire. It was therefore decided to select a range of representative schemes on the basis of the information from the first-stage questionnaire, and then arrange site meetings with the engineers involved in the design, construction and maintenance of the schemes. A second, more detailed questionnaire was prepared with DOT approval covering questions such as hydraulic design, construction tolerances and maintenance aspects. However, the questionnaire was only used as a check list during the site meetings, with the answers received being recorded by a member of the HR/TRL project team. The site visits were carried out between July and December 1993 and are described in Section B.3. The information obtained from the meetings and the second questionnaire are summarised and discussed in Section B.5. Overall conclusions about the applicability of channels to different types of surface drainage problem are presented in Section B.6.

B.2 GENERAL QUESTIONNAIRE

The questionnaires requesting information on road schemes using surface water channels produced positive replies from seven of the nine DOT Operation Units in England. No reply was obtained from the Northern Construction Programme Division (CPD), and no schemes are known to include surface water channels in the jurisdiction of the London CPD. Questionnaires were later also sent to the DOE Office for Northern Ireland and to the Scottish and Welsh Offices, and these again produced a positive response, in particular from Wales.

A total of 39 schemes in the United Kingdom were reported to use surface water channels for drainage of the road surface. Two other schemes also included channels but were not considered in the present study: one made use of small semi-circular channel blocks, and the other of triangular channels located at the toe of the embankment which were fed by conventional kerb and gully drainage. Both these types are outside the scope of the present study. The questionnaire is presented in Appendix I and a summary of the replies is shown in Tables 1 to 4.

B.2.1 Overall description of the schemes (Questions A and D of the questionnaire)

The replies concerning the overall description of the schemes, which correspond to Questions A and D of the questionnaire are summarized in Table 1.

From the total number of schemes with channels, approximately half (23 schemes) had been completed at the time the questionnaires were answered (between December 1992 and July 1993); amongst the other 16 schemes, 10 were being constructed. The great majority of surface water channels has been built in rural roads (as opposed to urban): 20 of the schemes are dual carriageways, 10 are single carriageways, 5 are motorways, and two schemes correspond to a slip road and an urban road, respectively.

In all but 4 schemes the type of road pavement is flexible, the exceptions being composite pavements. No information was obtained regarding rigid pavements associated with surface water channels. In total, the length of the schemes where channels have been incorporated adds to about 250km, although not all this length corresponds to the actual length of the channels. For instance, in dual carriageways the channel length can be twice the length of the road if channels are used continuously. A precise value for the channel length can not be directly obtained from the responses to the questionnaire, but the total length of the schemes gives some indication of the present usage of this type of surface road drainage in the UK. As can be seen in the Tables, surface water channels appear to be more widely used in the East and South East regions of England and in Wales: 10 schemes were identified in the Eastern CPD, 8 in the South East CPD and 6 in Wales. The fact that the first two schemes to be built in the UK were the A21 Pembury Bypass and the A11 Thetford Bypass in the South East and East Anglia, respectively, may possibly account for the more rapid spread of surface water channels in these two regions.

The replies from the questionnaires show that surface water channels have been used in schemes varying from generally flat to generally steep longitudinal gradients. The maximum gradients reported varied between 1:12 and 1:250, whereas the minimum gradients varied from 1:100 to virtually flat. Figure 1a) is a chart showing the maximum longitudinal gradients in the road schemes. It can be seen that the majority of schemes have maximum gradients of the order of 3-4%

Regarding the type of sub-surface drainage, it was found that the most common systems were either fin drains or narrow filter drains, each type accounting for approximately 40% of the schemes. A combination of these two types was used in 5 of the schemes and two schemes incorporated narrow filter drains and also French drains. One scheme reported no sub-surface drainage due to the free draining nature of the subgrade (gravel).

B.2.2 Description of the channels (Question E of the questionnaire)

The replies concerning the description of the schemes, which correspond to Question E of the questionnaire, are presented in Tables 2 and 3.

In what regards the construction of the channels, slipforming appears to be by far the most commonly used method, corresponding to about 80% of the answers obtained. Combinations of slipform and in-situ concrete, and slipform and extruded concrete were reported in two cases. Precast channels were used in three schemes and in-situ concrete in another scheme.

Apart from two exceptions, all the channels described had a triangular cross-section which was either asymmetrical (mainly used in channels in the verge of the carriageway) or symmetrical (mainly used in channels in the central reserve). The range of side-slopes adopted for these triangular channels was quite wide: in the verge it varied from slopes as steep as 1:1 (verge side) and 1:2.3 (carriageway side) to a symmetrical channel with slopes of 1:5. In the central reserve the steepest channel recorded had cross-falls of 1:1 and 1:2.3, and the flattest was symmetrical with cross-falls of 1:20. However, most of the channels in the central reserve were designed to be symmetrical and to have slopes not steeper than 1:4. A trapezoidal channel with side-slopes of 1:1 was also reported, as well as a dished channel but no detailed information was given on the dimensions of this latter channel. It should be noted that the design of some of these schemes started before the publication in 1991 of the Amendment No.1 to the Advice Note HA 37/88 which recommended the use of symmetrical channels with cross-falls of 1:5 both in the verge and in the central reserve.

Another feature of the channels that varied considerably from scheme to scheme was the design flow depth of the channel. Some of the values of the design flow depth (and in a few cases of the channel width) given in Tables 2 and 3 were amended in order to be consistent with the values of the side-slopes and design width given in the replies to the questionnaires. Figure 1b) shows the channels (in the verge and in the central reserve) identified in the survey according to their depth. It can be seen that channels of depth between 100 and 150mm appear to dominate.

B.2.3 Description of channel outfalls (Question F of the questionnaire)

The replies concerning the description of the channel outfalls, which corresponds to Question F of the questionnaire, are summarized in Table 4.

The replies to the questionnaire showed that in approximately two thirds of the schemes the outlets were set in the channel invert and in one third these were set back in the verge of the carriageway. One scheme reported both types of positioning.

Figure 2 shows two bar charts with the minimum and maximum distances between outlets that were adopted in the schemes. The outlets were spaced as far apart as 1100m in one scheme, but the minimum distances reported were as short as only 5m, presumably in sag points and at crests. The maximum distances between outlets varied from 30 to 1100m, whereas the minimum distance varied between 5 and 364m. The chart for maximum distances shows that in almost 30% of the schemes the outlets were spaced more than 500m apart; however, in half the schemes this distance was less than 150m.

A variety of systems has been used to convey the water from the outlets in the schemes reported, but the majority of the schemes relied on carrier drains or a combination of these with other systems, as shown in Figure 3. Toe ditches, watercourses and soakaways have been used in conjunction with carrier drains in about 20% of the schemes. Soakaways as the sole means of discharging the flow were adopted in two schemes, and the exclusive use of toe ditches was adopted in only one scheme.

B.3 SITE VISITS TO SELECTED SCHEMES

B.3.1 Considerations

Visits were made during the project to thirteen different road schemes with surface water channels. The schemes are listed in Table 5 and were selected on the basis of the information from the first-stage questionnaire so as to cover a representative range of channel and outlet types. Ten of the schemes were already open to traffic and three were under construction. Eight of the schemes that were open were visited without requiring the involvement of local highways staff. The five other schemes were visited in the company of site staff either as part of the meetings carried out for the second-stage questionnaire (see Section B.4) or during the field tests on outlet performance (see Section C.2). The principal objectives of the visits were to: assess the design and construction aspects of the surface water channels; identify any problems with the durability or maintenance of channels already in use; and identify and record the main types of outlet design already in use.

B.3.2 Data from visits

During the visits particular attention was focused on the following aspects: the general state of the channels (ie concrete roughness, contraction joints, sedimentation, debris, plant overgrowth); the general state of the gratings (ie blockage by debris); the geometry of the channels and outfalls; the position of the outfalls in relation to the channel invert. The size of the step between the carriageway side of the channels and the pavement surface was also noted as well as any signs of cracking in the channels. Photographs were taken of each scheme visited, and the dimensions of the outfalls were recorded in order to supplement the information gathered from the first-stage questionnaire. Some general conclusions have been drawn from these visits and are summarised under the following headings:

(a) Overall state of channels and outfalls
From the structural point of view the overall condition of the channels and outfalls of all the schemes visited was generally good. Small cracks could, however, be seen to have developed in most channels, but these should not, in the short-term at least, affect the performance of the channels on conveying run-off from the road. This conclusion applies even to a scheme (Reference J in Table 5) where signs of an accident were still visible at the time of the visit. A vehicle had apparently crossed over the central reserve channel at an angle of about 45° to the edge and crashed against the safety barrier, but only limited damage had been caused to the channel. By chance, the vehicle impacted at the position of a gully grating, and the shock appeared to have cracked the concrete around the grating and shifted it slightly.

(b) Concrete roughness
The roughness of the concrete varied significantly from scheme to scheme. Whereas in some schemes the concrete was very smooth, in others a brush finish had been carried out which increased the roughness substantially; some channels also showed moderate undulation and striation of the concrete.

(c) Sediment and debris
In most of the schemes visited small amounts of sediment and debris could be seen in the channel, but considerably more could be found at the outfalls. In one of the schemes (Reference D in Table 5) the sediment had become cemented along part of the channel. The debris often consisted of grass cuttings from the verges, twigs and rubbish thrown by passing motorists. In some cases silt and soil had deposited in the flatter sections of channel; this was particularly noticeable on newly-constructed schemes where the verges were still un-grassed. At the gratings, the debris could produce a serious blockage of the outlet, of up to about 50% of the area of the grating. Vegetation growing into the channels was also noted in most schemes, even in schemes not yet open to traffic. This suggests the need for regular maintenance of the channels, as vegetation overgrowth can produce a blocking effect in the channels which obviously affects their capacity.

(d) Concrete joints
The contraction joints observed were in generally good condition but some small cracks were found to form in their vicinity, in particular on the steeper side of asymmetric channels. In an older scheme (Reference A in Table 5) the concrete had deteriorated considerably at the joints; in another scheme (Reference B) there were signs of possible expansion of the concrete at the joints which had cracked the sealant.

(e) Geometry of channels
The cross-sectional shape of surface water channels has been specified in the Highway Construction Details (HCD) published by the DOT. In the December 1987 edition of the HCD, asymmetrical triangular channels were allowed in the verge, with maximum cross-falls of 1:5 (vertical: horizontal) on the carriageway side of the channel and 1:1 on the verge side. Symmetrical channels with equal cross-falls of 1:5 or flatter were required to be used in the central

reserve. Contemporary with the publication of DOT Advice Note HA 39/89 on Edge of Pavement Details and Amendment No 1 to HA 37/88, the December 1991 edition of the HCD no longer permitted use of asymmetrical channels in the verge, but required symmetrical channels, generally with cross-falls of 1:5 or flatter, in all locations. In exceptional cases, Amendment No 1 allows cross-falls of up to 1:4 on one or both faces of the channels.

As a result, most of the older schemes have asymmetrical channels in the verge with 1:1 and 1:5 cross-falls. This is also the case with some of the more recent schemes, the designs of which were presumably carried out earlier on the basis of the 1987 edition of the HCD and not revised in accordance with the 1991 edition when they subsequently came to be built. It was noted that outlets with gratings set horizontally on the invert of symmetrical triangular channels did not conform with the requirement that cross-falls should not be steeper than 1:4. Two examples of surface water channels are presented in Plates 1 and 2.

It was also found that a misinterpretation of the Highway Construction Details (both editions) has commonly occurred regarding the level of the outer edge of channels in the central reserve. This level has been set higher than that of the carriageway side. Exactly the opposite is recommended for safety reasons to prevent water from encroaching on the fast lane of the carriageway. This misinterpretation probably stems from the recommendation for channels set in the verge where allowance for encroaching on the hardstrip is adopted in the design to increase the channel capacity. In the recent amendment to the HCD (August 1993) changes were introduced to clarify this issue and avoid misinterpretations.

It was noted in a few instances that outlets had unnecessarily been constructed at the upstream ends of drainage lengths even though no significant amount of water would have flowed towards them. Some of the channels were terminated at the downstream end by vertical walls at almost 90° relative to the edge of the road. Better detailing was shown by some other schemes which had inclined or rounded end walls; these features would make it safer and easier for vehicles that are accidentally driven into the channels to come out of them and back on to the road.

One of the schemes visited (Reference B in Table 5) presented an unusual feature which is worthy of mention. In a symmetrical channel in the central reserve, a hump was built across the channel about 0.5m downstream of an outlet with the possible purpose of causing a ponding effect. This would increase the head over the grating and therefore the discharge capacity of the outlet. This feature, which may be hydraulically effective, presents in principle a safety hazard for motorists.

(f) Geometry of outfalls

The geometry of the outfalls varied considerably from scheme to scheme; features such as the dimensions of the outfall, the position of the gratings (in-line or off-line), the number of gratings, the local slope of the channel floor and the space allowed between gratings are normally scheme-specific.

The transition from the channel to the outlet was in a few cases quite abrupt. In one particular scheme (Reference D in Table 5) the outlet, which consisted of a single grating set back in the verge, was positioned without any sort of transition to assist turning of the flow from the channel. This solution is likely to be inefficient at collecting flow from the channel and preventing it bypassing the outlet. However, in most schemes, transitions have been built to direct the flow more smoothly from the channel towards the gratings. Transitions with slopes from 1:1 to 1:3 in plan were recorded during the visits and some of the designs had curved walls. From the hydraulic viewpoint, some of these slopes may still be too sharp to give a satisfactory performance.

In some outfalls the gratings were set at a level lower than that of the channel invert (depressed gratings) in order to increase the head over the grating; some outfalls included more than one grating and the longitudinal spacing between them was found to vary from about 1m to 2.5m. The variety of geometries observed highlighted the need for guidelines on an efficient design for outfalls.

(g) Step between channel and top layer of road

Difficulties in complying with the tolerances for the level of the channel edge in relation to the level of the road surface were very apparent in most schemes. There were visible signs of remedial work carried out to bring the levels within the specification: the bituminous wearing course had, in places, been planed down along the edge of the channels to reach the correct level. In some schemes this appeared to have been taken too far and the wearing course was lower than the lip of the channel (eg Scheme J in Table 5). In sections of road where the levels did not meet the specification along a reasonable distance, the existing channels were removed and new sections were built.

One of the schemes (Scheme A in Table 5) showed a step of up to 90mm between the black top and the channel edge along the section inspected. It appears that an overlay was placed without regard for the specification that requires the step to be smaller than 10mm.

B.4 MEETINGS AND SECOND-STAGE QUESTIONNAIRE

B.4.1 General considerations

As mentioned in Section B.1, a second-stage questionnaire was prepared (and approved by DOT) covering more general qualitative information about the use of surface water channels and the design of outfalls. It was concluded that it would not be appropriate to circulate it in the same way as the first-stage quesionnaire because the questions were less specific and would have required much more effort to answer. It was therefore decided to select a number of representative schemes on the basis of the informaiton from the first-stage questionnaire and then carry out visits to discuss the schemes with the engineer's responsible for their design, construction and maintenance. The second-stage questionnaire was used as check list during the meetings, with the answers being recorded by members of the HR/TRL project team. A copy of the questionnaire is included in Appendix II.

A number of meetings was held with designers, resident engineers and channel contractors in the period between the end of July and the end of October 1993. Two major road schemes were selected from the list of projects identified by the first-stage questionnaire: the A20 between Folkestone and Dover and the A47 Norwich Southern Bypass. All the relevant parties involved in the construction of these two schemes were contacted and meetings were arranged to gather more detailed information. Meetings were also held with the Resident Engineers of two other schemes: the A11 Red Lodge Bypass and the A19 Easingwold Bypass (still under construction). A timetable of the meetings is presented in Table 6.

Although the discussions revolved around the four specific schemes, general comments based on the experience of using this type of road drainage were also recorded. This applied particularly to the meetings with the two channel contractors (SIAC and Extrudakerb). During the first of these meetings it became evident that the list of schemes obtained through the first-stage questionnaire was incomplete. Since most channels have been slipformed it was thought useful to ask slipforming companies for lists of the schemes that they had built. The umbrella organisation BRITPAVE was contacted for this purpose and provided the names of a number of other channel contractors. These were in turn contacted and led to 49 more schemes being added to the ones identified from the first-stage questionnaire.

Although there were differences of opinion amongst the engineers regarding the advantages and disadvantages of surface water channels and the difficulties (or not) of their design and construction, some conclusions could be drawn. These conclusions, which are believed to be representative of the general view on each matter, were grouped in the categories described in the following sections.

B.4.2 Hydraulic design of channels

(a) Choice of surface water channels:

Other drainage options such as kerbs and gullies were considered in the early stages of design by most of the designers contacted. In one scheme (Reference E in Table 5) the Department of Transport specifically asked Suffolk County Council to compare traditional drainage with surface water channels. Surface water channels were chosen in some schemes because they were considered to be DOT's preferred option. The general view is that, where channels can discharge into soakaways or directly into watercourses they are a good alternative to traditional kerbs and gullies. Kerbs and gullies may prove more economical if the number of discharge points is limited and flow from the channel outlets needs to be conveyed to them by long lengths of carrier pipe. If the NRA imposes restrictions on the discharge of road runoff into watercourses, filter drains may be more suitable since pollutants will travel more slowly and some of them will be retained in the drain. However, expensive remedial work may later be necessary to remove the pollutants from the filter material and the associated collecting pipes.

(b) HA37/88 design method

The method described in HA 37/88 was seen by one designer as too complex, but another had produced tables for use in worksheets which simplified the design. The designers of Scheme E (see Table 5) decided upon two different geometries (one for the verge and the other for the central reserve) and checked their capacities using HA 37/88. In this way they reduced the number of variables involved. The designers of Scheme B developed their own method which followed the Wallingford Procedure and Road Note 35 for the design of the channels. The design of Scheme G was based on previous experience in Hong Kong where the consultants had designed channels with similar characteristics.

The main criticism of the HA 37/88 method was that it did not include any guidance for the outlet design. Also, with the present need for carrier pipes to convey the flow from the outlets to a suitable site, the designer is faced with two different systems of design: one for the channels and another for the pipes. A single package would be welcomed by most designers. It should be noted that, with computer spreadsheets being widely used, graphical methods are not generally favoured.

B.4.3 Pavement drainage

Rodding eyes for inspection purposes are commonly used with fin drains. The need for sub-surface drainage was questioned by one respondent for highly permeable soils. However, the DOT always requires the inclusion of pavement drainage. It may be noted that the first-stage questionnaire showed that a scheme was built in Northern Ireland, without sub-surface drainage because the road was constructed on a free-draining gravel embankment; the design standard for the scheme was a DOE (NI) Roads Service document. In Scheme B the fin drain did not connect to carrier pipes serving the surface drainage but drained directly into catchpits.

B.4.4 Hydraulic design of outfalls

At present there are no guidelines for the design of the outfalls. Therefore designers use their engineering judgement bearing in mind that the outfalls may easily be blocked by debris. This may lead to a conscious decision to overdesign in terms of the size or number of gratings required.

If a system needs to incorporate carrier pipes, as seems to be the case in many schemes, there are no guidelines for how they should be designed; uncertainties arise concerning the time of entry into the drains due to the considerable length of the channels.

Some outfalls have been designed with the grating(s) set back in the verge and/or with a depressed sole to increase the head over the grating (eg Schemes E and G in Table 5). In some cases two or three gratings were installed so that, if blockage of one grating were to happen, there would still be sufficient capacity available to drain the flow. Some bypass flow (25%) has been allowed in the design of the outfalls of Schemes E and G.

B.4.5 Constructional aspects

<u>(a) Channels:</u>

General aspects

Resident engineers of the schemes visited expressed contradictory views regarding the constructional aspects of surface water channels. Some feel that they hinder the work programme because they add another major stage to the construction process. Channels involve specialised contractors who are separate from the pavement drainage contractors; this can create additional difficulties in the managing of the project. On the other hand, the channels were said to "tidy" up the edges of the road: because they are normally built on the road sub-base, the channels form a longitudinal edge against which the road layers can be neatly built.

The amount of concrete used to form the channels can be quite considerable since the channels are generally founded on the road sub-base. The overall depth of the channels could be reduced if structural tests showed that their strength would still be satisfactory. A concrete thickness below the invert of approximately 100mm has been suggested as suitable from the channel contractors viewpoint. Assuming a 150mm depth channel profile it would result in an overall concrete depth of about 250mm. At present it is common to find overall depths (measured from the carriageway edge) of the order of 450mm. However, channel construction represents only part of the total construction costs. If thinner channels are laid on a bound pavement material, little economy would be achieved. Construction on a locally thicker layer of unbound sub-base would result in savings in material costs but would complicate pavement construction.

Shape

From the constructional viewpoint the shape of the channels does not appear to be a critical factor. Very wide channels (2m or more) can be built with no special difficulties by the major slipforming contractors. Flatter cross-falls (as opposed to 1:1 cross-falls), rounded edges and sloping outer sides (which are now recommended by DOT) are generally preferred.

It was noted during the site visits that in some channels the sides are locally convex in cross-section and so do not have the required triangular shape. This probably results from differential slump of the concrete. The bowing out of the sides will tend to reduce the capacity of the channel.

Constructional tolerances

Matching the tolerances on level for the carriageway edge of the channels and for the bituminous wearing course appears to have been difficult in many schemes. The slipform contractors normally aim to construct the channel so that the top edge on the carriageway side is 5mm below the level specified for the finished road surface. Allowing for a working tolerance of ±5mm (which experienced contractors consider to be reasonable) should result in a final concrete level of 0 to -10mm relative to the specified level; this variation is equal to the HCD limit on the allowable size of step at the edge of the channel. However, this does not take account of the permitted tolerance on the level of the bituminous wearing course which is 0 to +6mm relative to the specified level. In combination, the separate application of the two sets of tolerances could result in a maximum downward step of 16mm from the pavement to the adjacent edge of the surface water channel, which is beyond the permitted limit of 10mm. From their viewpoint, the slipform contractors feel that they are sometimes unfairly penalised for errors in level that were not of their own making. The level pins used to set out a surface water channel have often been lost or replaced by the time the pavement is laid so it can then be difficult to prove whether or not the channel was constructed to specification.

Remedial work can be carried out to bring the step between the channel and the road surface within specification. Solutions include: planing of the bituminous material forming the hardstrip, grinding the concrete in the channels and introduction of epoxy-concrete patches to bring the channel to a higher level locally. This latter approach was used in parts of Scheme B (see Table 5) but does not seem satisfactory because the joints between the epoxy patch and the underlying concrete remain weak points which could cause premature deterioration of the channels. A better solution, which has been adopted in some other cases, is to remove sections of the channel and construct them again to the right level. Grinding of the channels is also not considered advisable over longer stretches as it exposes the aggregates, increases the roughness of the channel and may lead to frost damage.

Tolerances should not only apply to the edge of the channel but also to the overall shape of the channel. It is possible for the concrete in the channel to show different slump levels depending on whether the concrete is on the edge, or say, the invert of the channel. Since the capacity of the channel is a function of its depth, it would be advisable to check that the invert level is correctly set.

Overlays
It is often necessary to rehabilitate flexible roads as they wear and their surfaces deform, crack and loose their texture. This can be readily performed with roads with surface water channels by planing off the old pavement surface and replacing it by a new pavement surface whilst at the same time maintaining the surface level of the pavement and limiting the step between the pavement and the edge of the channel to the specified limit of 10mm.

Flexible pavements, however, often require strengthening during their life either due to wear or to increase in the amount of traffic that can be carried in their design life. This can involve pavement overlays with thicknesses between 40mm and 300mm. The surface water channels would also require to be built up to this thickness to avoid a large drop into the channel. Overlay techniques should be developed prior to the need to provide structural overlays to roads that have been built with surface water channels.

Concrete mix
The workability of the concrete used in surface water channels is a major constructional factor. From experience the slipforming contractors have come up with their own specifications for the concrete which maintain workability with a lower than normal slump while producing the cube-strengths required by the HCD. It appears that concrete grade C35 with a 6% air content has been generally adopted by the contractors with success. Some Resident Engineers have argued that certain mixes did not conform to the DOT requirements, but this problem has since been resolved by changes adopted in the August 1993 amendment to the HCD and Specification for Highway Works (SHW); this question is discussed again in Section B.5.

The contractors consider they need to be able to add some water on site to improve workability when the concrete mixture is too dry, and some Resident Engineers permit this to be done. The contractors argue that if the amount of water added were sufficient to reduce the concrete strength below specification, the mix would be too wet to be used successfully for slipforming. The counter argument is that the unquantified addition of water can lead to inconsistent batches of concrete and an increased risk of sub-standard sections of channel. There is some confusion over whether there is an applicable DOT specification for the workability of the concrete. Clause 1005 of the Specification for Highway Works (1991) states that the optimum workability required "shall be determined by the Contractor and approved by the Engineer". The slipform contractors consider that slumps in the range 15mm to 35m are generally satisfactory.

The current sampling of the concrete ever 30 linear metres (which is suitable for pavements) may be excessive for surface water channels. The criterion should preferably take account of volume of concrete laid as well as the length. The distance between samples should be chosen so that unsatisfactory sections of channel are identified and satisfactory concrete is not removed unnecessarily.

Joints

There appears to be some confusion regarding the depth at which the joints should be formed. The recommended depth is 25mm below invert level but it is not clear whether, on the sides of the channel, the joint should be cut 25mm below the surface or to a horizontal level corresponding to 25mm below the invert. Clarification on this point by the DOT seems necessary.

Although there is no universal agreement on the best joint forming method (wet or dry), it appears that in most schemes the joints are wet formed. It was observed, however, that local distortion of the concrete may occur with this method. From the hydraulic viewpoint this may result in localized accumulation of sediment or debris and reduced capacity of the channels.

(b) Outfalls

The outfalls are normally moulded by hand. In some schemes, it appears that the slipforming machine was stopped just before the outlet position, moved forward a few metres and started again on the other side, the outlet was later formed in situ in the gap left between the two sections of channel. This method has the disadvantage that the machine tends to settle slightly when it is stopped so that the channel has a low point either side of the outlet. An alternative method (used, for example, in Scheme L in Table 5) is not to interrupt the slip-forming process but remove whole sections of channel at the outlet positions while the concrete is still green. This produces a better longitudinal profile but can waste a significant quantity of concrete when outlets are closely spaced. From the slipforming point of view it is better to design the outfall so that the gratings are positioned with their carriageway edges on the invert of the channel or set back into the verge. With this layout the channel can be slipformed continuously and only the verge side of the channel needs to be cut out to build the outfall.

(c) Sub-surface drainage

In schemes where fin drains are used for pavement drainage, difficulties have been found in bending the geotextile in the under channel drainage layer to a 90° angle around the verge edge of the channel, as indicated in the HCD. A normal procedure is to cut and overlap the geotextile at this position.

Sub-surface drainage would be improved if geotextiles used under surface water channels were to extend about 150mm horizontally towards the road side and not just 50mm, as is currently recommended. This recommendation stems from the fact that membranes can be shifted quite easily by the passage of a slipforming machine.

(d) Alternative shapes of channel

From the constructional (slipforming) viewpoint there seem to be no reasonable limitations to the slopes of channel that can be formed. Channels of trapezoidal cross-section, or novel designs such as slotted channels or channels incorporating carrier pipes, can be built without major adjustments to the slipforming machines already in operation.

B.4.6 Maintenance

Surface water channels require frequent maintenance if their discharge capacity is to be safeguarded. Particular attention should be taken when cutting the grass along the verges of the channels, as grass cuttings can easily block the gratings at the outfalls or obstruct the channels causing local flooding upstream.

The frequency of cleaning of the channels adopted in Scheme G (see Table 5) appears to be satisfactory: regular cleaning is done on a monthly basis, but maintenance is carried out ad hoc when there are visible signs of debris accumulating in the channels.

B.4.7 Safety

Safety questions were raised during the meetings but no incident was reported where a surface water channel had been considered to have caused an accident or made one worse. Signs of an accident were observed at one site (Reference J in Table 5). Although the vehicle had crossed the central reserve channel at an angle of about 45°, the adjacent safety barrier appeared to have successfully absorbed the energy of the vehicle by means of plastic deformation. The presence of the channel did not therefore appear to have altered the level or pitch angle of the vehicle beyond the limits within which the safety barrier was able to perform as intended.

B.5 DISCUSSION

The purpose of this Section is to discuss the implications of some of the information obtained from the second-stage questionnaire (see Section B.4) and to include additional comments on the use of surface water channels based on the experience of the HR/TRL project team

(a) Size and shape of channels

Current usage of surface water channels is strongly constrained by safety considerations which limit the allowable depth to a maximum of 150mm and cross-falls to values no steeper than 1:5 (or 1:4 in exceptional cases). Constructional factors and the limitation on maximum depth make it difficult to set the channel at a different longitudinal gradient from that of the road, and this can result in relatively short spacings being required between outlets on level or nearly level roads. Also, the shallow triangular channels used are not very efficient from the hydraulic point-of-view because the ratio of wetted perimeter to cross-sectional area is relatively large. Many of these limitations are unavoidable as long as the channels need to be contiguous with the road surface and there is a possibility of vehicles driving into them. Despite the limitations, the existing types of surface water channel still allow considerably larger spacings between outlets than equivalent kerb-and-gully designs, as well as offering some of the other practical advantages discussed in Section A.1. However, the relative lack of capacity of the channels has made it necessary for parallel carrier pipes to be used in most schemes to collect flow from the outlets and convey it to suitable discharge points. The extra cost of the carrier pipes will have removed some of the economic benefits of using surface water channels. Also it has not always been possible to keep the pavement drainage separate from the surface water drainage, as surface water and sub-surface water have sometimes been collected in the same carrier drain.

Much greater flexibility in design becomes possible in situations where safety barriers are necessary and vehicles are thereby prevented from entering the channels. The longitudinal gradient of the channel need no longer be tied so closely to that of the road, and deeper, narrower channel shapes with better hydraulic performance can be used. As a result, it would be relatively easy to achieve much greater distances between outlets than are possible with the current types of triangular channel. Consideration should therefore be given to developing standard construction details for channels that are protected by safety barriers so that designers are made aware of the wider range of options that then become available.

Some developments are also possible in the geometry of the existing type of surface water channel. The limitations on maximum depth and cross-fall do not prevent the use of trapezoidal cross-sections, and these will usually give a higher flow capacity than equivalent triangular profiles. Sediment may deposit on the flat invert of a trapezoidal channel but such deposition is also observed to occur in triangular channels when flow velocities are low. The width of deposition in a trapezoidal channel may be reduced if the invert is not truly horizontal but given a small transverse cross-fall (eg 1/40 as in conventional kerb-and-gully designs).

Another method of increasing flow capacity could be to form a carrier pipe within the mass concrete below the invert of the surface water channel. The slipform contractors say that this can be done using an inflatable plastic lining that is held in position as the concrete is poured; it is understood that this technique has already been used on a road in Scotland. One possibility would be to let the water from the surface channel drain directly into the internal carrier pipe by means of regularly-spaced slots. However, maintenance problems might be caused by sediment blocking the slots or depositing in the pipe. An alternative possibility would be to discharge water from the surface channel into the carrier pipe only at the normal outlet locations. Possible drawbacks to the use of an internal carrier pipe include: the limited size and capacity of pipe that can be fitted within the channel block; the effects of possible leakage due to cracks and construction joints; and the inability to vary the longitudinal gradient of the pipe from that of the surface channel. Further study is therefore needed to determine the feasibility of this possible development.

(b) Hydraulic design method

Experience in the use of HA 37/88 has shown that many lengths of surface water channel have design storm durations of less than 8 minutes, which is the recommended lower limit for application of the design curves. Short durations occur in the design procedure if the outlet need to be closely spaced due to high rates of run-off from wide carriageways and/or insufficient flow capacity in the surface water channels. The principal reason for the 8 minute limit was that the rainfall equation built into the design curves was optimised for durations of about 15-40 minutes and significantly overestimates rainfall intensities in short storms. HA 37/88 therefore recommends use of an alternative design procedure in Contractor Report 2 (1984), which deals with the spacing of British Standard gully gratings. However, in retrospect, it might have been better to allow use of HA 37/88 for shorter durations and accept an element of over-design. This is because Contractor Report 2 also contains approximations and uses a fixed storm duration of 5 minutes irrespective of channel length. It is considered that the best solution would be to revise the design curves in HA 37/88 so that the they are optimised for storm durations of about 2-20 minutes and can then be used for all channel designs without restriction.

A linked problem concerns the time taken for rain falling on a carriageway to reach the adjacent surface water channel. This time is typically 1-2 minutes and is not taken into account by the design procedure in HA 37/88. It was originally expected that design storm durations would be much longer than two minutes so that any errors would be negligible. However, the time of entry to the channel will need to be included if the method in HA 37/88 is revised to allow durations of less than 8 minutes.

(c) Constructional aspects

The information on tolerances obtained during the site meetings (see Section B.4) indicates that the current specification of 0-10mm for the downward step between the pavement surface and the edge of the concrete channel can be difficult to achieve. Part of the problem is that the actual step height is the result of two separate operations (the slipforming of the channel and the laying of the pavement) which are usually carried out by different contractors and at different stages in the project. The performance of the slipforming machines has been improved since the first schemes were constructed and a tolerance of ±5mm relative to the specified level is considered quite practicable. Often the slipforming contractors will aim to form the channel edge 5mm below the pavement surface to reduce the possibility of the channel being built higher than the pavement. When combined with the allowable variation of 0 to +6mm in the pavement surface a step height of 16mm, instead of the limit of 10mm can be produced. If the pavement tolerance is not altered, the slipform contractors would have to achieve an accuracy of ±2mm to satisfy the specification, and this is probably not achievable with present equipment and methods. One possibility would be for the pavement contractor to use the edge of the channel as the level reference so that a step height of more than 6mm should not occur. This might require some changes to the levelling system of the pavement-laying machine and could result in a slightly undulating road surface; an effect that may be considered more undesirable than an out of tolerance step height at the pavement edge. Other possibilities include allowing a somewhat greater maximum step height, or making minor and infrequent adjustments to required pavement levels (on the basis of a detailed survey of the channel) so that any discrepancies are minimised. Some of the remedial measures that have been adopted when tolerances were exceeded appear to have created more problems than they solved. Some latitude is needed so that potentially harmful grinding or patching does not automatically have to be carried out to correct small localised errors.

Some of the comments received on concrete mixes referred to the 1991 editions of the Specification for Highway Works (SHW) and the Highway Construction Details (HCD). Amended editions in 1993 dealt with some of these points and now permit the use of concrete C35 in slipformed channels. In BS 5931, 1980 "Machine laid in-situ edge details for paved areas" it is stated that the optimimum workability for the mix to suit the paving plant being used shall be determined by the Contractor and approved by the Engineer. Slipform contractors consider Grade C35 concrete with 6% air content to be satisfactory for the purpose. It can be deduced from BS 5328 Part 1 that concrete with a 16mm maximum aggregate size could be used with a 6% air content.

Surface water channels normally have either a brushed finish or a float finish. The brushed finish used in some channels seems excessively rough, both visually and from a hydraulic viewpoint, and could reduce the flow capacity. However, the float finish may offer a somewhat lower skidding resistance if vehicles or cyclists enter the channel.

(d) Maintenance

Surface water channels need regular cleaning because sediment and debris washed from fairly large areas of road tend to become concentrated at low points and outlets. Little cleaning appeared to have been carried out on some schemes that were visited. Blocked gratings can cause more serious problems in surface channels than in kerb-and-gully systems where the outlets are more closely spaced and flows are smaller.

It is not known whether surface water channels in the central reserve are proving difficult to clean. Use of cleaning vehicles in the fast-lane of a dual carriageway would seem to present safety problems, but the site inspections did not suggest that the level of maintenance was any worse than for channels in the verge. The adoption of symmetrical 1:5 triangular channels in most of the new schemes suggest that the development of a lorry-mounted cleaning system using appropriately shaped brushes would be justified.

B.6 CONCLUSIONS

(1) The general questionnaire had a very positive response which led to the identification of 39 schemes where surface water channels have been (or are going to be) used as the major surface drainage system. The regions which appear to have a higher density of this type of road drainage are the East and South East of England, and Wales. Approximately 50% of the schemes had not been completed at the time of the survey (between December 1992 and July 1993), which appears to indicate a rapid adoption of surface water channels in recent years.

(2) Most channels identified were triangular in cross-section with cross-falls which varied considerably from scheme to scheme. Some channels showed very steep slopes on the carriageway side: the steepest cross-falls found in the survey were 1:2.3 in channels positioned both in the verge and in the central reserve. The dominant design flow depth of the channels was found to be in the range of 100mm to 150mm.

(3) The replies to the general questionnaire highlighted some important points regarding the channel outlets. The first one is the positioning of the gratings in relation to the channel invert: in two thirds of the schemes the gratings were set on the invert of the channel. Many of the outlets in the invert of symmetrical triangular channels use cross-falls that are locally steeper than the allowable limit of 1:4. The second was that in the majority of schemes (65% approximately) carrier drains had to be used, in some cases in conjunction with other systems, to convey the flow from the outlets. Only two schemes mentioned the exclusive use of soakaways and toe ditches for the direct discharge of the flow. The need for carrier drains imposes an extra cost on the construction of a scheme; it also tends to weaken the case for using surface water channels in road schemes since the carrier drains represent, in a sense, a duplication of the drainage system.

(4) Outlets in surface water channels have been positioned with spacings as great as 1100m but minimum spacings of the order of 10m were also reported.

(5) Visits to several road schemes showed that a misinterpretation of the HCD has commonly occurred regarding the level of the outer edge of channels in the central reserve. It is suggested that a stronger recommendation is included in the HCD so that designers do not make the error of adopting a higher level for the verge side of the channel than for the carriageway side which leads to water encroaching on the fast lane of the road.

(6) It is suggested that standard construction details are developed for channels protected by safety barriers so that designers are made aware of the wider range of options that then become available.

(7) Consideration should be given to increasing the flow capacity of channels by adopting trapezoidal cross-sections or, for example, by forming a carrier pipe within the mass concrete below the invert of the surface water channel.

(8) It is recommended that rodding eyes are installed for inspection and cleaning of fin drains. Sub-surface drainage should be kept separate from the surface water drainage in order to retain one of the advantages of surface water channels.

(9) Constructional tolerances on level for the carriageway edge of the channels and for the bituminous wearing course need to be reviewed so that the required overall tolerance can be achieved. Tolerances should also be specified for the level of the channel invert.

(10) Suitable techniques for adding overlays to roads with surface water channels need to be developed at this stage in time before structural overlays are required in the existing schemes.

(11) Some earlier discrepancies regarding the grade of concrete were resolved in the 1993 editions of SHW and HCD. The question of the workability of the concrete mix has generated some confusion amongst the different parties involved in the construction of surface water channels. However, the SHW states that the channels should comply with the Code of Practice BS 5931, 1980 "Machine laid in-situ edge details for paved areas" which recommends that the slump of concrete for slipforming should be within the range 25-65mm ± 10mm. Within this range, the value of slump that is acceptable should be first agreed between the slipforming contractor and the concrete supplier. Provided the concrete workability is within the recommended range, the acceptability of the channel should be judged on the basis of the actual tolerances on level achieved and concrete strength.

(12) The depth at which joints are formed in surface water channels needs clarification. The present DOT recommendation is not clear about whether the recommended depth of 25mm corresponds to a horizontal level below the invert or below the surface.

(13) The construction of the sub-surface drainage would be improved if the geotextiles under the surface water channels extended to about 150mm horizontally towards the road side instead of only 50mm, as currently recommended.

PART C EXPERIMENTAL STUDY OF OUTFALLS

C.1 INTRODUCTION

A detailed experimental study was necessary to complement the information gathered from the questionnaires and enable the development of suitable methods for the design of channel outfalls. This involved both field measurements and laboratory tests, but the methods for the hydraulic design of outfalls were mainly developed from the laboratory tests because these allowed a more systematic way of varying the geometric features of the outfalls and the flow conditions.

In this study the outfall is defined as the drainage system that collects and removes water from the surface water channels and conveys it to a downstream point of discharge. The outfall is formed by the outlet (which collects the flow and removes it from the surface) and the outfall structures (which consist of the chamber below the outlet and of the arrangements for conveying the water to a collector pipe, a soakaway or a watercourse).

As mentioned before (see Section A.1), an in-line outlet is defined as an outlet where the water is essentially collected symmetrically either side of the channel invert; in an off-line outlet the channel is widened away from the carriageway and the outlet is off-set from the centreline of the channel.

C.2 FIELD TESTS

C.2.1 Objectives

The purpose of the field tests was to measure the performance of existing outlets in two selected road schemes. The performance of an outlet can be assessed both qualitatively (by visual observation) and quantitatively (by measurements of the approach flow and the bypass flow).

Features of surface water channels such as shape and surface texture, as well as contraction and expansion joints, affect the total roughness of the channels and the velocities approaching the outlet. Measurements allow an estimate of the Manning's roughness value for the channel "as built". Visual observation of the flow patterns at the gratings of the outlets can help identify causes of possibly inadequate capacity of an outfall. Improvements in the performance can then be obtained by changes to the outfall layout.

C.2.2 Description of the sites

The choice of the two sites for field tests was based on the following criteria:

1 The road scheme should be completed or nearly completed but not yet open to traffic to avoid traffic disruption caused by the need to cone off part of the road;

2 The scheme should preferably have a wide range of slopes so that different flow conditions can be produced in the channels;

3 The channels in the two road schemes should preferably have different geometries (ie. different surface width and side-slopes); the outfalls

should have different layouts and different positions in relation to the channel invert (ie. in-line and off-line outlets).

The two sites chosen for the tests were selected from the road schemes visited earlier in this study. They were the A20 Folkestone to Dover - Contract 1, in Kent, and the A487 Port Dinorwic Bypass, in Gwynedd, Wales. The tests in the first scheme were carried out on 26 and 27 October 1993, and in the second scheme on 29 and 30 November 1993. Both schemes were due to open soon after the field tests were carried out.

C.2.2.1 A20 Folkestone to Dover (Contract 1)

The A20 Folkestone to Dover - Contract 1 has surface water channels both in the verges and in the central reserve but of different characteristics. The channels in the verge are asymmetrical with side-slopes of 1:1 on the verge side and 1:5 on the carriageway side. The design depth of the channels is 150mm and the corresponding surface width is 900mm. The channels in the central reserve are symmetrical with side-slopes of 1:5, a design depth of 120mm and a surface width of 1.2m.

The outlets are formed by a triple grating set on the invert of the channel. These gratings, Brickhouse Dudley, Triple Waterway 450mm x1350mm, were set horizontally on the channel invert. A smooth transition was formed between the channel invert and the outlets. The same geometry was used for the terminal and intermediate outlets.

In most of the scheme the channel outlets are set in relatively mild longitudinal slopes which vary between 1:2000 and 1:400. It was initially thought that the water necessary for the tests might be obtained from a nearby watercourse. However, there were no streams in the vicinity of the scheme from where water could be easily pumped to supply the flow for the tests. For this reason it was necessary to use a road tanker which had to return to a depot in Folkestone to refill before each test.

C.2.2.2 A487 Port Dinorwic Bypass

In the A487 Port Dinorwic Bypass, surface water channels are only present in the verges (the scheme has only one carriageway in each direction and no central reserve). The channels are symmetrical with side-slopes of 1:5, a design depth of 78mm and a surface width of 780mm.

The outlets are formed by double triangular gratings, Glynwed Niagara 200 type, 650mmx650mm in size. In the terminal outlets two gratings are used with a horizontal distance of 1570mm between them. The terminal outlets end with a kerb at right angles to the carriageway. Intermediate outlets consist of a single grating. In both types of outlets the gratings are set back into the verge with the side of the grating closest to the carriageway coinciding with the channel invert.

This scheme includes some very steep longitudinal slopes of the order of 1:16.7 (or 6%) in places, which is the normal maximum value permitted for high-speed roads. It was found most convenient to use a tractor-drawn tanker for the tests and to re-fill it from a nearby watercourse.

C.2.3 Measurements

C.2.3.1 Equipment and test procedure

The equipment used in the tests consisted of three point gauges to measure the water level, and current meters to measure the flow velocity in the channel and in the vicinity of the outlets. Two types of current meter were used: two miniature current meters which can measure flow velocities up to 1m/s, and two miniature impeller meters (Valeport BFM004) designed to measure flow velocities up to 2m/s. The two types of current meter were connected to counter units which gave readings of the rate of rotation of the propellers. These readings were later converted into values of flow velocity by using the calibration curves of the instruments.

The procedure adopted in the field tests was first to identify suitable outlets to test. As mentioned before, it was considered important to cover a wide range of channel and outlet geometries, slopes and flow conditions. Once the outlets were selected, a careful survey was carried out of the channel upstream and downstream of the outlets. The survey of the invert level extended from about 30m upstream of the outlets to 4m downstream of intermediate outlets. In terminal outlets the survey was carried out to the downstream end of the outlet. The longitudinal slope of the channel was thereby determined. Detailed surveys were carried out at particular sections where the measurements were to be taken with the point gauges and the current meters. This was used to determine the channel cross-section as built, in particular the side-slopes and depth of the channels. The location of these sections relative to the outlets varied from test to test and will be described later.

One tanker load was required to carry out each test. After some trial runs the tankers proved to be adequate in supplying water to the channels. However, the tankers could not supply enough water to fill the channels in the steeper slopes of the Port Dinorwic scheme.

The water was introduced into the channel about 40m upstream of the outfalls by means of flexible hoses. The hose used at Folkestone was nominally 75mm in diameter, whereas two different sizes of hose, of 25 and 100mm nominal diameter, were used at Port Dinorwic. In this scheme it was possible to vary the flow rate by either pumping the water or allowing it to flow by gravity. Readings of water levels and flow velocities in the channel were taken when the levels were considered stable after the initial surge caused by the introduction of the water by the tanker hose. The tests in the two sites were carried out with various water depths in the channels ranging from almost channel-full to about one third full.

In each cross-section the velocity measurements were taken at approximately mid-depth at transverse spacings of 50 or 100mm. Readings at mid-depth are close to the mean velocity which occurs at approximately 0.6 of the water depth measured from the surface.

C.2.3.2 Tests at A20 Folkestone to Dover (Contract 1)

The tests were carried out at three outfalls: two in the central reserve (Plates 3 and 4) and one in the verge (Plate 5). All of these were terminal outfalls. Point gauges were installed on the channel centreline at 2m and 0.2m upstream of the front edge of the gratings, and at 0.1m downstream of the end of the gratings. A detailed survey of these sections had been carried out previous to the measurements to determine the channel geometry accurately. Readings of flow velocity were also taken at these sections. The flow was pumped from the tanker by a 75mm diameter hose.

No bypass flow of any significance was recorded during the tests, ie all the flow was collected by the gratings. In most tests the water was all intercepted by the first and second sections of the triple grating; only in one test of the outfall in the verge was the flow also collected by the last section of the triple grating.

C.2.3.3 Tests at A487 Port Dinorwic Bypass
The tests were carried out at three outfalls: one terminal on a steep slope (Plates 6 and 7), one intermediate on a steep slope (Plate 8) and one intermediate on a milder slope (Plate 9). The measuring sections for the terminal outfall were located 3m upstream of the front edge of the first grating and at mid-distance between the two gratings. The measuring sections for the intermediate outfalls were positioned 3m upstream and 4.5m downstream of the front edge of the grating. A detailed survey of these sections had been carried out previous to the tests. Readings of velocities were taken at these sections.

Flow bypassing the gratings occurred only during two tests of the intermediate outfall set in the steeper slope of 6%. In all the other tests the outfalls had enough capacity to collect the flow introduced into the channel.

C.2.4 Analysis of results
The longitudinal slopes of the channels and the side-slopes at the chosen sections were determined from the survey readings. The survey showed that one of the channels in the central reserve of the Folkestone scheme had an irregular longitudinal slope. The outlets tested in the central reserve were terminal outlets, ie each was located at the end of a channel.These two channels run in the West-East direction; for identification purposes, they were called the Central Reserve West channel (CRW) and the Central Reserve East channel (CRE), respectively. Close to the outlet the CRW channel exhibited a series of alternating positive and negative gradients but the overall slope was practically flat. The irregularity of the channel slope made it impossible to relate the flow measurements to the Manning resistance equation, which assumes flow conditions to be uniform. Therefore, no reliable roughness value could be obtained for this channel. It was also found that the side-slopes of the channels were slightly flatter than the nominal ones; the surveyed side-slopes were used in the determination of the flow cross-sectional area.

In sections where the water depth was sufficiently high to allow several readings of the flow velocity, the mid-section method was applied to calculate the flow discharge. In this method the channel cross-section is subdivided into several vertical sections; the mean velocity and depth at a subdivision point are multiplied by the section width measured between the mid points of neighbouring sections. In tests where only a single value of velocity was possible to record, this was done at the centreline of the channel, at about mid-depth. A correction was necessary in these cases in order to obtain a value representative of the mean flow velocity of the whole cross-section. From published data on velocity patterns in triangular channels, it was found that the measured values should be multiplied by a factor approximately equal to 0.8. This was carried out before proceeding with the analysis of the field data.

During the tests some difficulties were encountered in measuring the flow velocity with the Valeport impeller meters. These were therefore replaced by the miniature current meters in tests where the flow velocity did not significantly exceed the range of the current meters (1m/s). In tests with higher flow velocities, which occurred mainly in the steeper channels of the A487 Port Dinorwic Bypass, the flow discharge had to be estimated by an alternative means. The procedure adopted was to calculate the Manning's roughness value corresponding to the tests where a high number of velocity readings was available, ie the tests carried out on the milder slope. The Manning's roughness value, n, can be obtained as follows:

$$n = \frac{A\ R^{2/3} S^{1/2}}{Q} \qquad (1)$$

where A is the flow area, R is the hydraulic radius, S is the longitudinal gradient of the channel and Q is flow in the channel. The hydraulic radius is defined as the ratio of the flow area and the wetted perimeter, ie the perimeter of the channel in contact with the water flow.

The average value obtained, n=0.009, was then used to estimate the velocity and flow discharge in the steeper channels.

The results of the tests are shown in Tables 7, 8 and 9. In the tables, h is the water depth and V is the mean flow velocity at the measuring sections which were a few metres upstream of the outlets; Q is the flow rate in the channel approaching the outlet, and h_p is the water depth of the flow Q_p that bypasses the outlet. In Table 9, η represents the efficiency of the outlet which was calculated as the ratio (Q-Qp)/Q.

As mentioned before, the values of Q were determined with the flow area based on the side-slopes obtained in the survey of the channels' cross-sections. The values of velocity between brackets in Table 8 were obtained by dividing the estimated flows by the measured flow area.

As can be seen from Tables 7 and 8, the values of Manning's n varied between 0.007 and 0.011 for the tests where the measurements of the flow velocity were more reliable. This occurred both at Folkestone and in the milder slope at Port Dinorwic. In the steeper slopes of Port Dinorwic considerably higher values of Manning's n were obtained. It should also be noted that the value of velocity measured in Test no.7 appears to be unrealistically small and therefore gives an extremely high value of Manning's n.

Further analysis was carried out on the results of the field tests following the laboratory tests and the preparation of the Advice Note on Outfalls (see Section C.4.6).

C.2.5 Conclusions

The following conclusions can be drawn from the field tests:

1. Roughness values were determined for the surface water channels in the two sites selected for field tests. The tests carried out in the first site, the A20 Folkestone to Dover (Contract 1) showed values of Manning's n of

0.007 to 0.008 whereas the second site, the A487 Port Dinorwic Bypass, indicated values of n of about 0.009.

In general, the roughness values of these channels appear to be smaller than the value of n=0.013 recommended in HA37/88 for surface water channels in an average condition. Assuming that the design was based on n=0.013, this may imply that the channels tested can actually convey somewhat bigger flows than the design flow. It should be noted, however, that the channels tested were relatively free of sediment, and that the effect of runoff entering along the length of the channel was not reproduced. These two factors tend to increase the channel roughness value and therefore have to be taken into account in the design of surface water channels.

2. Although the tests were not carried out at the full channel capacity, they showed that the outlets used in the milder slopes are adequate for channel-full conditions on these slopes. In the A487 Port Dinorwic Bypass, excessive bypass flow was registered in an intermediate outlet set on a slope of 6%. With channel-full conditions it is likely also that the terminal outfalls would not be able to cope with the very high velocities generated by slopes of this order of magnitude. An increase in the length of the outlet, and possibly the introduction of additional gratings, would be required for an adequate design.

C.3 LABORATORY TESTS

C.3.1 Introduction

The purpose of the laboratory tests was to develop suitable designs of outlet for surface water channels and to obtain in a systematic way quantitative information on their hydraulic performance for use in a new Advice Note. As explained in Section A.2, the Contract allowed for testing two different channel geometries and three outfall designs for each of the channel geometries.

The laboratory testing of the outlets was carried out following the survey of existing channels and the field tests described in Part B and Section C.2. These earlier stages of the study helped identify the sizes and slopes of channel that are representative of existing schemes and also highlighted the need to develop more efficient designs of outlet. When selecting the channel shapes to be tested, account was also taken of conclusions arising from a concurrent study of safety aspects that was being carried out by Mr B Robinson at the Transport Research Laboratory (TRL). Following discussions with DOT and TRL, it was decided to test outfalls in symmetrical triangular channels with cross-falls of 1:5 and in a higher capacity channel of trapezoidal cross-section, with a design depth of 0.150m and cross-falls of 1:4.5. The first channel shape corresponds to the standard profile recommended in the current Highway Construction Details and HA 39/89. The second shape is being considered by DOT as a means of providing higher flow capacity for wider or flat roads while not exceeding existing limitations on depth and cross-fall. The capacity of this trapezoidal channel is 45% higher than that of a 0.150m deep trangular channel with cross-falls of 1:5. The sole width of the trapezoidal channel was chosen so that the cross-falls would not exceed 1:4 at the outlet if gratings of 450mm x 450mm were to be installed on the sole of the channel. This meant that the sole width would be equal to 0.300m and therefore the design surface width would correspond to 1.65m. As will be explained later, DOT is also considering an alternative trapezoidal shape having the same values of depth and sole width but with cross-falls of 1:5 and a design surface width of 1.8m. Only the first trapezoidal shape (1:4.5 cross-falls and 1.65m surface width) was studied experimentally, but the results were used to make approximate estimates of outlet performance for the alternative trapezoidal shape.

Two types of outlet were studied, defined according to their position along the channel as 'intermediate' and 'terminal' outlets. Intermediate outlets are located at points part-way along a length of channel where the flow rate of water from the road reaches the carrying capacity of the channel. Terminal outlets are located at low points along a length of channel and need to be able to collect practically all the flow carried by the channel.

The design of outlets for surface water channels must be based on the flow conditions at the approach to the outlet, namely the velocity, depth and direction of the flow, since these are the fundamental independent factors that determine the performance of an outlet. It is therefore important to reproduce correctly in the laboratory test rig the flow depth and velocity approaching the outlet but it is not necessary to reproduce a particular roughness for the channel, defined usually by the Manning's roughness value, n.

C.3.2 Description of test rig

An existing tilting flume was adapted to carry out the testing of outfalls for surface water channels (see Figure 4). The flume is 2.4m wide by 25m long by 0.6m deep and can be tilted from horizontal to a maximum slope of 1:40. The water is circulated by means of two pumps with a combined capacity of 248 l/s; the water is drawn from a tank at the downstream end of the flume and returned by pipework to an upstream tank. The total flow is measured by a British Standard rectangular weir installed in the upstream tank.

The channels tested were built in a 12.5m long section which started 7.125m downstream from the upstream end of the flume. For the testing of both types of channel it was decided to adopt a length of at least 6m of uniform channel upstream of the test section to produce uniform flow conditions approaching the outlets.

A tank was built at the end of the channel to collect and allow measurement of the flow that bypassed the gratings. This tank measured 1.5m x 1.9m and discharged into the tank downstream of the flume by means of a 200mm diameter pipe. The outlet from the tank was protected by an anti-vortex device which is formed by a horizontal plate fixed 0.10m above the orifice. The measurement of the bypass flow could be carried out in two different ways depending on the amount of flow. For very low flows (flows smaller than 3.0 l/s) it was found to be more accurate to use the tank as a volumetric device and to measure the depth of water accumulated during a given period of time. To achieve this the flow was retained in the tank by closing a valve at the downstream end of the discharging pipe (see Figure 4). The time was recorded with a stop-watch and the depth of water was measured with the aid of a ruler stick fixed to one wall of the tank. The flow rate was calculated by dividing the volume of water by the time. For higher flows, direct values of flow rate were obtained by means of a 200mm diameter electromagnetic flow meter installed in the pipework.

Water levels in the channel were measured using electronic point gauges mounted on horizontal bars set transversely to the channel. The location of the measuring sections varied with the type of outlet studied, but generally values of the water depth were recorded in the channel at a distance upstream of the outlets between 0.5 and 1m, and also at points between gratings and downstream of the outlets to measure the depth of the bypass flow. The distance upstream of the outlets was chosen so that the water levels were representative of the flow conditions in the channel undisturbed by the drawdown at the gratings.

The flume was tilted by electrically-powered jacks which allowed tests to be carried out at any slope from practically flat to a maximum of 1:40. At the beginning of the study the flume was surveyed at various slopes in order to calibrate the readings of the tilting mechanism, and a calibration equation was obtained. A calibration of the rectangular weir for measurement of the total flow was also carried out before the start of the test programme. During the study checks were regularly done of these two calibration equations.

Rather than testing real gratings, it was decided to use representative wooden models to avoid linking the results to any particular commercial patterns (see Figure 5). The gratings were designed so that the waterway areas corresponded to the minimum values recommended in BS 497: Part1: 1976. They were made to have a waterway area of $0.44G^2$, where G is the width of the gratings. Gratings with larger waterway areas than required by BS 497 or with relatively thinner bars should therefore provide some margin of safety compared with the present experimental results. The depth of the bars in the model gratings needed to be kept as small as possible because the walls of the flume strictly limited the total depth of the collecting chamber beneath the gratings. The model bars were therefore made 25mm deep which was judged sufficient to reproduce any 'choking' effect caused by water hitting the sides of the bars and flowing out again.

C.3.3 Test procedure

In general, each outlet design was tested by first measuring its performance at mild slopes and by then increasing the slope until its capacity limit was reached. It was decided to define the capacity limit as the point at which the flow collected by an outlet falls to less than a certain percentage of the approaching flow: the figures selected were 80% for intermediate outlets and 97.5% for terminal outlets, as will be justified later in Section C.4. When the capacity limit of an outlet was reached the number of gratings was increased in stages up to a number which was thought practicable and economical to build. Although the way the tests were carried out varied in specific cases, the general procedure can be summarised as follows.

The slope of the flume was adjusted at the beginning of a test and the flow rate was set so that channel-full or surcharged conditions were achieved in the channel. These were reached when the water depth measured with the point gauges corresponded to the design water depth in the channel (channel-full) and to flooding of 1m of hard-strip (surcharged). In the testing of terminal outlets a ramp with a slope of 1:4 in the direction of the flow was positioned downstream of the outlet to simulate the end of the channel.

Sets of electronic point gauges were positioned in the channel at a distance large enough from the outlet so that accurate measurements of the water depth in the channel could be recorded. In most tests, particularly those carried out in the beginning of the study, point gauges were used to monitor water depths not only upstream of the outlet but also between gratings and downstream of them. However, it was found that the readings between gratings were not very representative of the water depth because of the irregular nature of the water surface and were therefore not used in the analysis. The readings of water depth and flow rate were taken at the beginning of a test and then checked at the end.

As with the other measurements, the measurement of the bypass flow was carried out once the flow conditions had become stable, using either the volumetric tank or the flowmeter, depending on the amount of flow.

C.3.4 Triangular channels

The laws of hydraulic similarity allow the results of tests of a particular size of channel to be used for the design of channels of other sizes provided that ratios of relevant variables are kept constant. For open channel flow, where the forces of gravity and inertia are the most relevant, the ratio is the Froude number defined as $QB^{0.5}/(gA^3)^{0.5}$, where Q is the flow rate, B is the water surface width, g is the acceleration due to gravity and A is the flow area. The tests were carried out with a 0.100m deep channel and cross-falls of 1:5 but the results are therefore also applicable to other sizes of channel and outlet that are geometrically similar.

The triangular channel was reproduced in the flume in two different parts: the upstream reach was built in wood and the 6.5m long outfall section was made of wood for the in-line outlet and moulded in smooth concrete over a wooden base for the off-line outlet. In the test rig the width of the flume was made up of a small section of verge, the surface water channel (with the verge side at a higher level than the road side to allow surcharging of the carriageway), and a width representing the hard-strip (or hard-shoulder). This latter width was built to a slope of 1:40. Plate 10 shows a general view of the flume with the in-line outlet formed by two pairs of gratings.

For the testing of both types of channel it was decided to adopt a length of at least 6m of uniform channel upstream of the test section to ensure uniform flow conditions before the approach to the outlets.

C.3.4.1 In-line outlet

The in-line outlet tested for triangular channels is shown in Figures 6 and 7 and Plate 11 and is similar to a layout developed by Cambridgeshire County Council. Although the outlet illustrated in Figure 6 consists of two pairs of gratings, tests were first carried out with a single pair. In order to maximise flow interception, the lower edges of the gratings were positioned very close to the channel invert. The distance between the two edges was 0.022m in the test rig which was thought to be representative of what is feasible in practice.

As can be seen in Figure 6, the distance between the two pairs of gratings was chosen to be equal to 0.750m to allow space for debris to deposit in the channel rather than on the gratings. The same reasoning was followed in choosing the location of the terminal ramp in the tests of the terminal outlet. These were carried out at first with the ramp 0.250m downstream of the gratings but it was observed that a longer distance was beneficial in reducing blockage of the gratings by debris.

The size of the gratings adopted in the tests was chosen to maximise flow collection in the channel tested and to conform with commercially available sizes. Gratings of size 450mm x 450mm were adopted with a waterway area which corresponded to the minimum area recommended in BS 497:Part1:1976.Tests were carried out with two different bar patterns: a diagonal pattern and a straight pattern (see Figure 5). The gratings with straight pattern could be positioned in the channel either transversely or longitudinally to the flow direction.

The results of the tests of the intermediate outlet are summarised in Table 10, and those of the terminal outlet are presented in Table 11, where Q is the flow rate, Q_p is the flow that bypassed the outlet and h is the upstream water depth in the channel. The effect of the bar pattern was also investigated by carrying out tests with similar flow conditions but with gratings having bars set diagonally, transversely or longitudinally relative to the direction of flow. The results of these tests are shown in Table 12 where the data for tests with the gratings completely removed are also given for comparison purposes.

C.3.4.2 Off-line outlet

The off-line outlet tested is shown in Figures 8 and 9 and Plate 12. Although Figure 8 shows three gratings, tests were also carried out with one and two gratings. As with the in-line outlet, the gratings were 450mm x 450mm and had a diagonal bar pattern but were positioned horizontally on the invert of the channel. The side-slope on the road side was extended below the invert level of the channel to produce a ponding effect over the gratings which increased the amount of flow collected by the outlet. Gradual transitions at 1:3 were formed between the channel and the outlet (and vice versa) to promote a smooth expansion of the flow (see Figure 8).

The results of the tests of the intermediate outlet are summarised in Table 13 for channel-full and surcharged conditions. Plate 13 shows a test carried out with surcharged conditions at a slope of 1:100.

Additional tests were carried out to try to improve the efficiency of the outlet. The previous tests had highlighted the fact that in high velocity conditions the water tended to hit the bars of the gratings nearly horizontally, which reduced the efficiency. It was therefore decided to try methods of deflecting the flow vertically so that it entered the gratings at a more downward angle. After some attempts, an increase in performance was observed with the introduction of small ramps between the gratings (see Plate 14). The results of the tests carried out with these ramps are presented in Table 14. Plate 15 illustrates one of these tests with channel-full conditions, at a longitudinal slope of 1:60.

C.3.5 Trapezoidal channel

As mentioned before, the channel tested had 1:4.5 cross-falls, a sole width of 300mm and a design surface width of 1.65m. Both the trapezoidal channel and the outfalls tested were moulded in smooth cement mortar over a wooden base.

C.3.5.1 Scale of the model

Due to the size of the flume and limitations in the flow rate achievable, it was not possible to test the trapezoidal channel at full scale. An adequate scale had therefore to be selected. It had to be large enough to ensure that similar turbulent flow conditions would occur in the scaled channel as in the full-sized one for channel-full conditions. The other criterion that determined the scale was a practical one related to the size of gratings to be adopted for the trapezoidal channel. In view of its high capacity, gratings of dimensions 610mm x 610mm are likely to be required to collect the flow from the channel efficiently in off-line outlets. Gratings of dimensions 450mm x 450mm had been made for the study of outfalls in triangular channels and it was convenient to use them again. This meant that the linear scale adopted for the trapezoidal channel needed to be 610/450=1.36 (prototype to model). As mentioned before, the Froudian similarity laws also apply in this case and the following ratios are used to convert model values into prototype ones:

Quantity	Multiplying factor
length, width, depth	1.36
velocity	$1.36^{0.5} = 1.166$
discharge	$1.36^{2.5} = 2.157$

The large scale of the model means that any scaling errors associated with the turbulence and the local channel roughness are likely to be small and much less than in most studies carried out with Froudian models.

C.3.5.2 In-line outlet

The in-line outlet tested in the trapezoidal channel is shown in Figures 10 and 11 and Plate 16. Unlike the case of triangular channels, the available width of the sole made it possible to accommodate the gratings on the base of the channel. Transitions 0.740m long were built between the channel and the outlet (and vice versa) and cross-falls of 1:4 were adopted at the outfall. The tests for the intermediate outlet were carried out with two and three gratings, and those for the terminal outlet were only performed with three gratings.

The test results (in model values) corresponding to the intermediate outlet are shown in Table 15 whereas those corresponding to the terminal outlet are shown in Table 16. Plate 17 illustrates a test with channel-full conditions at a longitudinal slope of 1:400.

C.3.5.3 Off-line outlet

The off-line outlet tested in the trapezoidal channel is shown in Figures 12 and 13 and Plate 18. This layout was tested with one, two and three gratings for the intermediate outlet and with two and three gratings for the terminal outlet. The cross-falls at the outlet were kept, as along the channel, at 1:4.5.

The test results (in model values) corresponding to the intermediate outlet are presented in Table 17 whereas those corresponding to the terminal outlet are presented in Table 18. Plate 19 illustrates a test with channel-full conditions at a longitudinal slope of 1:400.

C.3.6 Weir outlet

Although the outlets tested were adequate for a wide range of flow conditions, it was apparent from the results that they would not be able to cope efficiently with very high velocity flows such as those occurring in steep roads. Due to the velocity of the flow, the water tended to jump over the grating slots and little was collected. The option of forming ramps upstream of each grating to direct the flow into the slots was not favoured by DOT on safety grounds (see Section C.3.4.2). For the same reason, the option of using gratings with longitudinal slots was also discarded. An altogether different geometry of outlet was therefore devised for these situations. After some trials, it became clear that a better solution would be to direct the water gently away from the carriageway onto the verge side and then over a side weir into a lower collecting chamber.

The options available for this type of outlet are very limited by safety considerations which do not allow any deepening of the channel below 150mm or side-slopes locally steeper than 1:4. A weir outlet was first built in the flume with a layout similar to that shown in Figure 14 but with a straight transition on the carriageway side. This geometry was later changed in order to produce a more gradual turning of the flow towards the weir. In the proposed geometry, the side transition is initially curved in plan and a safety fence will normally be necessary along the side weir to prevent the possibility of vehicle wheels dropping into the lower side channel. Although the weir outlet was tested only with the trapezoidal channel shape, it is also applicable with modifications to triangular channels.

When a high-velocity flow is turned laterally, the water level on the outside of the bend can increase significantly. Thus, with the weir outlet design, it is very difficult to prevent water spilling out on to the hardstrip if the channel is flowing full at the entrance to the transition section. It is therefore necessary to lower the water level entering the transition so that some freeboard is available when the flow is turned towards the side weir; the smaller the amount of freeboard, the more gradual and longer the transition needs to be to prevent water flowing back on to the road.

Tests were initially carried out with a 610mm x 610mm grating (prototype size) installed on the invert at the upstream end of the outlet (see Plate 20). At lower channel slopes and flow velocities, the grating removed enough water to create the necessary freeboard in the transition section. However, the grating became progressively less effective at higher velocities so that very long lengths of transition and side weir would have been needed to prevent overtopping. Results from these tests, which were carried out with channel-full conditions upstream of the grating, are given in model terms in Table 19. The table contains measured values of water depth in the channel as well as the water depths at the downstream end of the grating, y_a, and the water depths y_b, which will be explained later.

The use of a gully grating in combination with the weir outlet does not provide a complete solution for all cases and the extra complication and cost may not be justified. It was therefore decided that a more practical alternative would be to limit the design flow depth in the surface water channel to less than the channel depth upstream of the weir outlet; this would therefore provide the freeboard needed in the transition section of the weir outlet. If a triangular surface water channel is relatively small, it would be possible to locally increase its size just upstream of the weir outlet so that it is flowing part-full under design conditions. This option is not possible in the case of the trapezoidal channel or large triangular channels because the depth cannot be increased beyond the allowable limit of 150mm. In these cases, it will be necessary to determine the spacing between the outlets so that the channel flows only part-full under design conditions. Although this means that the channel has effectively to be over-sized, it will normally only be necessary at a few locations where steep longitudinal gradients occur. Although the channel cannot be used to its full capacity, long spacings will still be possible because the steep gradients will still produce high flow rates under part-full conditions.

Tests with the weir outlet shown in Figure 14 were therefore carried out with the trapezoidal channel upstream flowing 83% full and 68% full. In the test rig the angle θ was equal to 22°, the total length of the weir, L_w, was 4.5m and the lengths L_s and L_a were 1.3m and 3.2m, respectively. The results of the tests (in model terms) are given in Table 19.

C.4 PREPARATION OF ADVICE NOTE ON OUTFALL DESIGN

C.4.1 Analysis of laboratory tests

Although the field tests described in Section C.2 provided information for the preparation of the Advice Note on Outfall Design, the range of conditions achievable on site was not sufficiently wide to be used for the recommendation of suitable designs. The great variety of channel and outlet geometries that can be found in existing schemes is such that it does not allow a systematic study of a particular geometry at different slopes. For this reason the results of the laboratory tests were used as the basis for the preparation of the Advice Note on Outfall Design, which is presented in Appendix III.

Different approaches were adopted in the analysis of the grated outlets in the triangular channel and in the trapezoidal channel, and in the analysis of the weir outlet. As mentioned before, the study of outfalls in triangular channels was carried out in the laboratory using a particular size of channel. If the results are analysed in non-dimensional form, the triangular shape allows correct transposition of results between channels of different dimensions: for channels with the same cross-falls there is direct proportionality of the water depths and hydraulic radii in different sizes of channels. The study of the trapezoidal channel was carried out as a Froudian model at a scale of 1:1.36 (see Section C.3.5.1). Due to the limited number of test results with the weir outlet and the need to extrapolate results to very steep slopes, a theoretical approach was adopted for the analysis. This was based on the oblique wave theory as described later in Section C.4.4.

For the analysis of the test data it was necessary to assess the performance of the outlets under various flow conditions. For this purpose, the efficiency of an outlet was defined as the ratio of the flow intercepted by the outlet, Q_i, to the total flow approaching it:

$$\eta_o = Q_i/Q_o \tag{2}$$

$$\eta_s = Q_i/Q_s \tag{3}$$

where subscripts o and s refer to channel-full and surcharged conditions. Based on previous studies of road drainage, it was decided to adopt a minimum efficiency of 80% for intermediate outlets operating under channel-full conditions. When an outlet does not achieve this minimum a different geometry is assumed to be required for the outlet. For terminal outlets a minimum efficiency of 97.5% was adopted: terminal outlets need to be designed for efficiencies close to 100% because any substantial bypass may cause flooding of the verges or the carriageway. It should also be added that terminal outlets have generally higher efficiencies than intermediate outlets due to the ponding effect caused by the terminal ramp.

As mentioned before (see Section C.3.4.1), the present study also assessed the effect of bar patterns of gratings different from diagonal. In cases where surface water channels are built behind safety fences it is possible to adopt gratings with a longitudinal bar pattern. The tests showed that these have a higher efficiency when compared with gratings of the same overall size and waterway area but diagonal bars. From tests carried out with similar flow conditions it was possible to establish the following approximate relationship between the efficiency η of equivalent gratings with diagonal and longitudinal bars:

$$\eta_L = 0.5 + 0.5\eta_D \qquad (4)$$

where subscripts L and D correspond to longitudinal and diagonal bar patterns, respectively.

It was also observed in the tests that the efficiencies of gratings with bars transverse to the flow were significantly smaller than those with diagonal bars. This is more noticeable in higher velocity flows where the water tends to hit the bars of the grating; the bigger the horizontal angle between the bars and the direction of the flow, the more the water will jet over the grating (compare, for the same total flow, the values of the bypass flow Q_p for diagonal and transverse bars in Table 12).

C.4.2 Triangular channels

The Froude number was used as the basis for the analysis of the data for triangular channels and is defined as follows:

$$F = \frac{QB^{0.5}}{g^{0.5}\,A^{1.5}} \qquad (5)$$

where Q is the approach flow to the outfall, B is the water surface width just upstream of the outfall, A is the corresponding flow area and g is the acceleration due to gravity.

For triangular channels with side-slopes of 1:5 and channel-full conditions the Froude number is given by:

$$F_o = \frac{28.56\ Q_o}{B_o^{2.5}} \tag{6}$$

where Q_o is the approach flow (in m^3/s) and B_o is the surface width of flow (in m) for channel-full conditions. The numerical constant was determined so that F_o is equal to 1 when the flow in the channel is at critical depth for channel-full situations.

The flow conditions occurring under surcharged conditions are more complex because the velocity of the water in the main channel is considerably greater than in the shallow flow along the hard strip or hard shoulder. Since the allowable depth of surcharging in surface water channels is fixed at 25mm above the normal design depth, the resulting increase in flow capacity is relatively larger in small channels than in large ones. To assist users of the Advice Note, a design chart was produced showing the relationship between the surcharged capacity, Q_s, and the design capacity, Q_o, for different sizes of channel (as defined by the design flow width B_o). The curve, which is shown in Appendix III (Figure 3 of the Advice Note) was obtained from the equations for surcharged channels given in Section 15 of HA 37/88; the values assumed were $y_3 - y_1 = 25$mm, $y_3 - y_2 = 20$mm, n = 0.013 for the channel and n = 0.017 for the hard strip. The values of Q_s/Q_o obtained from the tests with the laboratory channel (B_o = 1.00m) were found to be in reasonable agreement with the corresponding value given by Figure 3 of Appendix III.

As explained above, the test data can be applied to other sizes of triangular channel if the results are expressed in terms of interception efficiency, η, versus a non-dimensional Froude number. Geometric similarity between different sizes of channel is not exactly achieved under surcharged conditions because the depth of surcharging is fixed at 25mm. However, the differences are relatively small and the laboratory results will err on the safe side for channels larger than the one tested (B_o = 1.00m). Since the flow on the hard strip has very little influence on the performance of an outlet, it is appropriate to define the Froude number in terms of the surcharged width, B_1, in the main channel (see Figure 1 of Advice Note in Appendix III). The definition adopted was therefore:

$$F_s = \frac{24.6\ Q_s}{B_1^{2.5}} \tag{7}$$

where Q_s is in m^3/s and B_1 is in m.

As in the case of F_o in Equation (6), it is convenient to choose the value of the numerical coefficient so that F_s = 1 when critical flow conditions occur in a surcharged channel. Several alternative methods of calculating critical flow in channels with compound cross-sections have been proposed, and the value of 24.6 was determined for the test channel using an approach due to Konemann (1982). The coefficient will vary somewhat for other sizes of channel, but the choice is purely a matter of convenience provided the same definition of F_s is used in the analysis of the test data and in design.

The Froude numbers F_o and F_s were calculated for all the tests carried out with triangular channels and were plotted against the efficiency of the outlet, as shown in Figures 15 to 18. Design curves for various numbers of gratings were drawn through the experimental points.

In a few cases the test results were extrapolated to cover conditions not reproduced in the laboratory tests. Two different situations were considered: extrapolation to a larger or smaller number of gratings; and extrapolation to flow conditions which could not be achieved in the test rig (eg flows at slopes steeper than 1:40). The first type of extrapolation was carried out for the in-line outlet with three pairs of gratings and for one and two gratings in the surcharged off-line outlet, as can be seen in Figures 15, 16 and 18 (dashed lines). A conservative approach was adopted when extrapolating results from the tests so that the recommended curves would lead to safe designs. It was assumed that, for the same flow conditions, the individual efficiency of any additional gratings that were not tested would be the same as that of gratings further upstream. This is a conservative approach because the efficiency tends to increase as more of the flow is intercepted by gratings positioned upstream. The shapes of the curves for which considerable amounts of data existed were also taken into account in the extrapolations. The second type of extrapolation was carried out in order to extend the range of the design curves to steeper slopes. The intercepted flow corresponding to the most severe conditions tested was assumed to remain constant for steeper slopes; the efficiencies were calculated by dividing the intercepted flow by the total flow calculated using Manning's equation with a mean roughness coefficient of 0.013.

The gratings recommended in the Advice Note for the in-line outlet are specified to have sizes within certain limits (see Appendix III). The limits are given in terms of the channel design depth so that they are applicable to channels of various dimensions. The lower limit was directly obtained from the tests and corresponds to the minimum width of grating that can achieve the necessary performance; the upper limit corresponds to the maximum width that can physically be installed in the channel. For the off-line outlet only the lower limit is applicable.

C.4.3 Trapezoidal channel

The design curves shown in Figures 19 to 22 give the efficiency of the outlets in terms of the total flow approaching the outlet (Q_o or Q_s for channel-full and surcharged channel, respectively). Assuming the Manning resistance equation for the trapezoidal channel tested there is a fixed relationship between Q_o and Q_s, ie. $Q_s = 1.21\ Q_o$.

The experimental points are plotted in these figures; in the case of the off-line outlet, data points corresponding to one single grating can be seen plotted in Figures 21 and 22. However, it was decided not to present a design curve in the Advice Note for an outlet with one grating only because of the risk of blockage in such a high capacity channel. Extrapolations from the test data were carried out using the same procedures as those described in Section C.4.2 for triangular channels.

After the tests were completed, DOT requested that the Advice Note should also include curves for the design of trapezoidal channels with the same base width of 0.3m but side-slopes of 1:5 (see Appendix III). The curves obtained for the efficiency of outlets in the 1:4.5 channel were therefore revised in order to produce conservative recommendations for the 1:5 trapezoidal channel. The concept of channel conveyance was used to estimate the relative difference in flow capacity between the 1:4.5 and 1:5 trapezoidal channels for equal values of longitudinal slope and roughness. The ratio of the conveyances of the two channels was calculated assuming the Manning resistance equation and is given by:

$$\frac{A_5 \, R_5^{2/3}}{A_{4.5} \, R_{4.5}^{2/3}} = 1.071 \qquad (8)$$

where $A_{4.5}$ and A_5 are the cross-sectional areas of the channels and $R_{4.5}$ and R_5 are the two hydraulic radii. It was assumed that the flow rate collected by an outlet in the 1:5 channel would be equal to that measured under the same conditions of slope and roughness in the 1:4.5 channel. This means that the efficiency of the outlet in the 1:5 channel is assumed to be about 6% smaller than in the tested channel because of the higher corresponding flow rate in the 1:5 channel. The design curves are therefore likely to produce slightly conservative designs.

Another modification to the Advice Note was requested by DOT after completion of the laboratory tests. In order to promote self-cleansing conditions, the 300mm wide soles of the trapezoidal channels are likely to be required to have a slope of 1:40 towards the verge or central reserve (see Figure 2 of Advice Note in Appendix III). Although the tests were carried out with horizontal soles, it is considered that the results for outlet efficiency should not be significantly affected by a bottom slope as small as 1:40. The values of the design flow capacity of the channels presented in the Advice Note were re-calculated taking the sloping sole into account.

C.4.4 Design of weir outlet

The experimental data that were available for the design of the weir outlet was limited to longitudinal slopes up to 1:40 since this was the maximum slope achievable in the tilting flume. However, as mentioned in Section C.3.6, an outlet such as the weir outlet where the water is diverted away from the carriageway, is recommended for steep road schemes. It was therefore necessary to extrapolate the test results in order to calculate the dimensions of the weir outlet that would guarantee a suitable performance of the outlet for flow conditions outside those tested. Due to the limited amount of data available, a theoretical approach was adopted for the calculations. The oblique wave theory, which can be found in standard hydraulics text books such as Henderson(1966), provided the basis for the analysis. In supercritical flow (ie. Froude number greater than 1), any disturbance to the flow creates a surface wave which propagates across the flow and also downstream, producing an oblique standing wave or jump. A change in direction of a channel wall as in the proposed weir outlet (see Figure 14) will generate such a disturbance; if the wall is angled towards the flow, the water level downstream of the wave front will be higher than on the upstream side.

The equations that describe the formation of oblique waves caused by such disturbances are:

$$\sin\beta = \frac{1}{F_a}\left[0.5\frac{y_b}{y_a}\left(\frac{y_b}{y_a}+1\right)\right]^{0.5} \qquad (9)$$

and

$$\frac{y_b}{y_a} = \frac{\tan\beta}{\tan(\beta-\theta)} \qquad (10)$$

where F_a and y_a are, respectively, the Froude number of the flow and the water depth upstream of the disturbance, and y_b is the depth of water downstream of the wave; the angle β is the angle of the oblique wave in relation to the direction of the flow and θ is the angle of deflection of the wall of the channel (see Figure 23).

The applicability of the oblique wave theory was investigated by comparing its predictions with the results of the tests carried out on the weir outlet (see Table 19). Some of the tests were made with the upstream trapezoidal channel flowing full but with a gully grating installed (see Plate 20) which had the effect of removing some of the water and lowering the flow depth approaching the transition on the carriageway side of the channel. Other tests were made without the grating in operation but with the upstream channel flowing either 83% or 68% full. In the tests the discharge and the slope of the channel were varied, and measurements were made of the water depth upstream of the transition and of the corresponding maximum downstream depth produced by the oblique wave. For a particular upstream condition, the limiting flow capacity of the outlet was obtained when the downstream water level just reached the top of the channel. The efficiency of the outlet in this limiting state was 100%; any increase in discharge would have caused some water to spill out onto the carriageway and bypass the outlet.

Calculations were first made using the measured values of water depth y_a and y_b just upstream and downstream of the oblique wave (see Table 19). On this basis, the predicted values of wall angle, θ, given by Equations (9) and (10) varied from 14.0° to 26.5°, with an average value for seven tests of 20.0°. These results compared satisfactorily with the actual weir angle of θ = 22° used in the model, and suggested that the oblique wave theory was a reasonable basis for design. However, study of the data in Table 19 showed that the relationship between the upstream water depth, h, in the channel and the local depth, y_a, at the start of the transition was complex and difficult to predict. It was therefore decided to re-analyse the data for the channel flowing 83% and 68% full using the measured values of h in place of y_a in Equations (9) and (10), since the upstream depth, h, is the parameter that is specified in the design situation. The predicted values of θ given by the equations varied from 9° - 11° for the channel flowing 68% full to 5° for the channel 83% full. As explained, the differences relative to the actual weir angle of θ = 22° were due to the local reduction in water depth that occurs as the flow approaches the side transition. Also, in some cases, the oblique wave formed on the curved portion of the transition where the effective value of θ was less than along the straight portion.

Based on these results, it was decided to use the oblique wave theory to produce general design curves for the Advice Note relating the wall angle, θ, and the total weir length, L_w, to the upstream flow conditions in the channel. The required values of θ were assumed to be greater than the predicted values, θ_p, given by Equations (9) and (10) according to the ratio: $\tan \theta = 2 \tan \theta_p$; this assumption is on the safe side compared with all the test data. After considering alternative options, it was decided to base the curves on a specified proportional flow depth of 67% in the channel upstream of the outlet. This gave a reasonable balance between the required length of the outlet structure and the loss of potential flow capacity in the channel due to the need to design for part-full conditions. Although the full capacity cannot be utilised, high flow rates can still be achieved because of the steep channel gradients that apply when weir outlets are necessary.

In the case of trapezoidal channels, the design curves are based on the flow rate approaching the outlet (see Figures 29 and 31 of Advice Note in Appendix III). Although tests were not carried out with triangular channels, the oblique wave theory is still applicable because the angle of the side transitions is the principal factor determining the limiting capacity of the outlet. The design curves (see Figure 27 of Advice Note in Appendix III) are defined in terms of the upstream Froude number, F_o, corresponding to the proportional flow depth of 67%; this enables the curves to be applied to different sizes of triangular channel.

The total length, L_w, of the weir (see Figure 14) is made up of two components. L_s corresponds to the upstream section of the outlet which has a straight side wall. Based on the laboratory tests, L_s is related to the overall width of the upstream channel by:

$$L_s = K B_1 \tag{11}$$

where K is a constant which is equal to 1.0 for the trapezoidal channel tested and is equal to 1.2 for triangular channels. Since the tests of the weir outlet were only carried out with a trapezoidal channel, it was decided to adopt a higher value of K for triangular channels. This higher value was chosen in order to give the required initial distance for the flow to expand. The second component, L_a, depends on the angle, θ, of the side transition and is given by:

$$L_a = \frac{B_1}{\tan \theta} \tag{12}$$

C.4.5 Design of outfall structures

The chambers beneath the outlet gratings and the structures that convey the water to suitable discharge points (eg watercourses, soakaways or surface water sewers) can take a variety of forms, and designers need to be able to develop solutions to suit the requirements of particular sites. However, the Advice Note does give some general guidance on possible layouts of chambers and on methods of sizing the outfall structures.

Standard circular gully pots can be used for outlets that consist of a single grating or of two gratings installed in the in-line arrangement shown in Figure A.2 of the Advice Note (see Appendix III). For larger outlets, it may be more convenient to construct a single brick or concrete chamber beneath the gratings. In this case, the invert level of the outgoing pipe from the chamber should be above the floor so as to enable sediment to deposit and not be discharged into the downstream pipe system or watercourse. As a rough guide, the depth allowed for storage of sediment should not be less than that provided by standard circular gully pots. The high flow rates that will occur through outlets from surface water channels may cause the sediment-collecting efficiency of the chambers to be less than is achieved with gully pots in normal kerb-and-gully situations. Standard circular gully pots also have a limited flow capacity and designs with an outlet pipe of 150mm diameter may not be able to pass more than about 25l/s without the water level reaching close to the underside of the gratings.

The dimensions of a collecting chamber should therefore be chosen so that there is sufficient depth for collection of sediment and sufficient head to allow the outgoing pipe to discharge the design flow without causing backing up to road level. The head, Z, required above the invert of the outgoing pipe can be estimated by assuming that the entrance to the pipe acts as an orifice with an area contraction ratio of 0.6; this leads to the following design equation:

$$Z = \frac{D}{2} + 0.23 \frac{Q^2}{D^4} \tag{13}$$

where Z is in m, D is the pipe diameter in m and Q is the flow rate in m^3/s. It is recommended that the water level in the chamber should be at least 150mm below the underside of the gratings when the flow rate is equal to that in the surface water channel under surcharged conditions (ie Q_s). The size of the outgoing pipe and the gradient at which it is laid should be determined from standard flow tables or resistance equations (eg Colebrook-white) so that the pipe is just flowing full at the flow rate of Q_s. In fact, this will usually be the first step in the design procedure. Once the pipe diameter, D, has been determined, the required level of the pipe in the chamber can be calculated using Equation (13); the floor level of the chamber is then set so as to provide a sufficient volume for collection of sediment.

If a weir outlet is required, it is necessary to determine the depth of the collecting channel into which the flow from the side weir discharges. The collecting channel receives inflow along its length and is hydraulically equivalent to a roof gutter for which design information is given in British Standard BS 6367 (1983). Using this information it can be shown that the required channel depths, J, below the level of the weir is given by:

$$J = 0.15 + 1.3 \left(\frac{Q}{E}\right)^{2/3} \tag{14}$$

where J is in m, Q is in m^3/s and E is the width of the rectangular channel in m. The 0.15m freeboard figure is included to ensure that the weir is able to discharge freely into the collecting channel. As before, it is recommended that the value of J should be determined at a flow rate of Q_s, corresponding to surcharged conditions in the surface water channel. The collecting channel should discharge into a chamber in order to collect any sediment and still the flow before it enters the piped drainage system. The chamber should be designed as described above and with the design water level at least 0.5m below the level of the weir.

C.4.6 Analysis of field tests

It was mentioned at the beginning of Section C.4.1 that, due to the difficulty in obtaining systematic results from field tests, the Advice Note was mainly based on the results of the tests carried out in the laboratory. The field tests were later analysed according to the recommendations of the Advice Note. This was not possible for the tests of the verge channels at the A20 Folkestone to Dover (Contract 1) because one of the channel cross-falls was 1:1 and the Advice Note only covers symmetrical channels with cross-falls of 1:5.

The outlets tested in the central reserve of the A20 Folkestone to Dover (Contract 1) were terminal outlets formed by triple gratings, measuring 450mm x 1350mm overall, set flat on the line of the channel invert. This particular layout does not fall into any of the types of outfall recommended in the Advice Note, but it was nevertheless decided to check its design against the recommendations for terminal in-line and off-line outlets. The values of the Froude number were calculated for the three tests and are presented in Table 20. It can be seen in Table 1 of Appendix III that one pair of gratings on the side-slopes or a single grating of dimensions 450mm x 450mm positioned on the invert would be adequate for all the tests. It appears that these outlets are slightly over designed but their extra capacity of the outlets may be put to use if the maintenance of the channels is relaxed for some reason. The over design of the outlets also reflects the adoption of a 'safe design' procedure by engineers in view of the lack of design guidelines available until now.

The same analysis was carried out on the test results obtained at the A 487 Port Dinorwic Bypass. The fact that the sizes of the gratings adopted in this scheme were, as at the Folkestone scheme, different from the sizes considered in the Advice Note means that only an approximate comparison can be made. The Froude numbers were calculated for all the tests from the measurements of water depth and flow rate and are presented in Table 20. It is important to note that the tests were carried out with the channels flowing part-full whereas the Advice Note was developed to apply specifically to channel-full conditions. In part-full flows the ratio of grating width, G, to water depth can be significantly different from the ratio present in laboratory tests which were carried out with channel-full conditions. This may affect the present comparison, and an additional difference is that at Port Dinorwic the gratings are positioned horizontally on the invert of the channel and are of a bigger size than considered in the Advice Note (note that the Advice Note is based on the minimum waterway areas of gratings currently recommended by the British Standard). It can be seen in Table 20 that for outlets in the 1:19 slope the Advice Note would recommend a weir outlet in most cases if the channel was flowing full. The fact that no bypass flow was observed is probably due to the channel being only part-full, which allowed the gratings to collect the flow more efficiently. Another factor is related to the uncertainty in some values of flow (and therefore velocity) measured on site. The tests on the lengths of channel at slopes of 1:16.5 and 1:204 slopes show a better agreement with the recommendations of the Advice Note. It is interesting to note in particular test no.7 where an efficiency of 97% was measured on site due to some flow bypassing. The Advice Note shows that a similar value is obtained with a grating of dimensions 450mm x 450mm. Overall it appears that the outlets in the steeper channels at Port Dinorwic may be underdesigned for the very high velocity flows that are generated in these locations.

C.5 CONCLUSIONS

(1) Although the field tests provided information for the preparation of the Advice Note on Outfall Design, the range of conditions achievable on site was not sufficiently wide to be used for the recommendations of suitable designs. Therefore, the Advice Note was mainly based on the results of the laboratory tests.

(2) The laboratory tests showed that grated outlets are not able to cope efficiently with very high velocity flows such as those occurring in steep roads (typically steeper than 1:50). In these situations it is recommended to direct the water gently away from the carriageway onto the verge side and then over a side weir into a lower collecting chamber.

PART D CONCLUSIONS AND RECOMMENDATIONS

D.1 MAIN CONCLUSIONS AND RECOMMENDATIONS FROM PART B

(1) Extensive information on the use of surface water channels for road drainage was obtained through questionnaires, meetings and site visits. The adoption of this type of surface water drainage has rapidly spread in the UK in recent years but most of the schemes identified are in the East and South East of England, and in Wales.

(2) It is suggested that DOT produces a stronger recommendation in the HCD regarding the level of the outer edge of channels in the central reserve, this level has sometimes been incorrectly built higher than the carriageway level which can potentially cause flooding of the fast lane of the road with associated safety hazards.

(3) Standard construction details should be developed for channels protected by safety barriers to increase the range of options available to designers.

(4) The current specification of 0-10mm for the downward step between the pavement surface and the edge of the concrete channel can be difficult to achieve and make consistent with the tolerances allowed for the bituminous wearing course. These two different tolerances need to be reviewed in parallel.

(5) Tolerances on level for surface water channels should not only apply to the edge of the channel but also to the invert since it is the difference of the two levels that determines the capacity of the channel.

(6) It is recommended that work should be carried out on the development of suitable techniques for adding overlays to roads with surface water channels. This work should be given high priority so that adequate techniques are available when structural overlays become necessary in existing schemes.

(7) The workability of the concrete mix should be primarily agreed between the slipforming contractor and the concrete supplier within the recommendations of BS 5931, 1980.

D.2 MAIN CONCLUSIONS AND RECOMMENDATIONS FROM PART C

(1) An Advice Note on Outfall Design was prepared using the results obtained from the laboratory and field tests, and from the information gathered from the questionnaires and site visits carried out in the first part of the study.

(2) The laboratory and field tests showed that grated outlets are not efficient in collecting very high velocity flows which typically occur in roads steeper

than 1:50. A new design which directs the water away from the carriageway over a side weir was developed from experimental data and theoretical calculations.

(3) The work carried out to develop methods for the design of outfalls in surface water channels has some implication on the existing Advice Note HA 37/88 and on Amendment No 1 which deal with the hydraulic design of surface water channels. These need to be revised so that consistency is achieved with the newly prepared Advice Note on Outfall Design. The topics that require development work are the following:

(a) Design curves for determination of the spacing between outlets in the trapezoidal channels considered in the present study;

(b) The effect that flow bypassing intermediate outlets has on the spacing of outlets;

(c) Revision of HA 37/88 to cover storm durations of 2 to 20 minutes. The need for revision of this topic had been identified some time ago, but it was made clear during the meetings with designers as described in Part B of this report.

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Tables

LEGEND FOR TABLES 1 TO 4

NOTATION:

C - Construction
D - Design
M - Maintenance
MP - Maintenance Period
T - Tender

R - Rural
U - Urban

COMP - Composite
FLEX - Flexible

FD - Fin Drain
NFD - Narrow Filter Drain

F - Flat

EX - Extruded
IS - In Situ
PC - Precast
SF - Slip Form

GC - Grating in Channel Invert
GV - Grating Back in Verge

CD - Carrier Drain
SO - Soakaway
TD - Toe Ditch
WC - Water Course

SCHEMES:

- ENGLAND -

(1) LONDON:

(2) EASTERN:

2.1 - A47 Norwich Southern BP (Contract 1)
2.2 - A47 Norwich Southern BP (Contract 2)
2.3 - A47 Norwich Southern BP (Contract 3)
2.4 - A47 Norwich Southern BP (Contract 4)
2.5 - A11 Thetford BP
2.6 - A11 Red Lodge BP
2.7 - A1-M1 Link (Contract 8)
2.8 - M40 Widening
2.9 - A11 Besthorpe - Wymondham Improvement
2.10 - A5 Little Brickhill BP

(3) EAST MIDLANDS:

3.1 - A16 Louth BP
3.2 - A16 Boston BP
3.3 - A6 Quorn-Mountsorrel BP
3.4 - A6 Market Harborough BP

(4) NORTHERN:

(5) NORTH WEST:

5.1 - A500 Nantwich BP
5.2 - A523 Macclesfield Relief Road
5.3 - A49 Weaverham Diversion

(6) SOUTH EAST:

6.1 - A21 Pembury BP
6.2 - M20 J5-J8 Maidstone BP
6.3 - A23 Muddleswood-Patcham
6.4 - A20 Folkestone-Court Wood (Contract 1)
6.5 - A20 Court Wood-Dover (Contract 3)
6.6 - M3 Bar End-Compton
6.7 - A27 Westhampnett BP

NOTATION:

O - Other

ND - Not Decided

CA - Carriageway

CR - Central Reserve

CH - Channel

SCHEMES:

6.8 - A3 Milford BP

(7) SOUTH WEST:

7.1 - A36 Beckington BP

7.2 - A30 Okehampton-Launceston Improvement

(8) WEST MIDLANDS:

8.1 - A49 Dorrington BP

(9) YORKSHIRE AND HUMBERSIDE:

9.1 - A15 Bonby Lodge BP

9.2 - A19 Easingwold BP

9.3 - A1 Motorway Walshford to Dishford

- NORTHERN IRELAND -

10.1 - Strabane BP

- SCOTLAND -

11.1 - A7 Moss Peeble to Bush

- WALES -

12.1 - A4042 Llantarnam BP

12.2 - A472 Maesycurmmer/Newbridge

12.3 - M4 Renewal

12.4 - A494 Mold BP

12.5 - A487 Port Dinorwic BP

12.6 - A465 Neath-Abergavenny Trunk Road

Table 1 Summary of results of questionnaire Overall description of scheme

Scheme	Stage	Carriageway cross-section	Type of Pavement	Length of Road (km)	Width of carriageway (m)	Longitudinal gradients		Type of Drainage
						Max	Min	
(1) London CPD					No schemes			
(2) Eastern NMD								
2.1	MP	R-AP / A2	FLEX	5.2	7.3 + 2 x 1	1:29	F	FD, T6, F19
2.2	MP	R-AP / A2	FLEX	4.3	7.3 + 2 x 1	1:25	F	FD, T6
2.3	MP	R-AP / A2	FLEX	7.1	7.3 + 2 x 1	1:25	F	FD, T6
2.4	MP	R-AP / A2	FLEX	5.8	7.3 + 2 x 1	1:36	F	FD, T6
2.5	M	R-AP / A2	COMP	7.6	7.3 + 2 x 1	1:25	1:285	FD, T6 mod
2.6	MP & M	R-D2 AP / A2	COMP	2.75	7.3 + 2 x 1	1:166	1:222	FD, T6
2.7	MP	R-AP / A2	FLEX	6.2	7.3 + 2 x 1	1:50	1:3000	FD, T6
2.8	MP	R-M / A1	FLEX	11.95 - 13.55	11 + 3.3	1:28	F	NFD, T8
2.9	T	R-AP / A2	--	8.6	7.3 + 2 x 1	1:24	F	--
2.10	C	R-AP / A2	FLEX	3.6	7.3 + 2 x 1*	1:25	F	NFD, T8
(3) East midlands CPD								
3.1	M	R-AP / A2	FLEX	6.0	7.3 to 10 + 2 x 1*	1:12.5	1:140	NFD, T9 ϕ100 mm
3.2	M	R-AP / A2	FLEX	9.1	7.3 + 2 x 1.65	1:175	1:2300	FD, T6
3.3	M	R-AP / A2	FLEX	8.8	7.3 + 2 x 1	1:200	F	NFD, T8
3.4	MP	R-AP / A2	FLEX	8.4 (1.48 CH)	7.3 + 1	1:56	1:500	NFD, T8

Table 1 Continued

Scheme	Stage	Carriageway cross-section	Type of Pavement	Length of Road (km)	Width of carriageway (m)	Longitudinal gradients		Type of Drainage
						Max	Min	
(4) Northern CPD					No reply			
(5) North West CPD								
5.1	M	R-AP / A2	FLEX	3.4	7.3 + 2 x 1*	1:33	F	NFD, T8
5.2	MP	R-AP / A2	FLEX	4.15	7.3 + 2 x 1	1:26	F	NFD, T8
5.3	MP	R-AP / A2	FLEX	3.3	7.3 + 2 x 1*	1:43	1:200	FD, T6
(6) South East CPD								
6.1	M	R-AP / A2	FLEX	5.0	7.3 + 2 x 1	1:18	F	NFD, T8
6.2	C	R-M/A1, AP/A2 and ML/A3	FLEX	11.0	14.6 + 5.3* 11.0 + 5.3*	1:27	1:200	FD, T5, T6 NFD, T8
6.3	MP	R-AP / A2	FLEX	5.6	9.3 - 11.0	1:20	1:4500	NFD, T8
6.4 a)	C	R-AP / A2	FLEX	6.0 (2.0 CH)	7.3 + 2 x 1*	1:93	F	FD, T6
6.4 b)	C	R-MS / A5	FLEX	--	6.0 + 2 x 1*	1:17	1:625	FD, T6
6.5	C	R-AP / A2	FLEX	7.0	7.3 + 2 x 1	1:15	F	FD, T6
6.6	C	R-M / A1	FLEX	6.0	11.0 + 5.3*	1:25	F	FD, T5, T6, T7
6.7	C	R-AP / A2	FLEX	3.05	7.3 + 2 x 1	1:250	F	FD, T5
6.8	MP	R-AP / A2	FLEX	3.4	7.3 + 2 x 1*	1:25	F	NFD, T9

Table 1 Continued

Scheme	Stage	Carriageway cross-section	Type of Pavement	Length of Road (km)	Width of carriageway (m)	Longitudinal gradients		Type of Drainage
						Max	Min	
(7) South West CPD								
7.1	M	R-AP / A2	FLEX	3.4	7.3 + 2 x 1*	1:25	1:284	NFD, T8
7.2	C	R-AP / A2	FLEX	20.0	7.3 + 2 x 1	1:12	1:200	NFD, T8
(8) West Midlands CPD								
8.1	D	R-AP / A2	FLEX	7.2	7.3 + 2 x 1	1:25	1:200	ND
(9) Yorkshire and Humberside CPD								
9.1	M	R-APS / A6	FLEX	1.5	6.0 + 2 x 1*	1:23	F	NFD, T8
9.2	C	R-AP / A2	FLEX	4.6	7.3 + 2 x 1	--	F	NFD
9.3	T	R-M / A1	FLEX	20.0	11.0 + 3.3	1:32	F	NFD, T9
(10) Northern Ireland								
10.1	M	U-AP / A9	FLEX	1.306 (CH)	7.3 + 2 x 1	1:50	1:1000	FREE
(11) Scotland								
11.1	D	R-WS / 2	FLEX	1.5	10.0	1:22	1:200	FD, T5, COMB

Table 1 Continued

Scheme	Stage	Carriageway cross-section	Type of Pavement	Length of Road (km)	Width of carriageway (m)	Longitudinal gradients		Type of Drainage
						Max	Min	
(12) Wales								
12.1	D	R-AP / A2	FLEX	3.2	7.3 + 2 x 1*	1:29	1:100	FD, T5, NFD, T8
12.2	C	R-AP / A2	--	6.2	7.3 + 2 x 1*	1:16	F	FD. T6. NFD, T9, O
12.3	MP	R-M / A1	COMP	5.5 (2.0 CH)	VAR	1:100	F	NFD T8
12.4	MP	R-AP / A2	FLEX	5.8	7.3 + 2 x 1	1:16.7	1:148	NFD, T8
12.5	C	R-AP / A2	FLEX;COMP	5.3	10.0	1:16.6	1:2000	NFD, T8
12.6	D	R-AP / A2	FLEX	15.0	14.6 + 3.3*	1:85	F	FD, T5, T7

* Width of hard-strip or hard-shoulder was estimated

Table 2 Summary of results of questionnaire Surface water channels in verge

Scheme	Pavement Edge Detail		Method of Construction	Geometry of channel			
	Verges in cutting	Verges in Embankment		Design Flow Width (mm)	Overall Depth (mm)	Side slopes	
(1) London CPD			No schemes				
(2) Eastern NMD							
2.1	B3, F21	B12, F21	SF	920	175	1:4.43	1:0.83
2.2	B3, F21	B12, F21	SF	945	175	1:4.43	1:0.83
2.3	B3, F21	B12, F21	SF	945	175	1:4.43	1:0.83
2.4	B3, F21	B12, F21	SF	920	175	1:4.43	1:0.83
2.5	O	O	SF	950 1075	175 150	1:1 1:1	1:4 1:6
2.6	B3	B12	SF	1000	150	1:5	1:1
2.7	--	--	--	--	--	--	--
2.8	O	B12	SF	600	88	1:1.25	1:5
2.9	B2, B3	B11, B12	--	750	125*	1:1	1:5
2.10	B3	B12	SF	840	140	1:1	1:5

Table 2 Continued

Scheme	Pavement Edge Detail		Method of Construction	Geometry of channel		
	Verges in cutting	Verges in Embankment		Design Flow Width (mm)	Overall Depth (mm)	Side slopes
(3) East Midlands CPD						
3.1	B3	B12	SF	600 - 800	120 - 160	1:1 1:4
3.2	--	B12	SF	500*	58*	1:2.67 1:6
3.3	B3	B12	SF	380	40*	1:1 1:8.714
3.4	--	B12	SF	460	80	1:1 1:5
(4) Northern CPD			No reply			
(5) North West CPD						
5.1	B3	B12	SF	750	125	1:1 1:5
5.2	B3	B12	SF	800	120	1:1 1:5
5.3	B3	B12	SF	600 440	75 55	1:4 1:4

Table 2 Continued

Scheme	Pavement Edge Detail		Method of Construction	Geometry of channel		
	Verges in cutting	Verges in Embankment		Design Flow Width (mm)	Overall Depth (mm)	Side slopes
(6) South East CPD						
6.1	B3	B12	SF	780*	120	1:1.5 1:5
6.2	Extruded kerb	B9	--	--	--	-- --
6.3	B3	B12	SF	500	150	1:1 1:2.3
6.4 a)	B3	--	SF	900	150*	1:1 1:5
6.4 b)	B3	B12	SF	850	150*	1:1 1:5
6.5	B3	B3	SF	875	150	1:1 1:5
6.6	--	--	--	--	--	-- --
6.7	B3	--	SF	1000	125	1:4 1:4
6.8	B3	B12	SF	900	150	1:1 1:5
(7) South West CPD						
7.1	B3	B12	SF	875	159	1:1 1:4.5
7.2	B3	B12	SF	1000	100	1:5 1:5
(8) West Midlands CPD						
8.1	ND	ND	ND	ND	ND	ND

Table 2 Continued

Scheme	Pavement Edge Detail		Method of Construction	Geometry of channel		
	Verges in cutting	Verges in Embankment		Design Flow Width (mm)	Overall Depth (mm)	Side slopes
(9) Yorkshire and Humberside CPD						
9.1	B3	--	PC	600	100	1:1 1:5
9.2	B3	B12	--	625	50	1:5 1:5
9.3	B3	B12	SF	975	97.5	1:5 1:5
(10) Northern Ireland						
10.1	--	B100SR	SF	735	150*	1:4 1:1
(11) Scotland						
11.1	O	B12	SF	1375 1300	138 130	1:5 1:5
(12) Wales						
12.1	O	O	PC	100, 800, 600	125, 100, 75	1:4 1:4
12.2	B4	--	PC	150	50	DISHED --
12.3	--	B12	SF	875	150	1:1 1:5
12.4	B3, T3A	B12, T23A	SF	725	150	1:1 1:4
12.5	B14	B14	--	900	78	1:5 1:5
12.6	--	--	--	--	--	-- --

* Amended value

Table 3 Summary of results of questionnaire Surface channels in central reserve

Scheme	Payment Edge Detail	Method of Construction	Geometry of Channel		
			Design Flow Width (mm)	Overall Depth (mm)	Side Slopes
(1) London CPD			No schemes		
(2) Eastern NMD					
2.1	B7 & **	SF	1120	140	1:4 1:4
2.2	B7 & **	SF	1125	140	1:4 1:4
2.3	B7 & **	SF	1125	140	1:4 1:4
2.4	B7 & **	SF	1120	140	1:4 1:4
2.5	0	SF	2470 1200	175 150	1:1 1:4 1:4 1:4
2.6	B7	SF	1200	150	1:4 1:4
2.7	O	SF, IS	2500	150	1:8 1:6
2.8	SPECIAL	IS	900	22.5	1:20 1:20
2.9	B6, B7	--	1150	115*	1:5 1:5
2.10	B7	SF	1125	125	1:4 1:5
(3) East Midlands CPD					
3.1	--	--	--	--	-- --
3.2	--	--	--	--	-- --
3.3	--	--	--	--	-- --
3.4	--	--	--	--	-- --

Table 3 Continued

Scheme	Payment Edge Detail	Method of Construction	Geometry of Channel		
			Design Flow Width (mm)	Overall Depth (mm)	Side Slopes
(4) Northern CPD			No reply		
(5) North West CPD					
5.1	--	--	--	--	-- --
5.2	B7	SF	950	120	1:4 1:4
5.3	--	--	--	--	-- --
(6) South East CPD					
6.1	--	--	--	--	-- --
6.2	SCHEME SPECIFIC	IS	700	150	1:1 1:1 Trapezoidal
6.3	B7	SF	500 500	125* 150*	1:1 1:2.9 1:1 1:2.3
6.4 a)	B7	SF	1200	120	1:5 1:5
6.4 b)	--	--	--	--	-- --
6.5	B7	SF	1150	120	1:5 1:5
6.6	B6, B7, B8	SF	1400	140*	1:5 1:5
6.7	B7	SF	1000	125	1:4 1:4
6.8	B7	SF	1000	125	1:4 1:4

Table 3 Continued

Scheme	Payment Edge Detail	Method of Construction	Geometry of Channel		
			Design Flow Width (mm)	Overall Depth (mm)	Side Slopes
(7) South West CPD					
7.1	B7	EX	1125	125	1:4.5 1:4.5
7.2	B7	SF	1300	120	1:5.5 1:5.5
(8) West Midlands CPD					
8.1	ND	ND	ND	ND	ND ND
(9) Yorkshire and Humberside CPD					
9.1	--	--	--	--	-- --
9.2	--	--	--	--	-- --
9.3	B7	SF	975	97.5	1:5 1:5
(10) Northern Ireland					
10.1	--	--	--	--	-- --
(11) Scotland					
11.1	--	--	--	--	-- --

Table 3 Continued

Scheme	Payment Edge Detail	Method of Construction	Geometry of Channel		
			Design Flow Width (mm)	Overall Depth (mm)	Side Slopes
(12) Wales					
12.1	O	PC	1000 800	125 100	1:4 1:4
12.2	B8	--	--	--	-- --
12.3	B5	--	--	--	-- --
12.4	--	--	--	--	-- --
12.5	--	--	--	--	-- --
12.6	--	--	--	--	--- --

* Amended value

** Stone filled trench over fin drain

Table 4 Summary of results of questionnaire Description of channel outfalls

Scheme	Total No of Outlets			Type of Outlet	Distance between Outlets		Outlets discharge into	Edition of HCD
	Verge A	Central Reserve	Verge B		Max (m)	Min (m)		
(1) London CPD			No schemes					
(2) Eastern NMD								
2.1	8	7	9	GV	960	100	CD	1987
2.2	10	8	2	GV	760	100	CD	1987
2.3	11	5	7	GV	850	120	CD	1987
2.4	13	6	9	GV	785	95	CD	1987
2.5	16	6	15	GC	1000	140	WC, TD, O	1987
2.6	8	14	6	GV	580 Verge 240 Carriageway	70	CD	1987
2.7	--	75	--	GC	100	40	CD	1987
2.8	110	244	109	GC	50	5	CD	1991
2.9	68	11	58	GC	1100	30	WC, CD	1987
2.10	8	9	5	GC	350	100	CD	1987
(3) East Midlands CPD								
3.1	8	--	11	GC	630	40	CD	1987
3.2	228	--	241	GC	35	15	WC	1987
3.3	82	--	122	GC	30	10	28% WC 72% CD	1987
3.4	23	--	39	GC	50	18	TD	1987

Table 4 Continued

Scheme	Total No of Outlets			Type of Outlet	Distance between Outlets		Outlets discharge into	Edition of HCD
	Verge A	Central Reserve	Verge B		Max (m)	Min (m)		
(4) Northern CPD				No reply				
(5) North West CPD								
5.1	71	--	--	GC	150	10	CD	1987
5.2	26	27	24	GC	50	50	CD	1987
5.3	22	--	14	GC	84	35	CD, O	1987
(6) South East CPD								
6.1	27	--	10	GC	400	10	7WC, 7CD, 24 SO	1991
6.2	--	30	--	GC	275	50	WC, CD, TD	1987
6.3	36	69	45	--	100	25	CD	1987
6.4 a)	5	5	4	GC	574	154	SO	1987
6.4 b)	1	--	1	GC	600	364	SO	1987
6.5	6	5	7	GC	300	90	SO	1987
6.6	--	21	--	GC	383	175	CD	1987
6.7	5	3	2	GV	140 90	60	CD	1987
6.8	15	21	8	GV	250	22	CD	1987

Table 4 Continued

Scheme	Total No of Outlets			Type of Outlet	Distance between Outlets		Outlets discharge into	Edition of HCD
	Verge A	Central Reserve	Verge B		Max (m)	Min (m)		
(7) South West CPD								
7.1	5	3	11	GC	350	80	CD	1987
7.2	118	49	136	GC	300	90	CD	1987
(8) West Midlands CPD								
8.1	ND	ND	ND	ND	ND	ND	ND	1992
(9) Yorkshire and Humberside CPD								
9.1	--	--	--	GC	90	30	CD	1987
9.2	59	--	87	GC	50	50	CD	1987
9.3	>200	>100	>200	GC	100	10	CD	1991
(10) Northern Ireland								
10.1	5	--	6	GV	240	100	CD	DOE (N1) ROADS SERVICE
(11) Scotland								
11.1	4	--	--	GC	500	20	WC, CD & CHANNEL	1991

Table 4 Continued

Scheme	Total No of Outlets			Type of Outlet	Distance between Outlets		Outlets discharge into	Edition of HCD
	Verge A	Central Reserve	Verge B		Max (m)	Min (m)		
(12) Wales								
12.1	8	8	5	GV	750	40	CD	1987
12.2	150	30	--	GV	100	20	CD	1987
12.3	--	--	--	GC, GV	150	75	CD	1987
12.4	42	--	46	GC	145	35	CD	1987
12.5	28	--	39	GC	107	35	CD, TD	1987
12.6	--	--	--	--	--	--	--	--

Table 5 Schemes visited

Scheme		Date of Visit	Ref
A21	Pembury Bypass	28/7/93	A
A20	Folkestone to Dover (Contracts 1 and 3)	28/7/93 and 26/10/93 and 27/10/93	B
M20	Maidstone J5-J8 Bypass	28/7/93	C
A23	Muddleswood to Patcham (Contract 3)	2/8/93	D
A11	Red Lodge Bypass	10/8/93	E
A11	Thetford Bypass	10/8/93	F
A47	Norwich Southern Bypass (Contracts 2 and 4)	11/8/93	G
A30	Okehampton-Launceston Improvement	13/8/93	H
A39	Wadebridge Bypass	15/8/93	I
A5	Little Brickhill Bypass	6/9/93	J
A40	M40 to B4027	30/9/93	K
A19	Easingwold Bypass	22/10/93	L
A487	Port Dinorwic Bypass	29/11/93 and 30/11/93	M

Table 6 List of meetings with resident engineers of schemes visited

A20 Folkestone to Dover (Contracts 1 and 3)		Date of meeting
Designers:	Mott MacDonald (Winchester Office) Mr N Paisley Mr C Rice	27/7/93
Resident Engineers:	Mott MacDonald (Site Office-Capel-le-ferm) Mr P Knight (RE for Contract 3) Mr I Jones (RE for Contract 1)	28/7/93
Channel Contractors:	Extrudakerb (Maltby Engineering Ltd) - part of Contract 3 Mr J Charlesworth	20/10/93
A47 Norwich Southern Bypass		
Designers:	G Maunsell & Partners (Witham Office) Mr B Bartlett	10/8/93
Resident Engineer:	G Maunsell & Partners (Site Office-Contract4) Mr M Vine	11/8/93
Channel Contractors:	SIAC Construction Ltd (Hitchin Office) Mr J Donegan	6/9/93
Maintenance Organisation:	Norfolk County Council Mr K Townly	2/8/93 (telephone conversation)
A11 Red Lodge Bypass		
Resident Engineer:	Suffolk County Council Mr A Bilby	11/8/93
A19 Easingwold Bypass		
Resident Engineer:	North Yorkshire County Council Mr R Christiansen	22/10/93

Table 7 Results of field tests at A20 Folkestone to Dover (Contract 1)

Outlet location	Channel slope	Test No	h (m)	V (m/s)	Q (m^3/s)	Manning's n
C R W	0.0016	1	0.102	0.256	0.0152	*
C R W	0.0016 (1:622)	2	0.100	0.277	0.0159	*
C R E	0.0019 (1:526)	3	0.086	0.628	0.0254	0.008
V	0.0019	4	0.092	0.818	0.0239	0.007
V	0.0019 (1:526)	5	0.101	0.789	0.0275	0.007

C R W and C R E - Central reserve (West and East)
Symmetrical 1:5 channel.

V - Verge
Asymmetrical channel with side slopes 1:1 and 1:5

* - Uncertain

Table 8 Results of field tests at A487 Port Dinorwic Bypass

Outlet type	Channel slope	Test No	h (m)	V (m/s)	Q measured (m^3/s)	Q estimated (m^3/s)	Manning's n
Terminal	0.053	1	0.055	(2.30)	--	0.0378	0.009
	0.053	2	0.062	(2.50)	--	0.0526	0.009
	0.053	3	0.040	0.790	0.0069	--	0.021
	0.053	4	0.026	0.727	0.0026	--	0.017
	0.053 (1:19)	5	0.035	0.556	0.0037	--	0.027
Intermediate	0.0606	6	0.027	0.731	0.0029	--	0.019
	0.0606	7	0.035	0.352*	0.0024	--	0.046
	0.0606 (1:16.5)	8	0.059	(2.59)	--	0.0494	0.009
Intermediate	0.0049	9	0.037	0.599	0.0044	--	0.008
	0.0049	10	0.055	0.677	0.0111	--	0.009
	0.0049	11a	0.057	0.595	0.0105	--	0.011
	0.0049	11b	0.053	0.784	0.0122	--	0.008
	0.0049	11c	0.053	0.713	0.0108	--	0.009
	0.0049 (1:204)	12	0.058	(0.729)	--	0.0136	0.009

Note: The values of velocity in brackets were obtained from estimated flows.

* Doubtful data

Table 9 Bypass flow at A487 Port Dinorwic Bypass

Test	Q (m^3/s)	h_p (m)	Q_p (m^3/s)	η (%)
7	0.0024	0.0050	0.000076	96.8
8	0.0494	0.0365	0.0152	69.2

Table 10 Triangular channel In-line outlet, intermediate

A) Channel - full

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
1 pair of gratings			
0.0597	0.02123	0.094	1:60
0.0516	0.0133	0.096	1:100
0.0412	0.00476	0.094	1:250
0.0288	0.00230	0.094	1:500
0.0323	0.00197	0.100	1:2000
2 pairs of gratings			
0.0655	0.01122	0.094	1:50
0.0592	0.00658	0.094	1:60
0.0572	0.00257	0.098	1:100
0.0418	0.00376	0.093	1:200

B) Surcharged

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
1 pair of gratings			
0.0830	0.02598	0.116	1:100
0.0615	0.01034	0.103	1:250
0.0515	0.00616	0.121	1:500
0.0489	0.00456	0.119	1:2000
2 pairs of gratings			
0.0902	0.02171	0.111	1:50
0.0901	0.01643	0.112	1:60
0.0831	0.00854	0.116	1:100
0.0722	0.00196	0.123	1:200

Table 11 Triangular channel In-line outlet, terminal

A) Channel - full

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
1 pair of gratings			
0.0597	overtopping	0.093	1:60
0.0528	overtopping	0.096	1:100
0.0410	0	0.096	1:250
0.0288	0	0.094	1:500
0.0288	0	0.097	1:2000
2 pairs of gratings			
0.0902	overtopping	0.111	1:50

B) Surcharged

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
1 pair of gratings			
0.0830	overtopping	0.115	1:100
0.0650	overtopping	0.121	1:250
0.0515	0	0.115	1:500
0.0489	0	0.124	1:2000
2 pairs of gratings			
0.0902	overtopping	0.111	1:50

Table 12 Effect of different bar patterns

Triangular channel
In-line intermediate outlet - 1 pair of gratings

Bar pattern	Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
Diagonal	0.0597	0.02123	0.094	1:60
	0.0516	0.0133	0.096	1:100
	0.0412	0.00476	0.094	1:250
Transverse	0.0572	0.0236	0.093	1:60
	0.0516	0.01576	0.095	1:100
	0.0414	0.00186	0.094	1:250
Longitudinal	0.0572	0.0103	0.093	1:60
	0.0516	0.00531	0.095	1:100
	0.0414	0.00186	0.094	1:250
Open area	0.0597	0.000123	0.094	1:60
	0.0516	0.000091	0.095	1:100
	0.0412	0.00742	0.094	1:250

Table 13 Triangular Channel Off-line outlet, intermediate

A) Channel - full

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
1 grating			
0.0538	0.02653	0.098	1:80
0.0492	0.01941	0.097	1:100
0.0413	0.00682	0.106	1:250
0.0357	0.00177	0.105	1:500
2 gratings			
0.0540	0.01584	0.097	1:60
0.0500	0.01142	0.097	1:70
0.0502	0.00982	0.098	1:80
3 gratings			
0.0627	0.00748	0.101	1:50
0.0540	0.00443	0.097	1:60
0.0560	0.00270	0.103	1:70
0.0467	0	0.098	1:100
0.0463	0	0.109	1:150
0.0352	0	0.111	1:500

B) Surcharged

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
3 gratings			
0.0873	0.02144	0.121	1:50
0.0755	0.00541	0.126	1:100
0.0706	0.00152	0.134	1:150
0.0460	0	0.123	1:500

Table 14 Triangular channel Off-line outlet, intermediate Ramps between gratings

A) Channel - full

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
3 gratings and 2 ramps			
0.0493	--	0.094	1:100
0.0540	negligible	0.099	1:80
0.0522	0.00082	0.098	1:60
0.0585	0.00464	0.099	1:50
2 gratings and 1 ramp			
0.0492	0.00582	0.096	1:100
0.0525	0.00158	0.096	1:70
0.0550	0.01346	0.096	1:60

B) Surcharged

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
3 gratings and 2 ramps			
0.0738	0.00170	0.118	1:100
0.0800	0.00636	0.121	1:80

Table 15 Trapezoidal channel In-line outlet, intermediate Model values

A) Channel - full

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
2 gratings			
0.0515	0.0152	0.096	1:250
0.0463	0.0096	0.093	1:400
0.0423	0.0063	0.105	1:667
3 gratings			
0.0703	0.0231	0.097	1:100
0.0603	0.0129	0.099	1:200
0.0510	0.0067	0.098	1:250
0.0470	0.0024	0.094	1:400

B) Surcharged

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
3 gratings			
0.0740	0.0175	0.122	1:300
0.0649	0.0127	0.120	1:400
0.0598	0.0111	0.120	1:667

Table 16 Trapezoidal channel In-line outlet, terminal Model values

A) Channel - full

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
3 gratings			
0.0649	0.0095	0.096	1:100
0.0515	negligible	0.099	1:250
0.0418	--	0.104	1:667

B) Surcharged

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
3 gratings			
0.0598	negligible	0.123	1:667

Table 17 Trapezoidal channel Off-line outlet, intermediate Model values

A) Channel - full

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
1 grating			
0.0810	0.0491	0.103	1:150
0.0723	0.0393	0.103	1:280
0.0652	0.0318	0.102	1:300
0.0513	0.0158	0.107	1:667
0.0524	0.0164	0.110	1:711
2 gratings			
0.1161	0.0615	0.113	1:60
0.0849	0.0254	0.099	1:100
0.0797	0.0175	0.101	1:150
0.0747	0.0114	0.104	1:200
0.0663	0.0062	0.103	1:300
0.0568	0.0031	0.095	1:400
0.0513	0.0017	0.10999	1:667
3 gratings			
0.1177	0.0248	0.111	1:60
0.0884	0.0101	0.100	1:100
0.0731	0.0015	0.102	1:200
0.0633	--	0.099	1:300
0.0584	--	0.101	1:400

Table 17 Continued

B) Surcharged

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
1 grating			
0.0809	0.0372	0.136	1:667
0.0821	0.0372	0.137	1:711
2 gratings			
0.1395	0.0651	0.132	1:100
0.1006	0.0287	0.128	1:200
0.0947	0.0224	0.132	1:300
0.0857	0.0152	0.127	1:400
0.0782	0.0102	0.134	1:667
3 gratings			
0.1679	0.0455	0.134	1:60
0.1444	0.0339	0.135	1:100
0.1046	0.0102	0.127	1:200
0.0880	0.0025	0.129	1:400
0.0903	0.0004	0.134	1:667

Table 18 Trapezoidal channel Off-line outlet, terminal Model values

A) Channel - full

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
2 gratings			
0.0803	0.0138	0.109	1:200
0.0694	0.0021	0.109	1:300
0.0530	--	0.111	1:667
3 gratings			
0.0980	0.0112	0.111	1:100
0.0781	negligible	0.111	1:200

B) Surcharged

Q (m^3/s)	Q_p (m^3/s)	h (m)	Slope
2 gratings			
0.1138	0.0300	0.136	1:200
0.0921	0.0182	0.135	1:300
0.0757	negligible	0.136	1:667
3 gratings			
0.1494	0.0246	0.136	1:100
0.1110	0.0050	0.136	1:200
0.0988	negligible	0.136	1:300

Table 19 Weir outlet Model values

Q (m^3/s)	h (m)	y_a (m)	Slope	y_b (m)	Observations
A) Channel - full, 1 grating					
0.0645	0.104	0.0210	1:250	--	
0.0908	0.108	0.037	1:100	--	
0.1160	0.110	0.042	1:60	0.103	
0.1267	0.111	0.042	1:51	0.110	
0.1200	0.108	0.046	1:50	0.108	
0.1330	0.110	0.052	1:43	0.112	Capacity of outlet exceeded
B) Channel 83% full					
0.0530	0.091	0.067	1:250	--	
0.0646	0.090	0.57	1:100	--	
0.0835	0.091	0.047	1:59	0.110	Capacity of outlet reached
C) Channel 68% full					
0.0481	0.075	0.057	1:100	--	
0.0582	0.075	0.062	1:50	0.103	
0.0631	0.076	0.062	1:43	0.110	Capacity of outlet reached

Table 20 Analysis of field tests according to new Advice Note

Test	Slope	Type of outlet	Measured by-pass flow	F_o	Recommended outlet in Advice Note
A) A20 Folkestone to Dover (Contract 1)					
1	1:622	Terminal, in-line, triple grating 450 x 1350	No	0.41	One pair or single grating on invert 450 x 450
2	1:622	Terminal, in-line, triple grating 450 x 1350	No	0.45	One pair or single grating on invert 450 x 450
3	1:622	Terminal, in-line, triple grating 450 x 1350	No	1.06	One pair or single grating on invert 450 x 450
B) A487 Port Dinorwic Bypass					
1	1:19	Terminal, off-line, 2 gratings 650 x 650	No	4.82*	Weir outlet
2	1:19	Terminal, off-line, 2 gratings 650 x 650	No	4.97*	Weir outlet
3	1:19	Terminal, off-line, 2 gratings 650 x 650	No	1.95	Weir outlet
4	1:19	Terminal, off-line, 2 gratings 650 x 650	No	2.16	Weir outlet
5	1:19	Terminal, off-line, 2 gratings 650 x 650	No	1.46	3 gratings 450 x 450
6	1:16.5	Intermediate, off-line, 1 grating 650 x 650	No	2.19	Weir outlet
7	1:16.5	Intermediate, off-line, 1 grating 650 x 650	Yes, η = 96.8%	0.95	1 grating 450 x 450 (η = 97%)
8	1:16.5	Intermediate, off-line, 1 grating 650 x 650	Yes, η = 69.2%	5.28*	Weir outlet
9	1:204	Intermediate, off-line, 1 grating 650 x 650	No	1.51	3 gratings 450 x 450
10	1:204	Intermediate, off-line, 1 grating 650 x 650	No	1.42	3 gratings 450 x 450
11a	1:204	Intermediate, off-line, 1 grating 650 x 650	No	1.22	2 gratings 450 x 450
11b	1:204	Intermediate, off-line, 1 grating 650 x 650	No	1.71	3 gratings 450 x 450
11c	1:204	Intermediate, off-line, 1 grating 650 x 650	No	1.51	3 gratings 450 x 450
12	1:204	Intermediate, off-line, 1 grating 650 x 650	No	1.52*	3 gratings 450 x 450

* Calculated from estimated flows

Figures

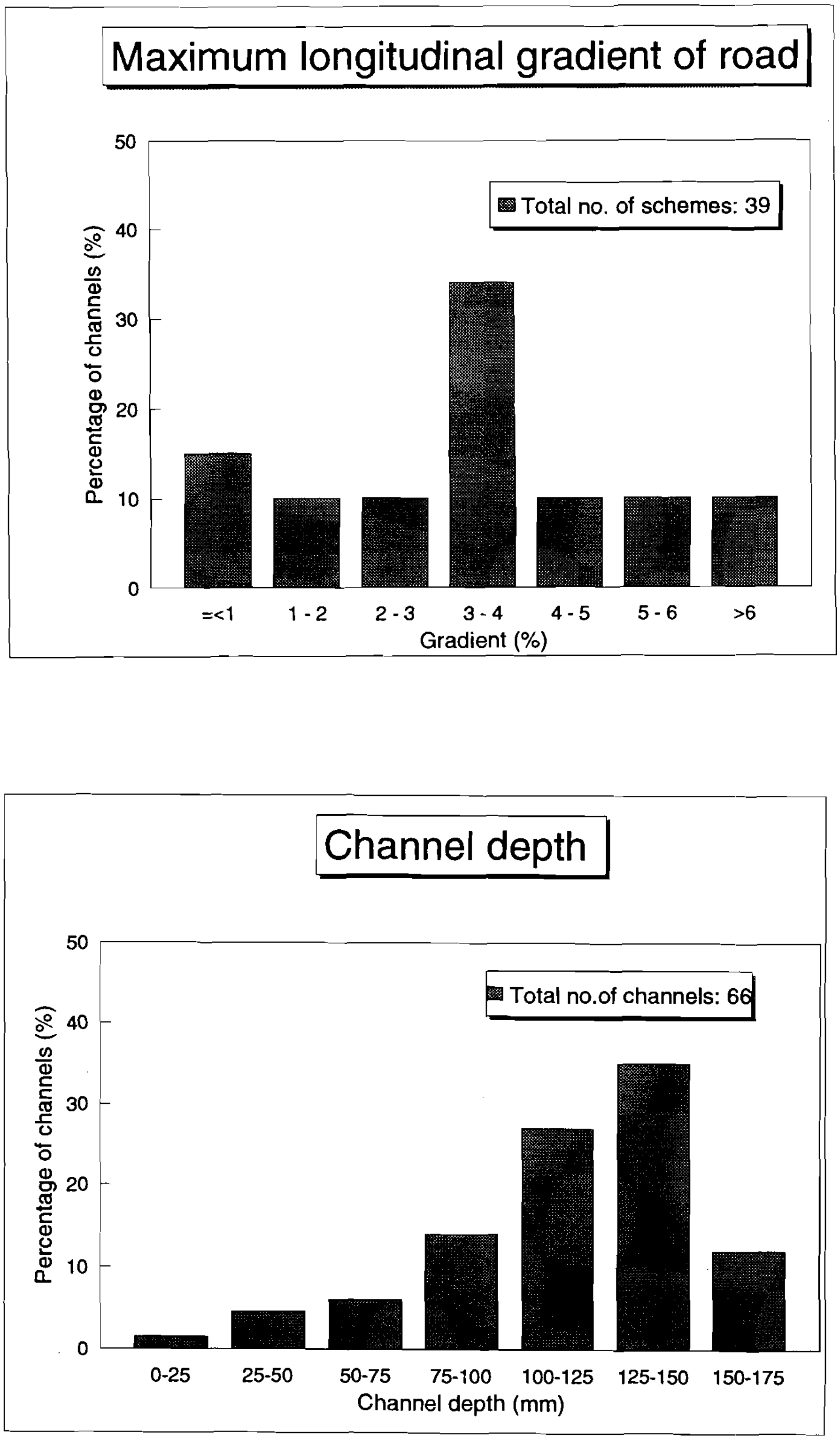
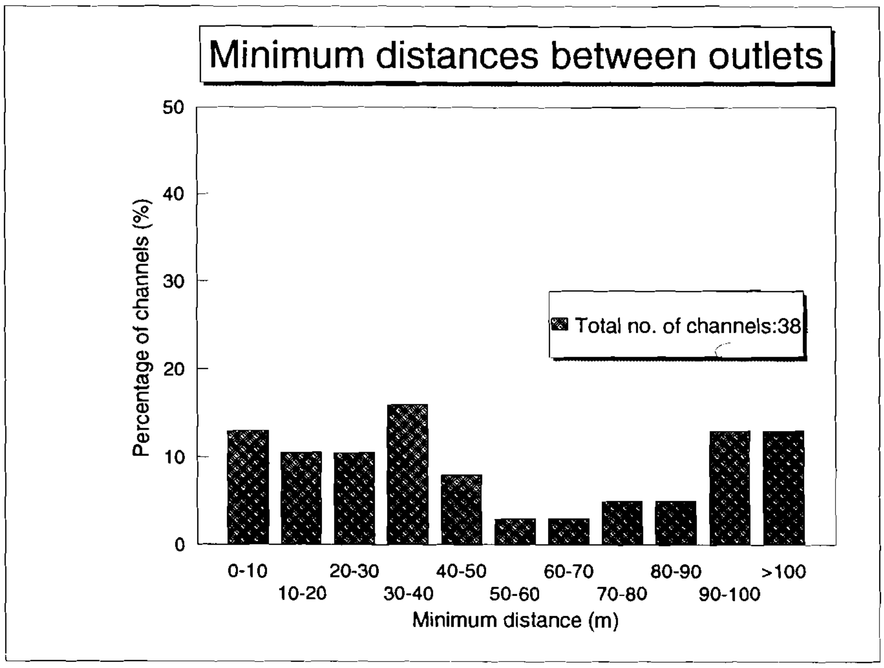


Figure 1 Results of general questionnaire
a) Maximum longitudinal gradient of road
b) Channel depth



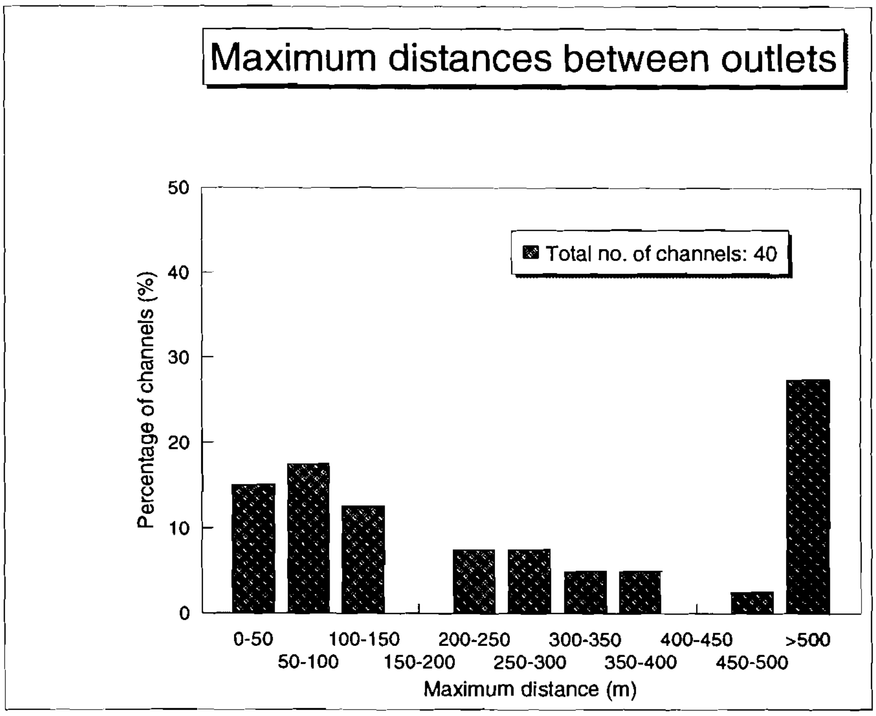


Figure 2 Results of general questionnaire
Minimum and maximum distances between outlets

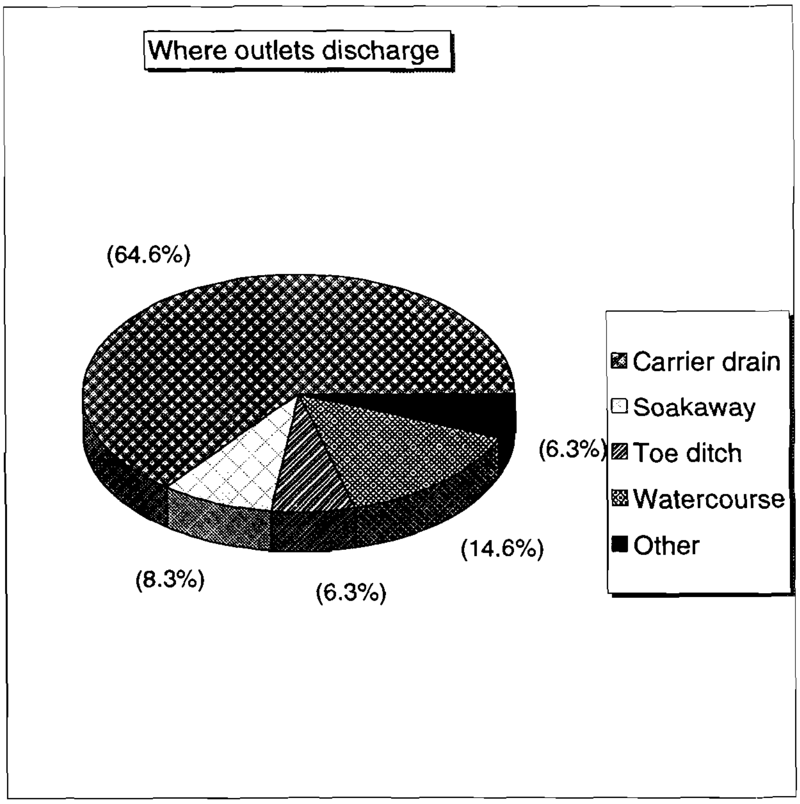


Figure 3 Results of general questionnaire Where outlets discharge into

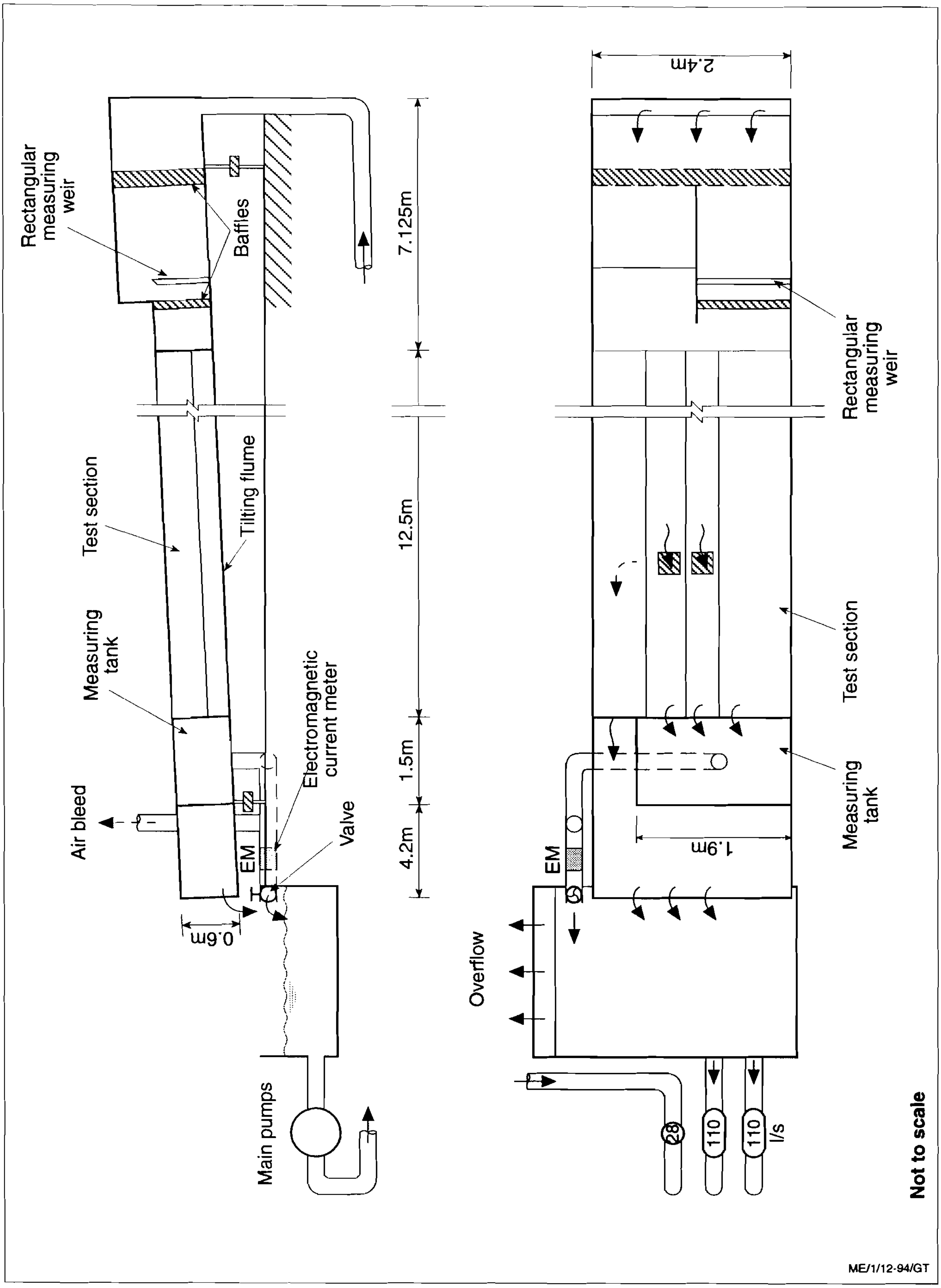


Figure 4 General layout of test rig

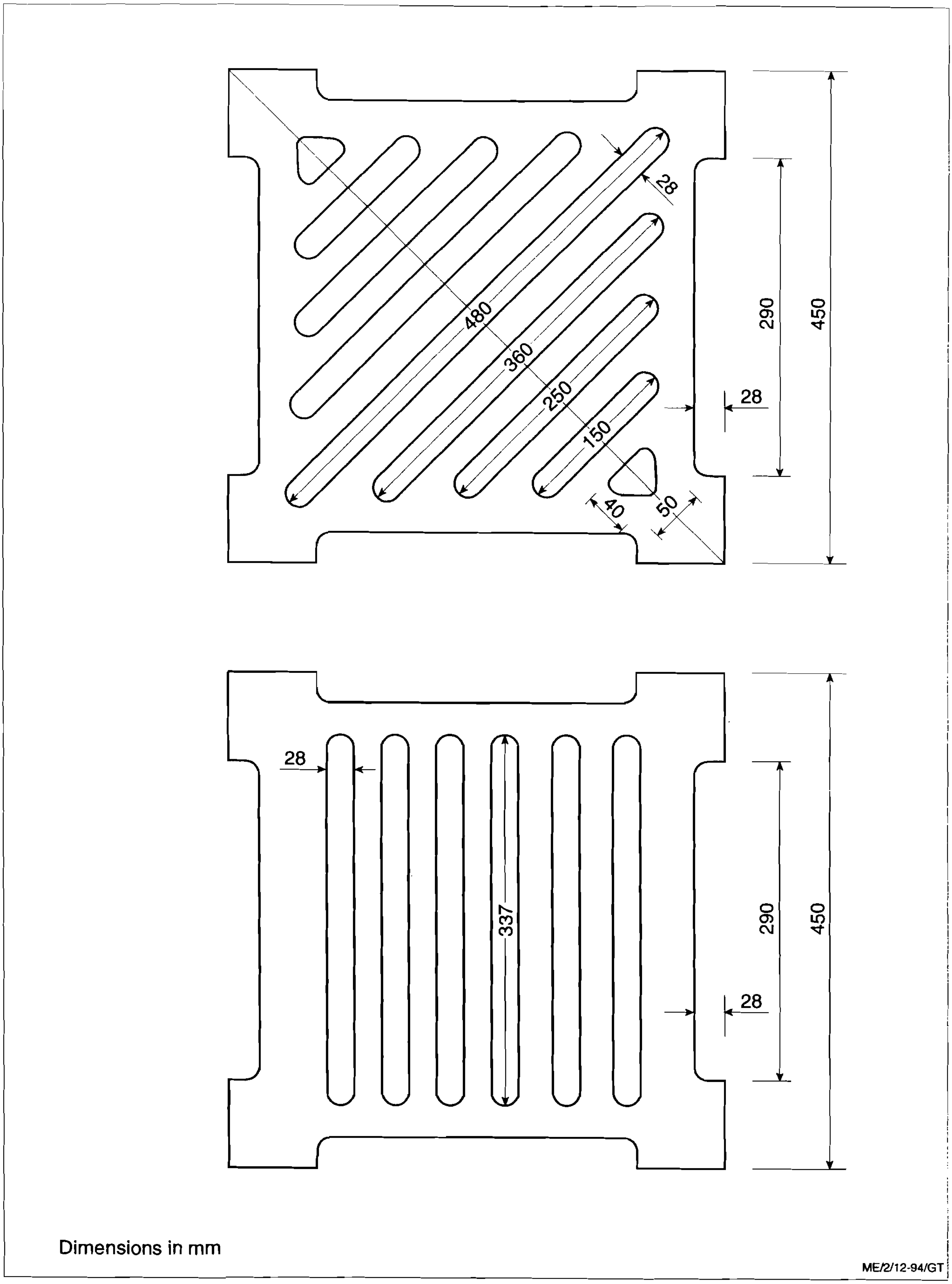


Figure 5 Bar patterns used in laboratory tests

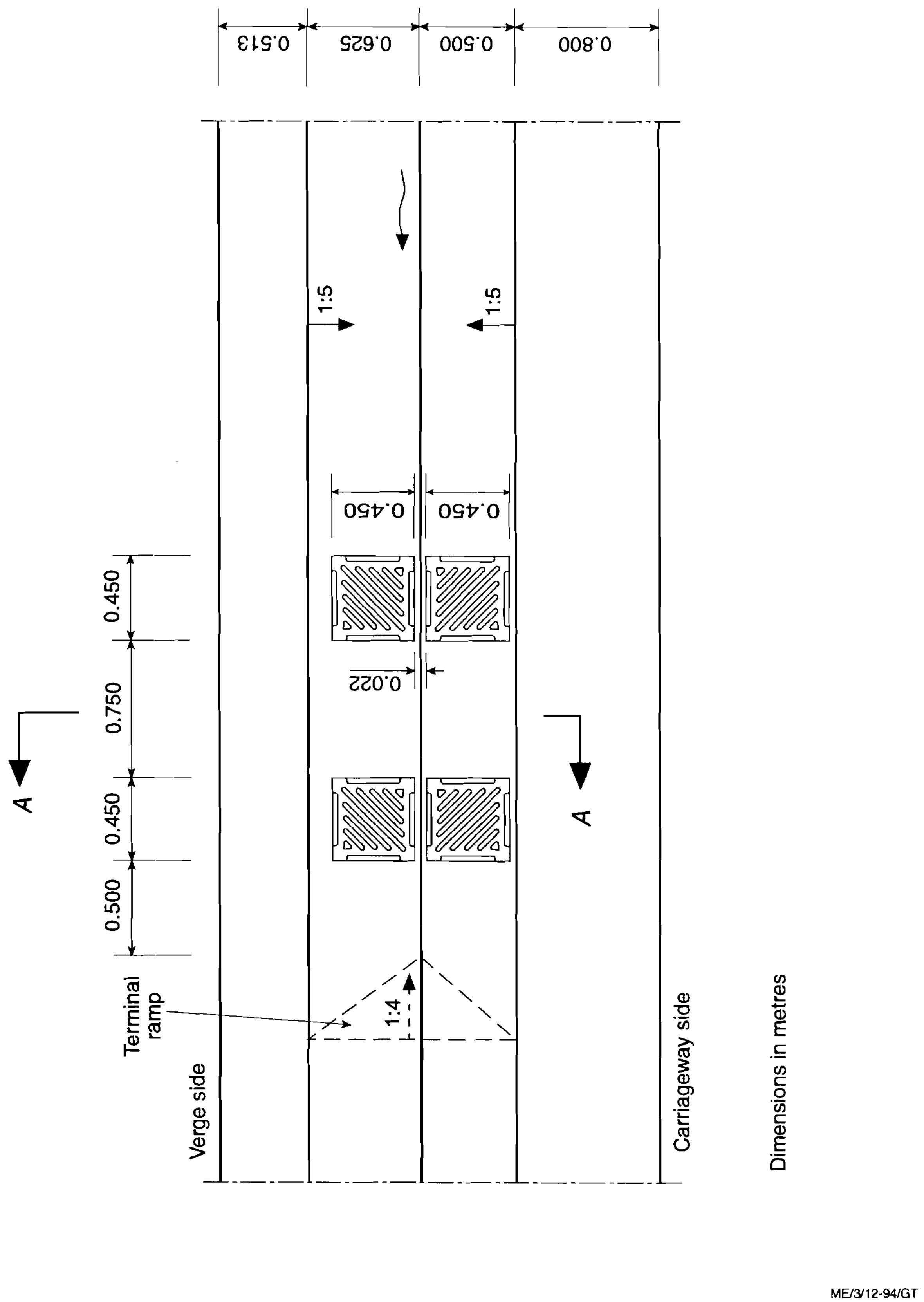


Figure 6 Triangular channel - In-line outlet

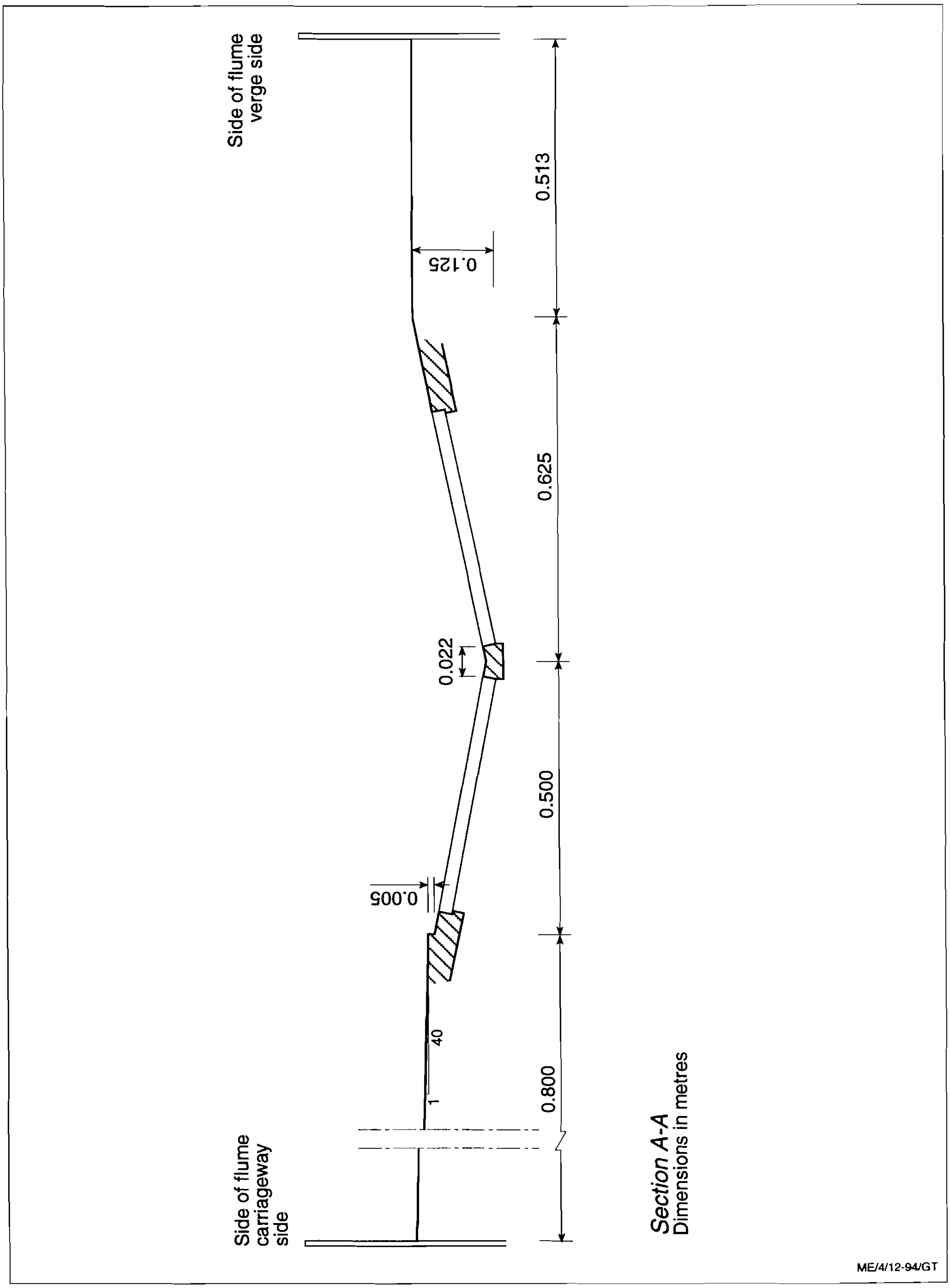


Figure 7 Triangular channel - Section A-A of in-line outlet

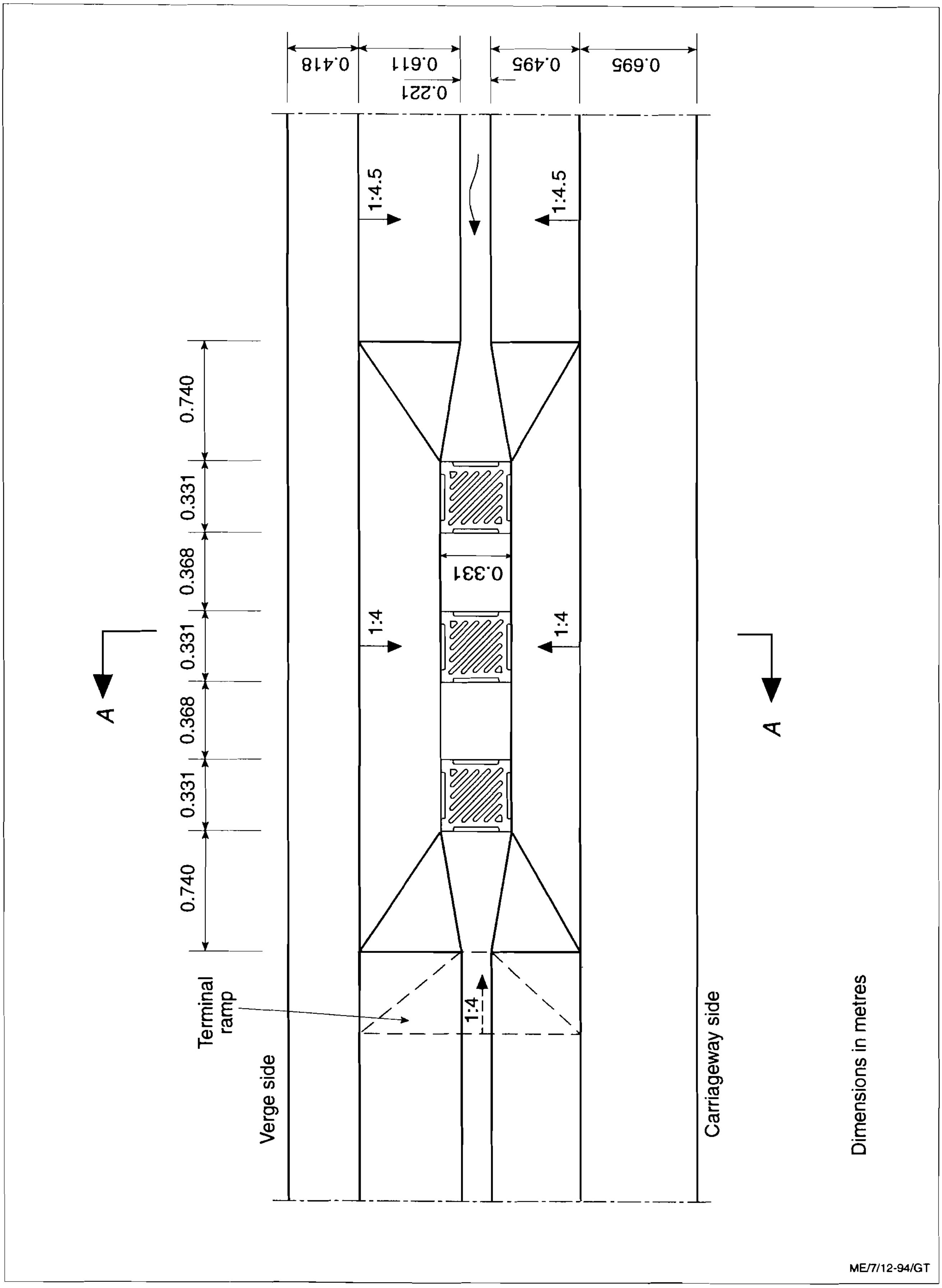


Figure 10 Trapezoidal channel 1:4.5 - In-line outlet

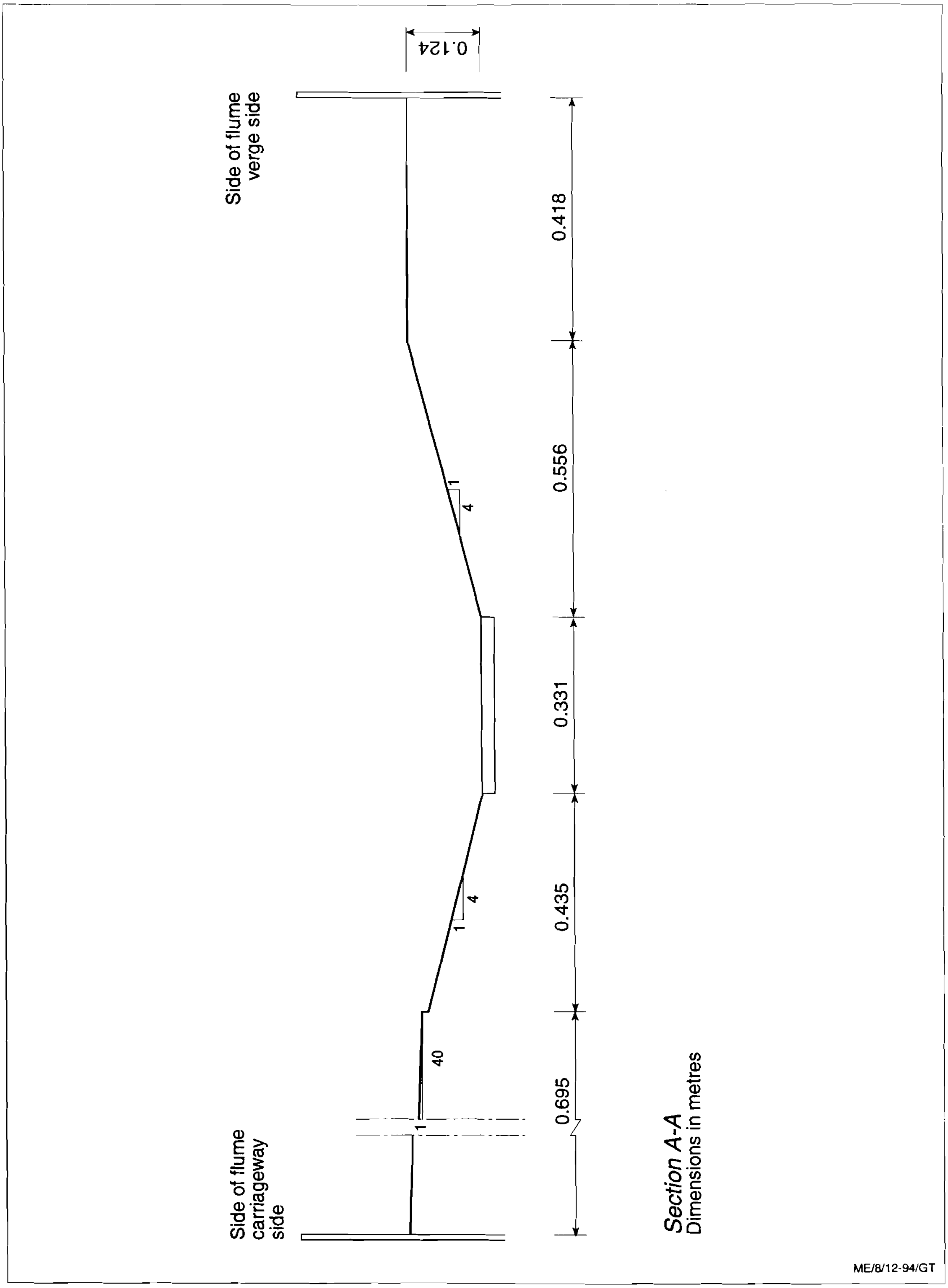


Figure 11 Trapezoidal channel 1:4.5 - Section A-A of in-line outlet

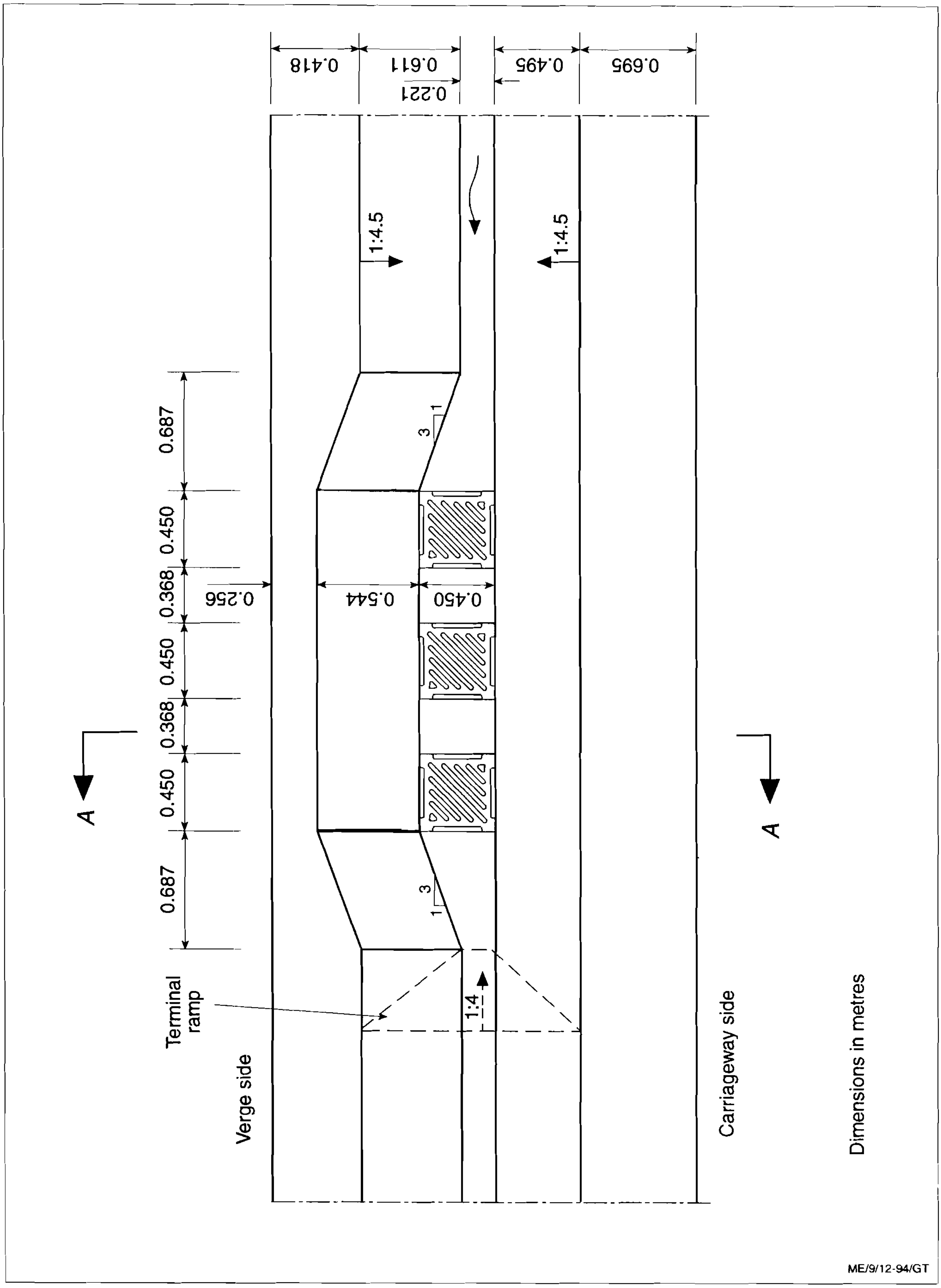


Figure 12 Trapezoidal channel 1:4.5 - Off-line outlet

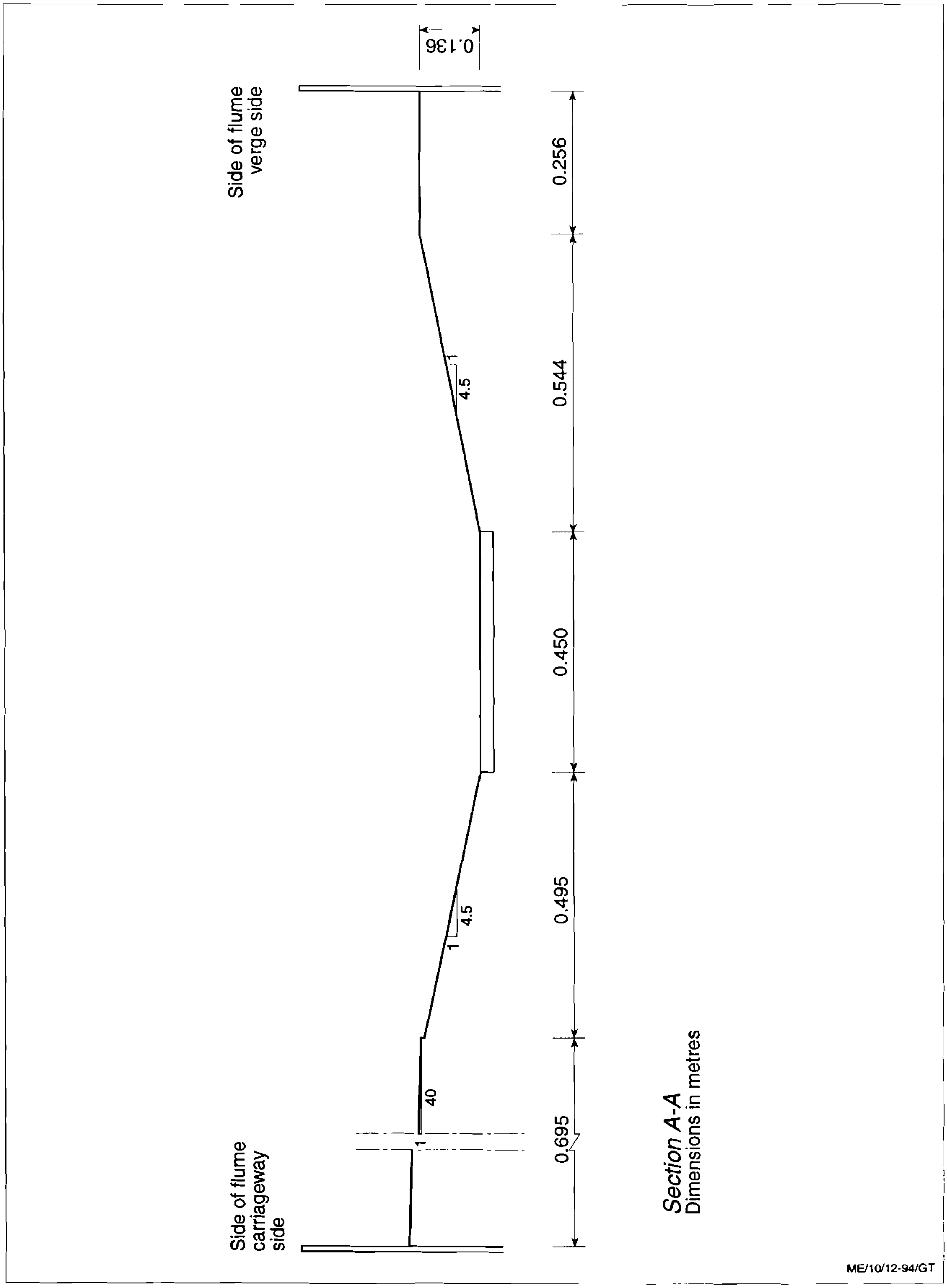


Figure 13 Trapezoidal channel 1:4.5 - Section A-A of off-line outlet

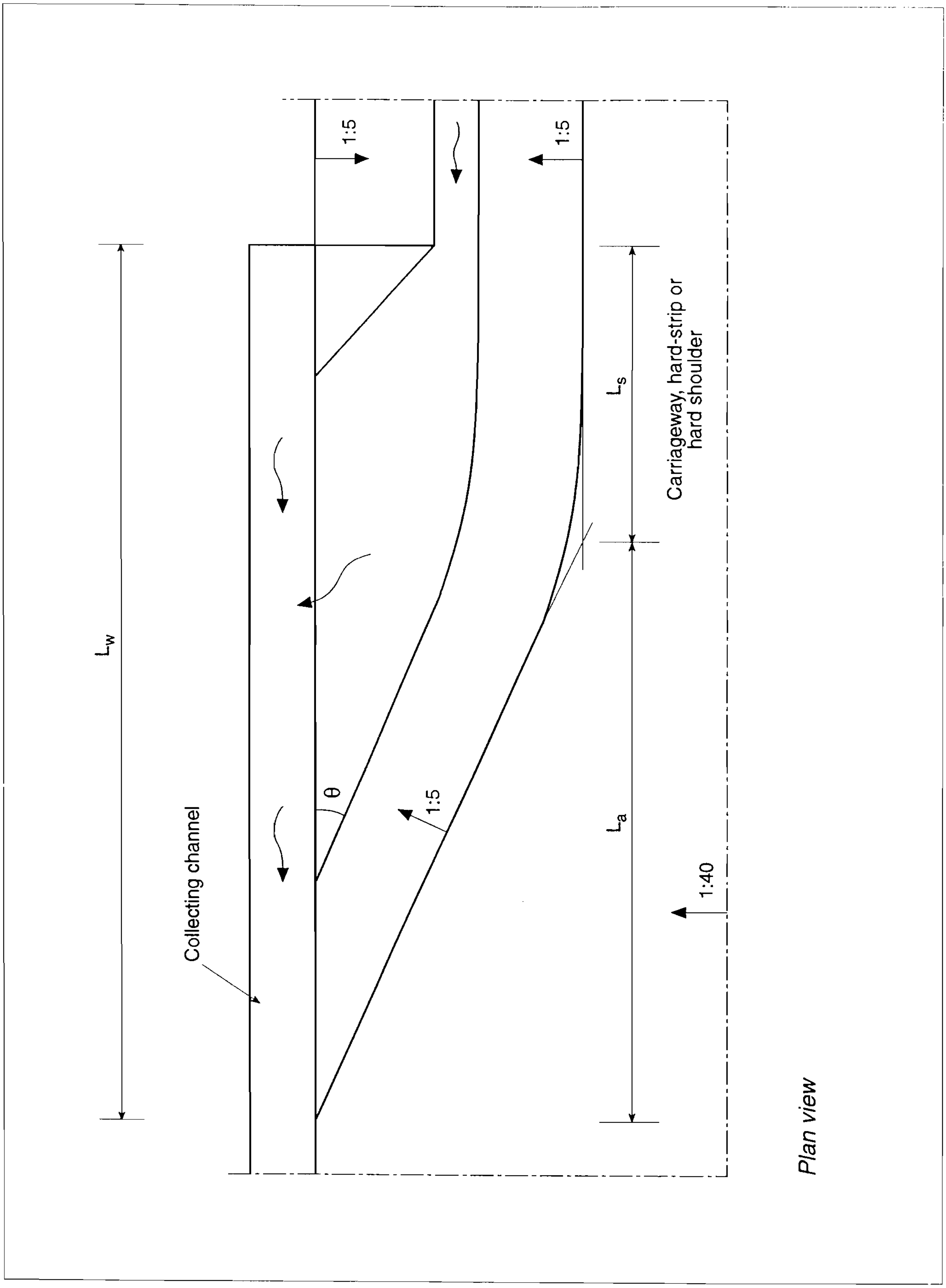


Figure 14 Weir outlet

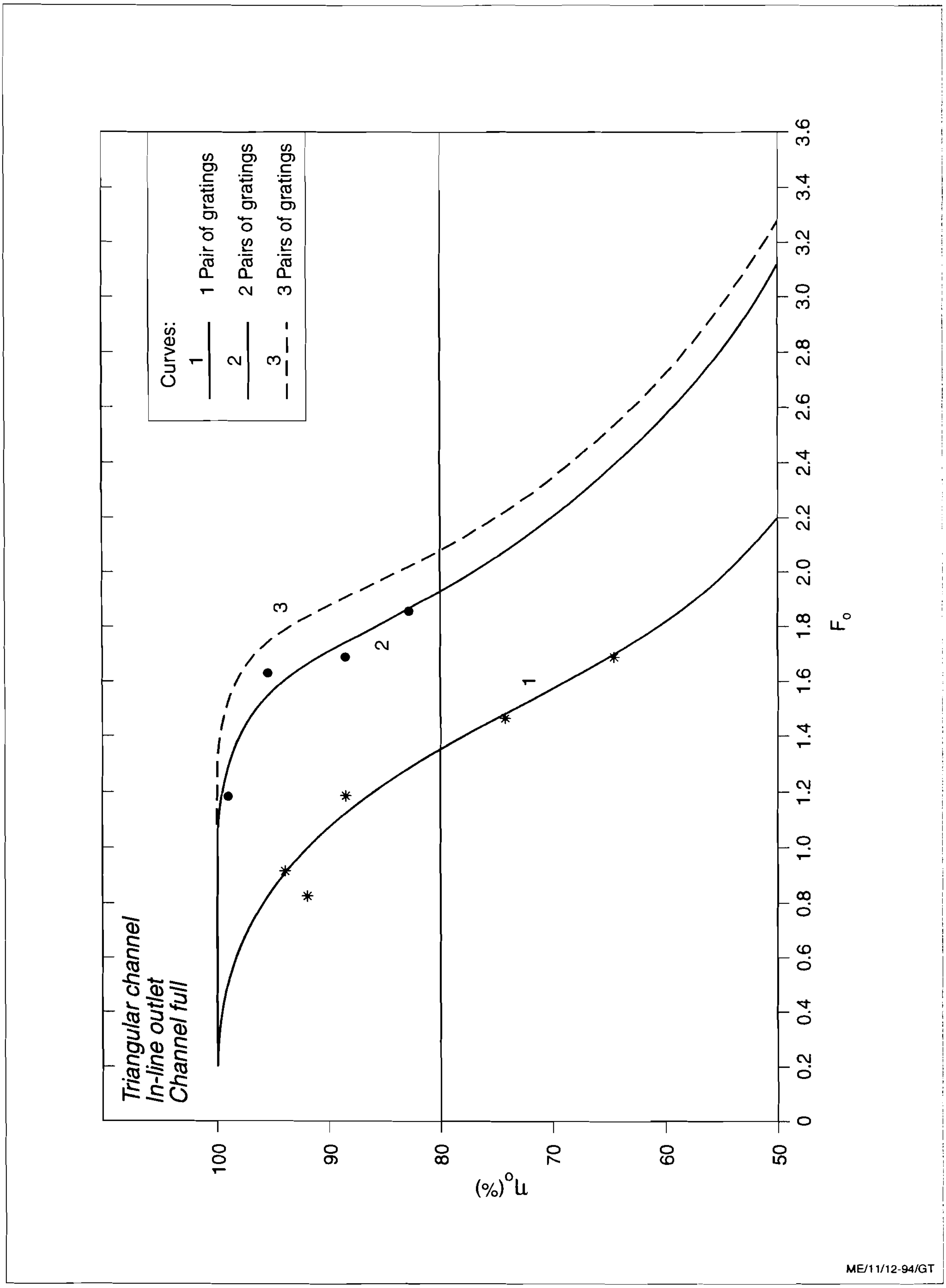


Figure 15 Design curves. Triangular channel - In-line outlet Channel full

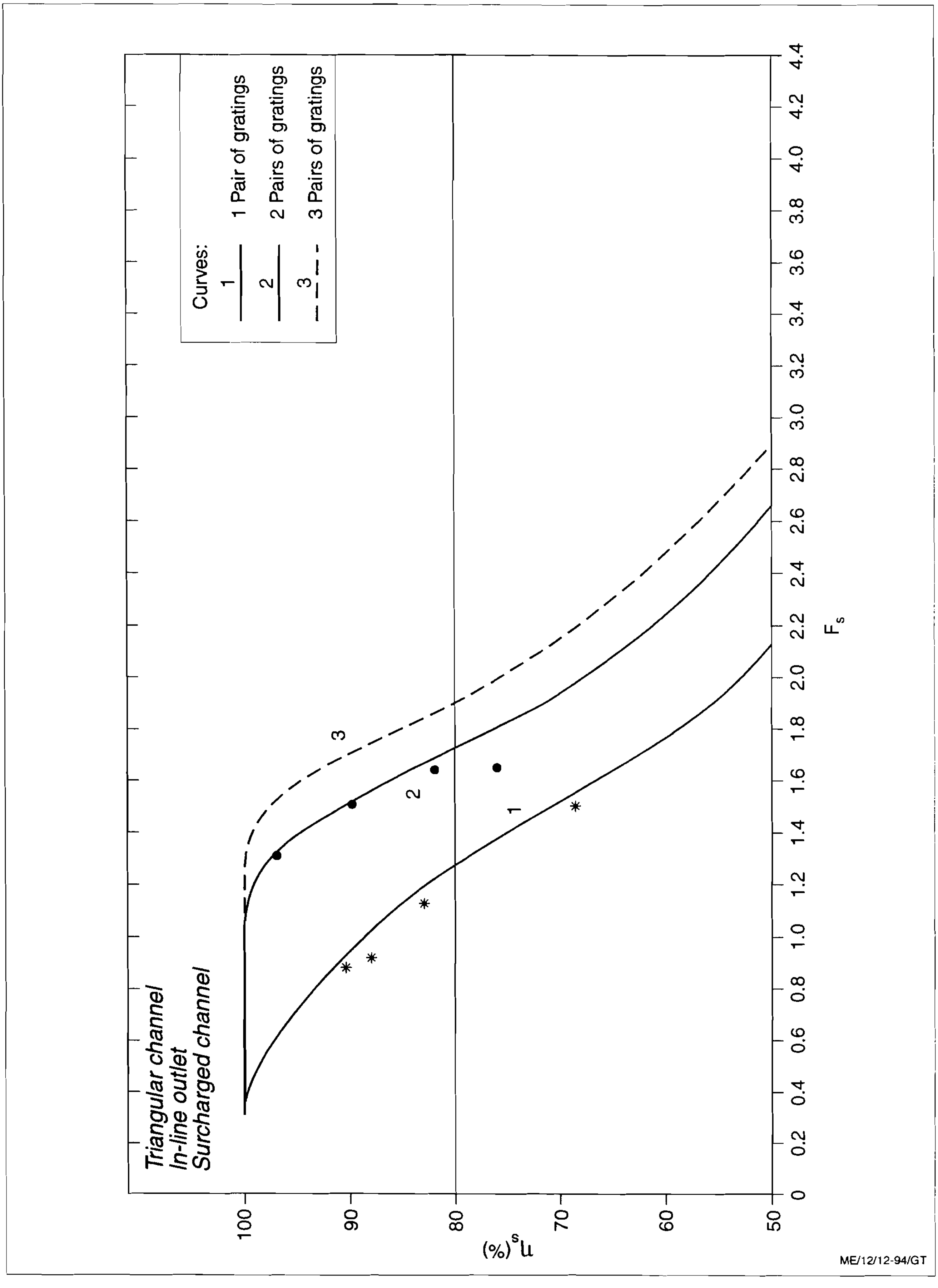


Figure 16 Design curves. Triangular channel - In-line outlet Surcharged channel

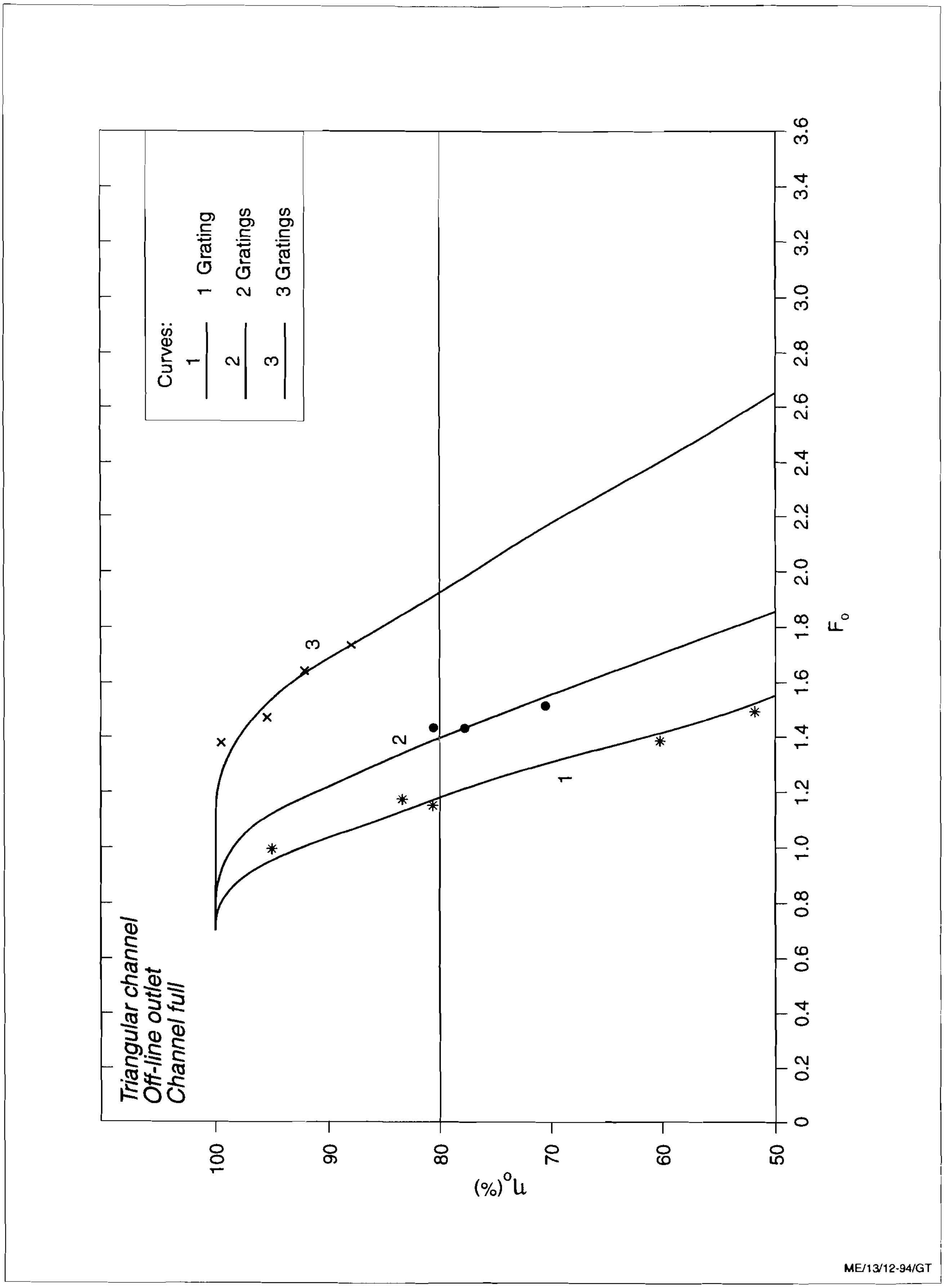


Figure 17 Design curves. Triangular channel - Off-line outlet Channel full

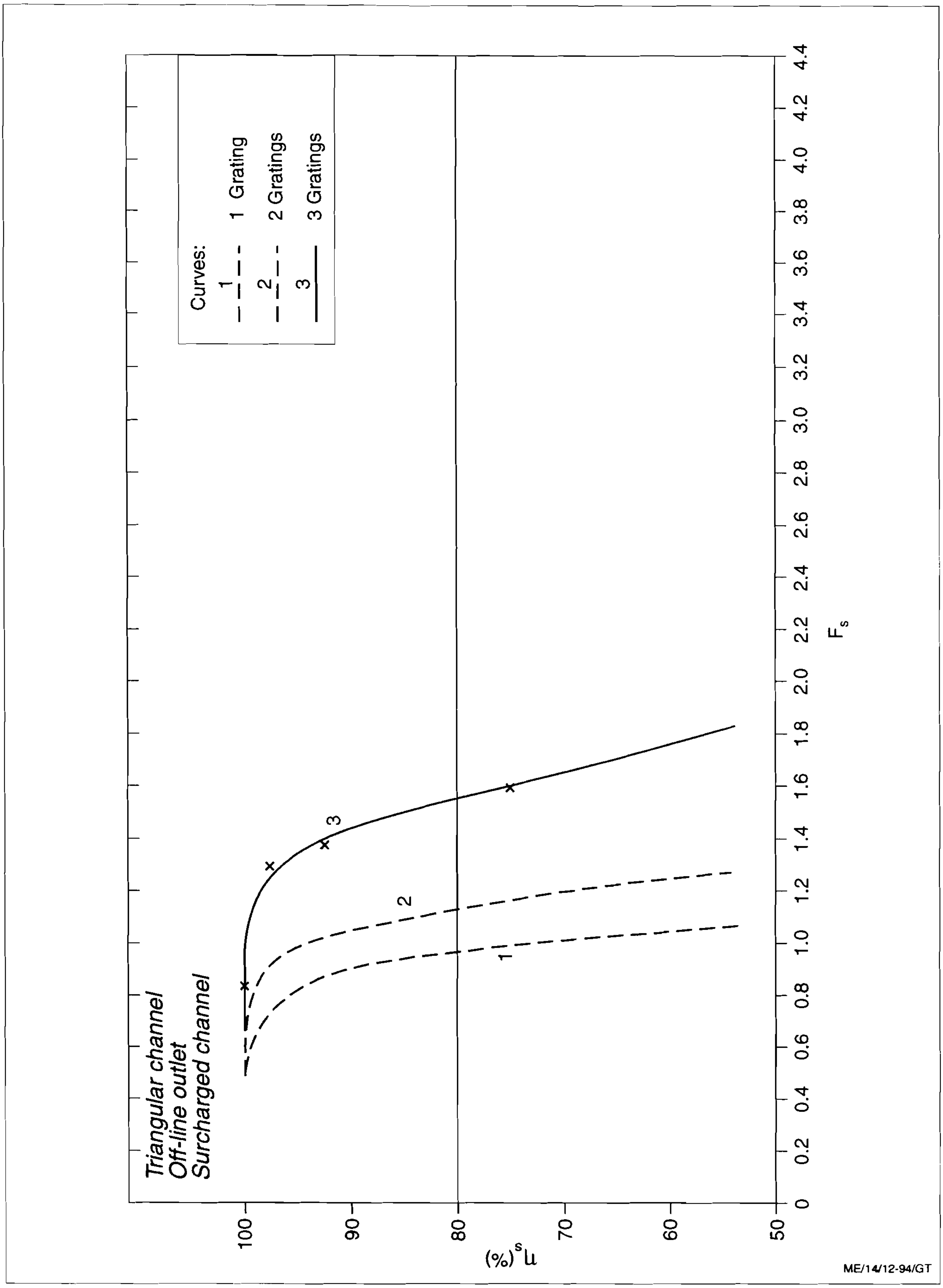
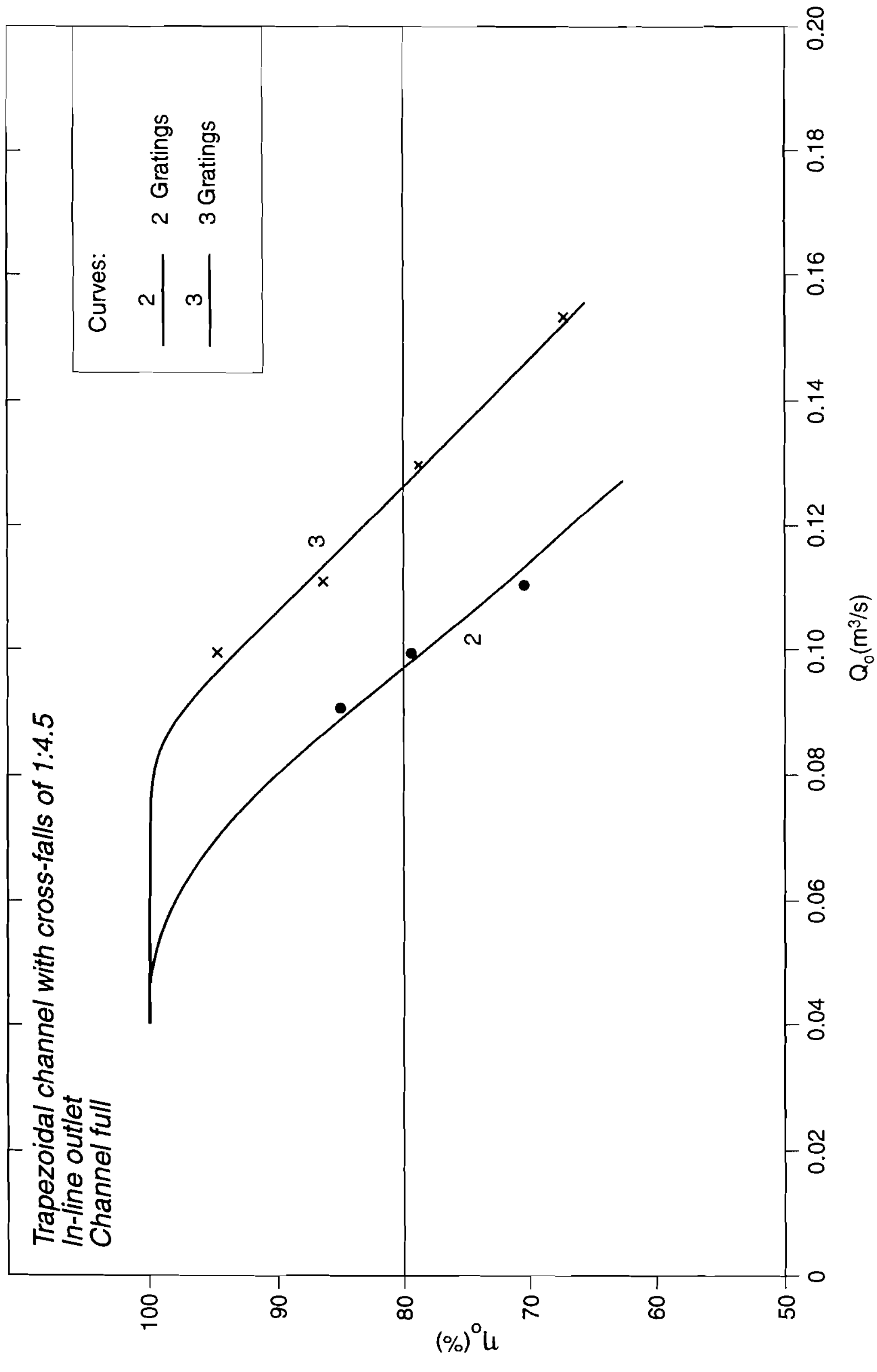


Figure 18 Design curves. Triangular channel - Off-line outlet Surcharged channel



ME/15/12-94/GT

Figure 19 Design curves. Trapezoidal channel with cross-falls of 1:4.5 In-line outlet. Channel full

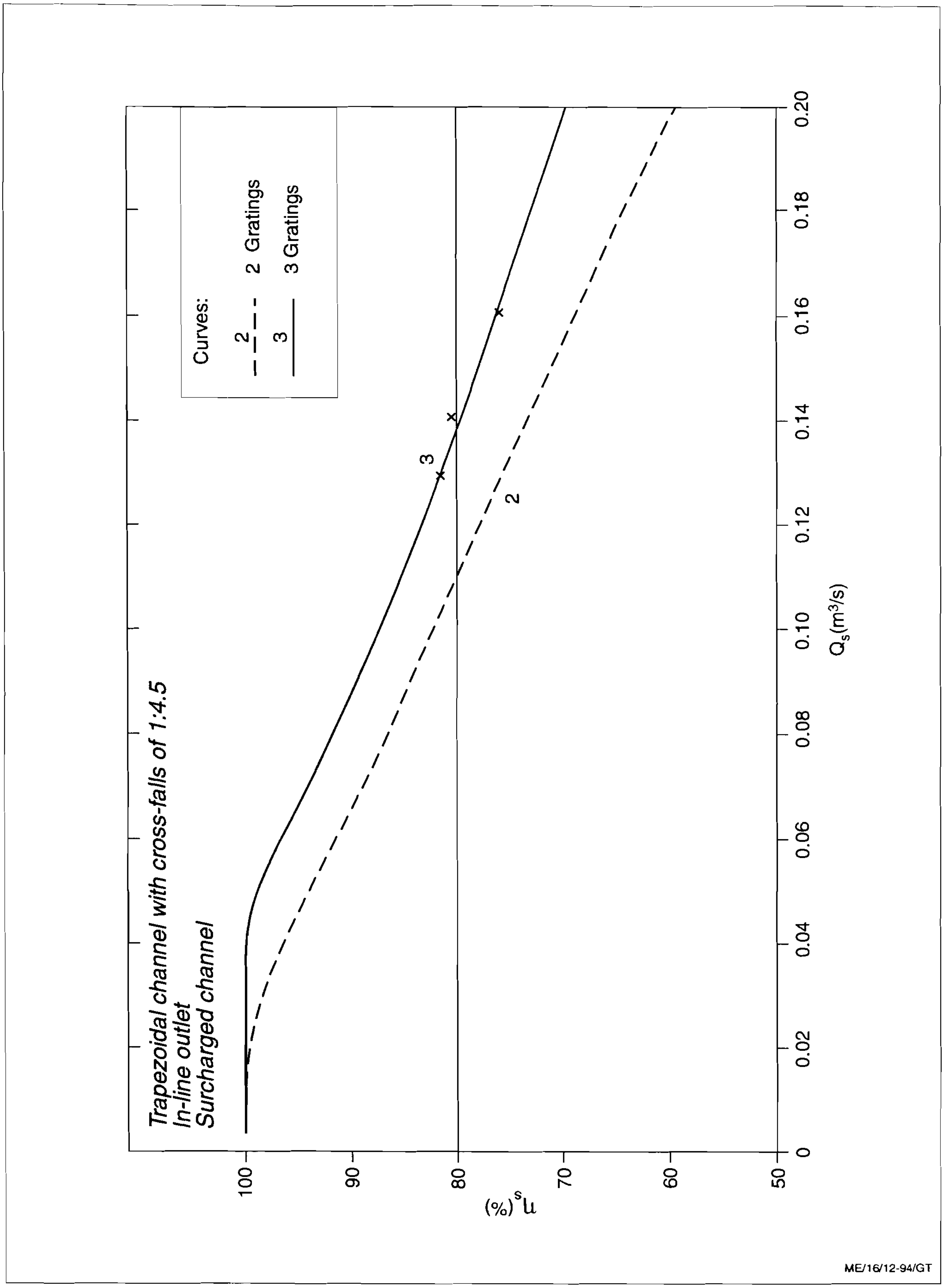


Figure 20 Design curves. Trapezoidal channel with cross-falls of 1:4.5 In-line outlet. Surcharged channel

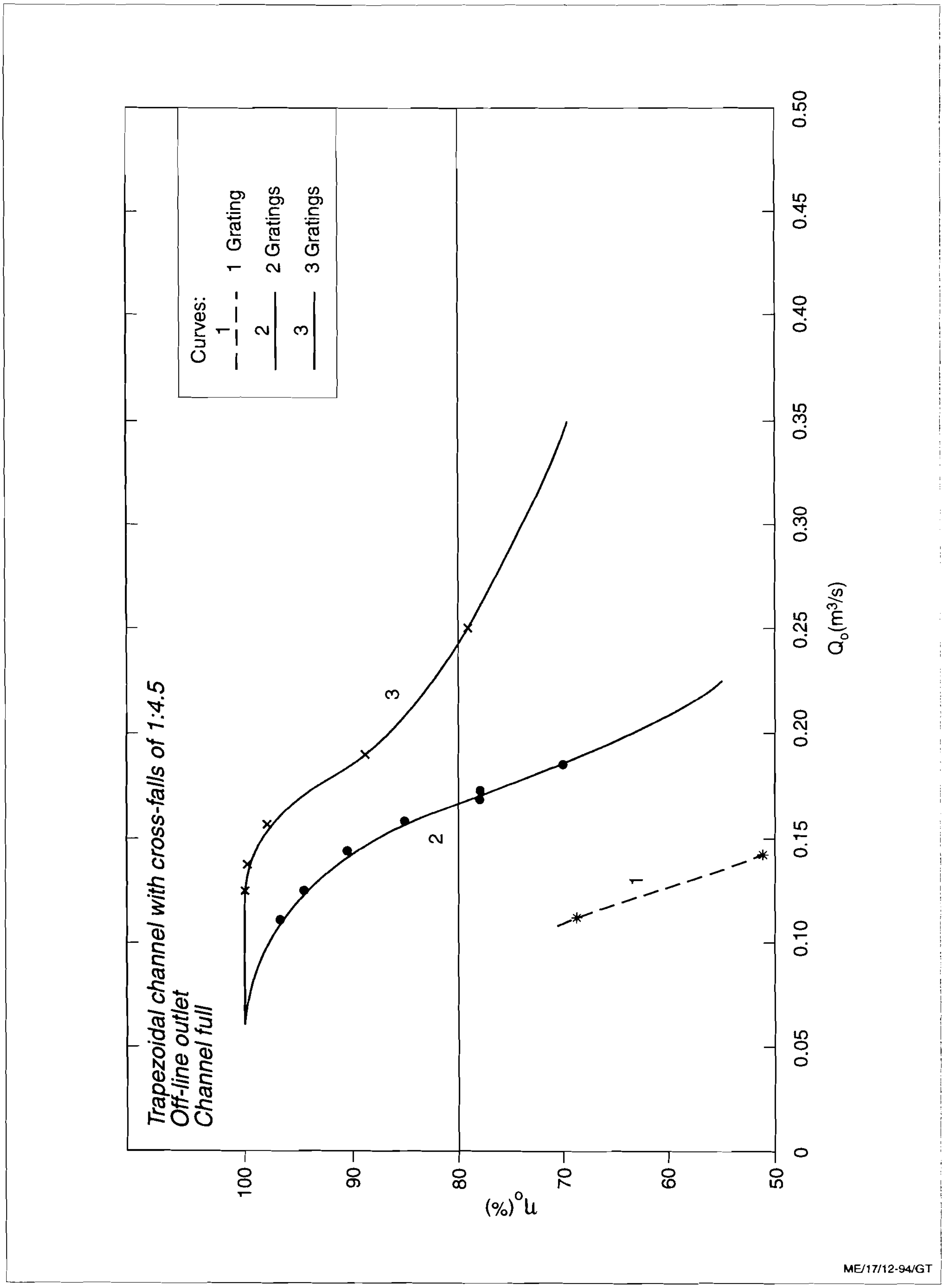


Figure 21 Design curves. Trapezoidal channel with cross-falls of 1:4.5 Off-line outlet. Channel full

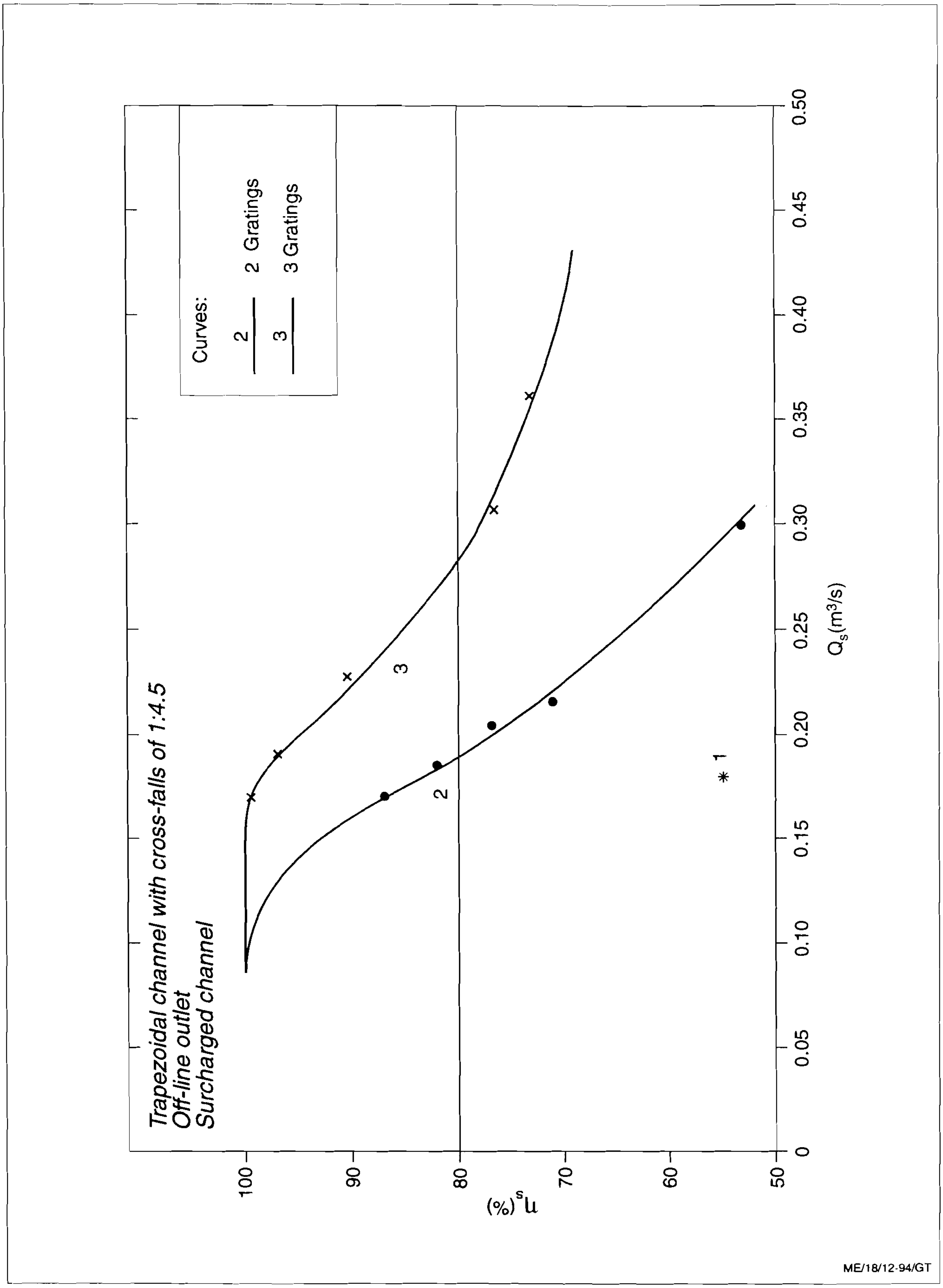


Figure 22 Design curves. Trapezoidal channel with cross-falls of 1:4.5 Off-line outlet. Surcharged channel

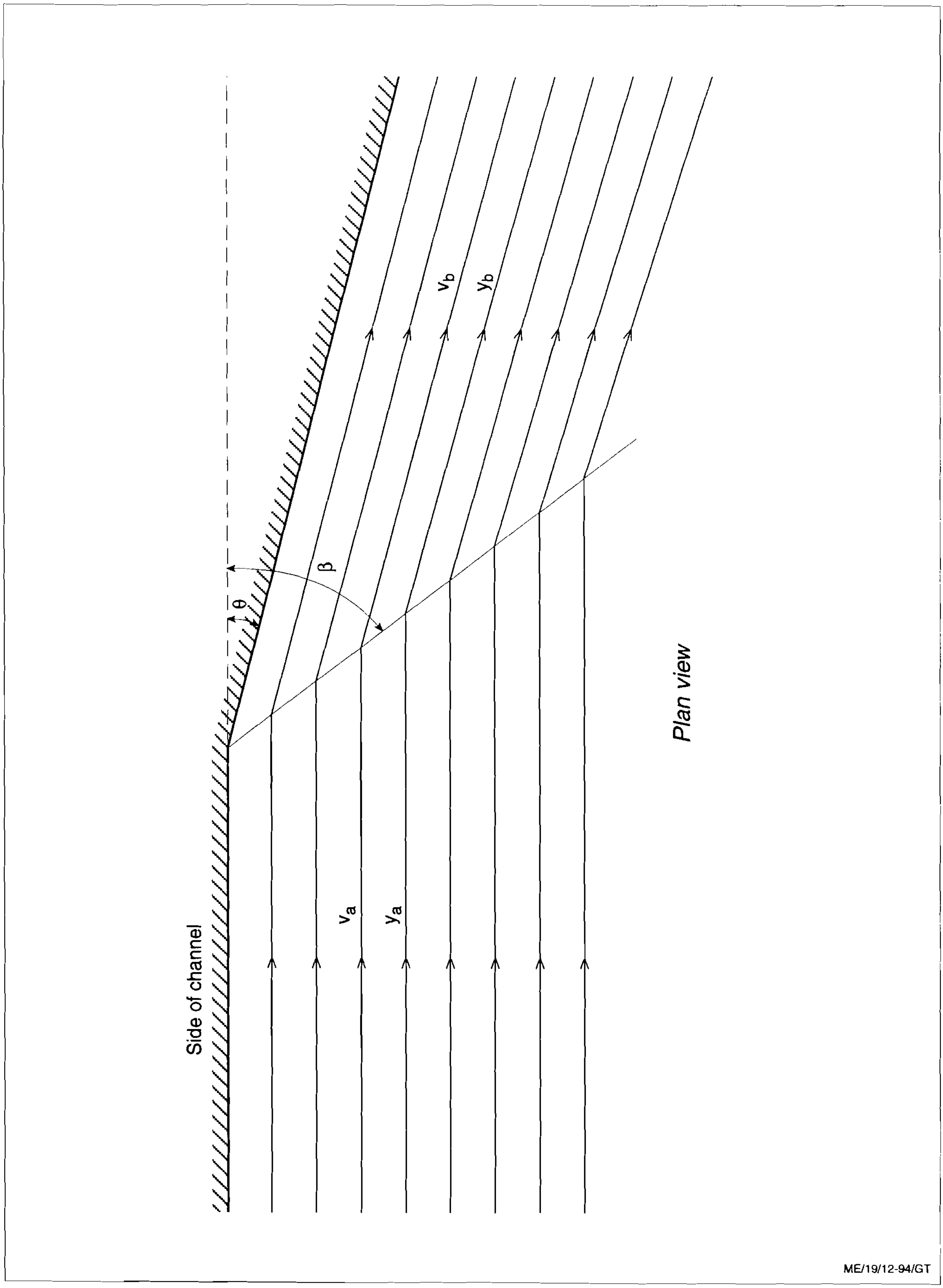


Figure 23 Formation of oblique wave due to deflection of channel walls

Plates

Plate 1 Example of asymmetrical triangular channel

Plate 2 Example of symmetrical triangular channel

Plate 3 A20 Folkestone to Dover (Contract 1). Central reserve outlets

Plate 4 A20 Folkestone to Dover (Contract 1). Test of central reserve outlets

Plate 5 A20 Folkestone to Dover (Contract 1). Test of verge outlet

Plate 6 A487 Port Dinorwic Bypass. Terminal outlet

Plate 7 A487 Port Dinorwic Bypass. Test of terminal outlet

Plate 8 A487 Port Dinworic Bypass. Test of intermediate outlet (slope 1:16.5)

Plate 9 A487 Port Dinworic Bypass. Test of intermediate outlet (slope 1:204)

Plate 10 General view of flume with in-line outlet in triangular channel (2 pairs of gratings)

Plate 11 Triangular channel
In-line outlet, 2 pairs of gratings
Channel-full, slope 1:60

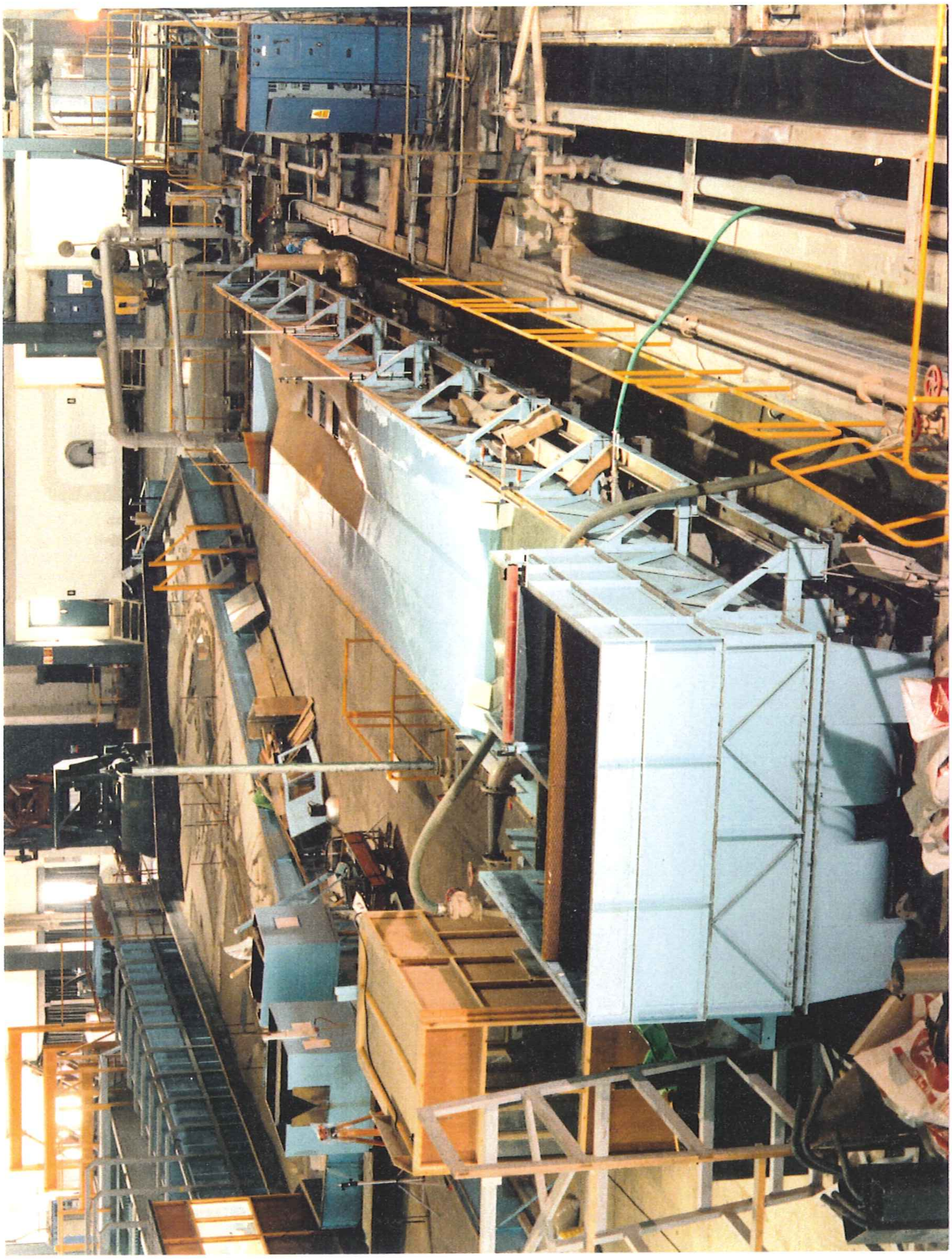


Plate 12 General view of flume with off-line in triangular channel (3 gratings)

Plate 13 Triangular channel
Off-line outlet, 3 gratings
Surcharged,slope 1:100

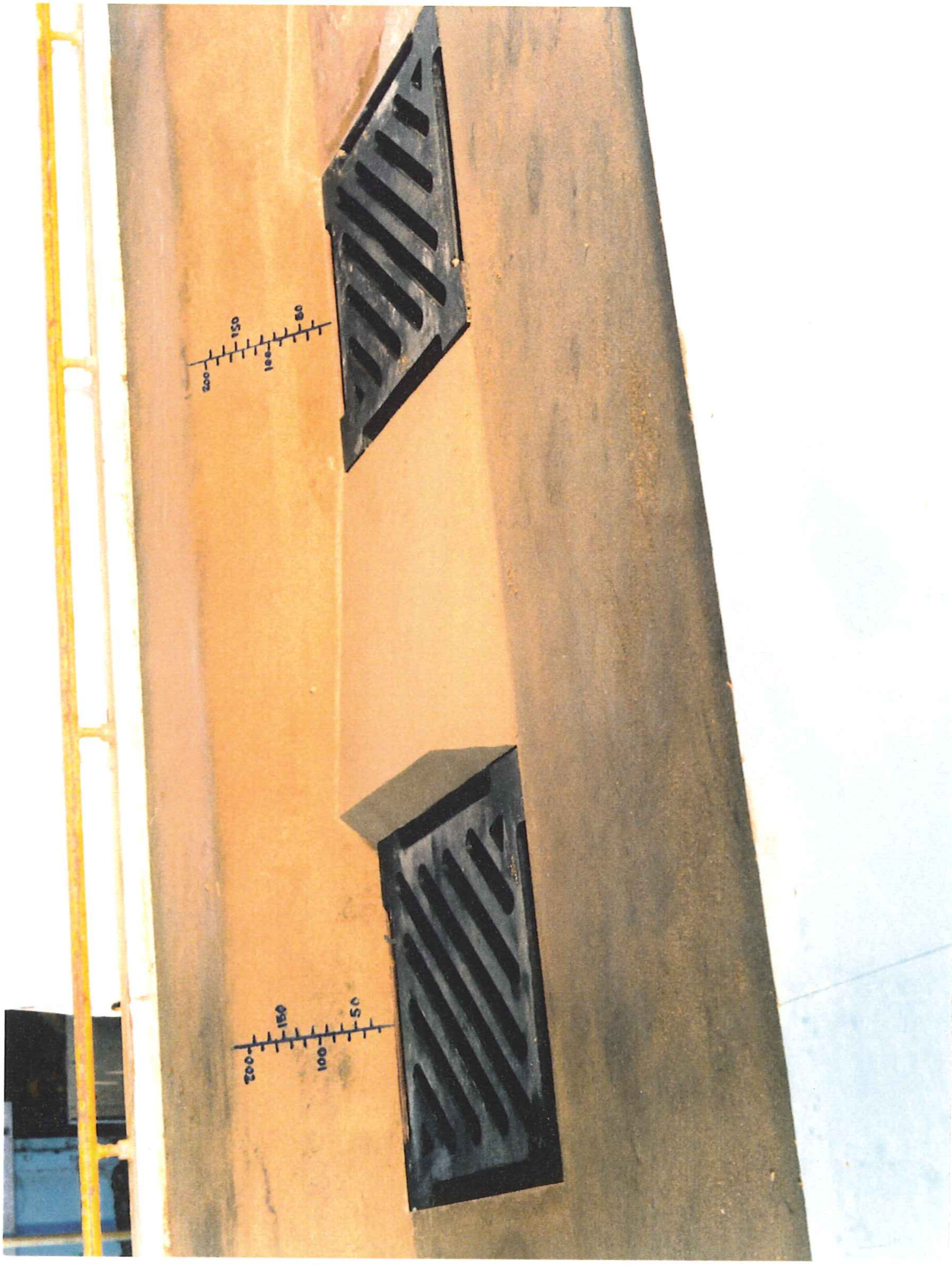


Plate 14 Triangular channel
Off-line outlet
Ramps between gratings

Plate 15 Triangular channel
Ramps between gratings
Channel-full, slope 1:60

Plate 16 Trapezoidal channel
In-line outlet, 3 gratings

Plate 17 Trapezoidal channel
In-line outlet, 3 gratings
Channel-full, slope 1:400

Plate 18 Trapezoidal channel
Off-line outlet, 3 gratings

Plate 19 Trapezoidal channel
Off-line outlet, 3 gratings
Channel-full, slope 1:400

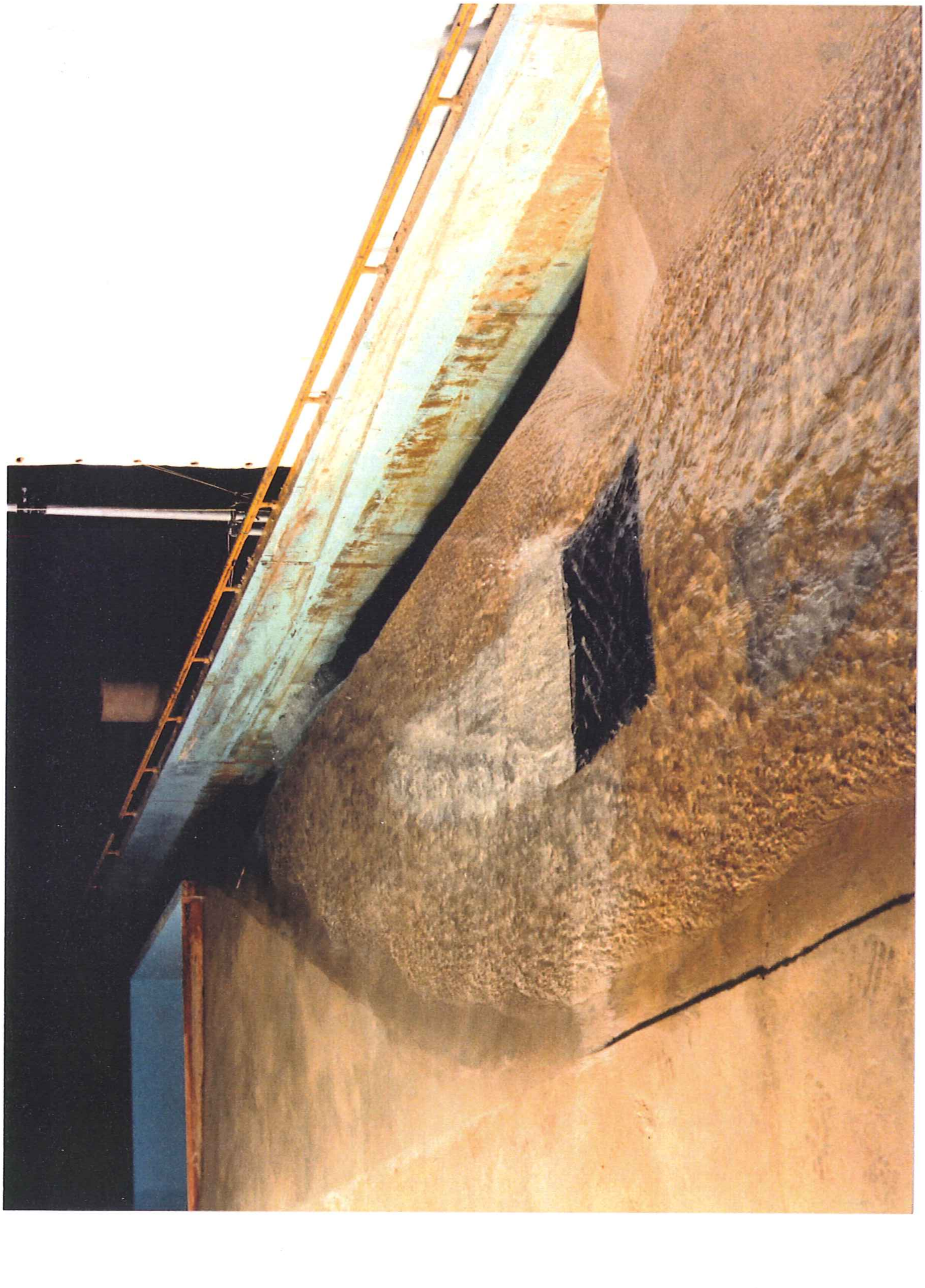


Plate 20 Weir outlet with grating
Slope 1:40

Appendix I

General Questionnaire

Questionnaire on Surface Water Channels and Outfalls

▷ Insert information in appropriate box
√ Tick appropriate box

Questionnaire No.	▷
DOT Operating Unit	▷

A

Name of Scheme	▷				
Stage	Design	Tender	Construction	Maintenance period	Maintenance
	√	√	√	√	√

B

Design Organisation	County Council / Met. Borough	Consulting Engineer	
	▷	▷	
Construction Organisations *(if applicable)*	Main contractor	Channel contractor	Site supervision
	▷	▷	▷
Maintenance Organisation *(if applicable)*	▷		

C

Contact Person(s) for Scheme	Name and Address	Telephone
	▷	▷

D

Overall Description of Scheme

	Rural						Urban		
Carriageway Cross-section *(as defined by HCD)*	M/A1	AP/A2	ML/A3	APL/A4	MS/A5	APS/A6	M/A8	AP/A9	SL/A10
	√	√	√	√	√	√	√	√	√

	Flexible	Rigid	Composite
Type of Pavement	√	√	√

	Length of road (km)	Width of carriageway (m)
Dimensions	▷	▷

	Maximum	Minimum	
Longitudinal Gradients *(eg 1:300)*	▷ 1:	▷ 1:	Flat √

	Fin drain			Narrow filter drain		Combined (French) drain	Other
Type of sub-surface drainage *(as defined by HCD)*	Type 5	Type 6	Type 7	Type 8	Type 9		
	▷	▷	▷	▷	▷	▷	√

▷ Insert information in appropriate box
√ Tick appropriate box
* If channel has vertical side, enter side slope 1: 0

E

Description of Surface Water Channels

in verge

	Verges in Cutting			Verges on Embankment		
Pavement edge detail *(as defined by HCD)*	B2	B3	Other	B11	B12	Other
	√	√	√	√	√	▷

	Slip-form concrete	Precast concrete	Other
Method of construction	√	√	▷

	Design flow width (mm)	Overall depth (mm)	Side-slopes *	
Geometry of channel	▷	▷	▷ 1:	▷ 1:

If more channel types, please use extra pages

in central reserve

	Central reserve		
Pavement edge detail *(as defined by HCD)*	B6	B7	Other
	√	√	▷

	Slip-form concrete	Precast concrete	Other
Method of construction	√	√	▷

	Design flow width (mm)	Overall depth (mm)	Side-slopes	
Geometry of channel	▷	▷	▷ 1:	▷ 1:

If more channel types, please use extra pages

F

Description of Channel Outfalls

	In verge A	In central reserve	In verge B
Total number of channel outlets	▷	▷	▷

	Grating in channel invert	Grating set back in verge	Kerb inlet	Side chute	Other
Types of outlet	√	√	√	√	▷

	Maximum distance (m)	Minimum distance (m)
Distance between outlets	▷	▷

	Watercourse	Carrier drain	Toe ditch	Another channel	Other
What do outlets discharge into?	√	√	√	√	▷

G

	Year of publication
Edition of HCD referred to in parts D and E	▷

NOTES FOR COMPLETION OF QUESTIONNAIRE ON SURFACE WATER CHANNELS AND OUTFALLS

1. This questionnaire concerns road schemes with surface water channels that have already been built or are at any stage between design and construction.

2. Separate forms should be completed for each scheme (the two pages of the questionnaire should be copied as many times as necessary). If an overall scheme consists of sections with different characteristics (eg, motorway, slip road, link road), each section should be described separately.

3. In Part F, give separately the numbers of outlets in both verges and the central reserve (if applicable). For example, verge A might be on the northbound carriageway and verge B on the southbound carriageway.

4. Some of the questions refer to standard designs in the DOT Highway Construction Details (HCD). Please identify in Part G which edition of the HCD applies to the scheme.

5. After analysis of the data, a few representative schemes will be selected for more detailed study. Please therefore identify in Part C of the questionnaire a contact person for each scheme who would be able to assist if such a follow-up is required.

6. Please return all completed forms by 7 May 1993 to :

Mr R W P May
HR Wallingford
Wallingford
Oxfordshire
OX10 8BA

Tel : 0491 35381 Fax : 0491 25428

If you have any problems in completing the questionnaire or comments on the project, please do not hesitate to contact Richard May at the above address.

Appendix Ⅱ
Detailed Questionnaire

SURFACE WATER CHANNELS AND OUTFALLS : DETAILED QUESTIONNAIRE

A. HYDRAULIC DESIGN

A.1 Channels

(a) (1) Were other drainage options studied?

(2) If yes, why were surface channels chosen?

(b) Did HA 37/88 explain the design method satisfactorily?

(c) Were there any problems in applying the method?

eg (1) storm durations too short for method
(2) estimation of run-off from cuttings
(3) non-uniform catchment characteristics
(4) non-uniform channel slope
(5) long sections of low gradient
(6) drainage of sag points
(7) lack of suitable discharge points
(8) need for separate carrier pipes
(9) compatibility with design method for piped system
(10) HA 37/88 gave different results from other methods for outlet spacing
(11) other : please specify

(d) How could the design method be improved?

eg (1) fewer restrictions on its use
(2) non-graphical method
(3) computerised method
(4) other cross-sectional shapes
(5) other : please specify

A.2 Pavement drainage

(a) What type of pavement drainage was chosen?

(b) Is a separate carrier pipe used for the pavement drainage?

(c) (1) Does the pavement drainage connect to a carrier pipe serving the surface drainage?

(2) If yes, how frequent are the connections?

A.3 Outfalls

(a) What alternative designs were considered?

(b) Describe selected outfall design in detail (drawings and photos if possible).

(c) How were the flow capacities of the outfalls estimated?

(d) Is field testing of one or more outfalls feasible?

B. CONSTRUCTION

B.1 From designer's viewpoint

(a) (1) Was the Specification for the channels satisfactory?

(2) If not, how could it be improved?

(b) Did the channels save money compared with alternative drainage methods?

(c) How could the overall drainage system (surface channels and pavement drainage) be improved?

B.2 From supervisor's viewpoint

(a) (1) Was the Specification for the channels satisfactory?

(2) If not, how could it be improved?

(b) Did the channels save time compared with alternative drainage methods?

(c) Did the use of channels help or hinder the construction of the road foundation and pavement?

(d) (1) Was it difficult to achieve the required tolerances on line, level and shape of the channels?

(2) If yes, were the tolerances impractical or are they achievable with experience?

(e) How could the construction of the channels be improved?

eg (1) different shape
(2) different size
(3) different construction method
(4) different concrete mix
(5) other : please specify

(f) Were there difficulties with the construction of the pavement drainage system?

(g) If the road were to require an overlay in the future, how best could the surface channels be modified?

B.3 From contractor's viewpoint (eg slip-form contractor)

(a) (1) Was the Specification for the channels satisfactory?

(2) If not, how could it be improved?

(b) (1) Was it difficult to achieve the required tolerances on line, level and shape of the channels?

(2) If yes, were the tolerances impractical or could they be achieved with experience?

(c) How could the construction of the channels be improved?

eg (1) different shape
(2) different size
(3) different construction technique
(4) different concrete mix
(5) other : please specify

(d) If the road were to require an overlay in the future, how best could the surface channels be modified?

C. SAFETY

(a) When was the road opened?

(b) How heavily used is the road?

(1) Number of commercial vehicles per day assumed in design

or

(2) Average daily traffic flow

(c) Are the channels outside the safety barriers?

(d) Have any accidents or near-accidents occurred as a result of the channels or outfalls?

(e) Are the channels or outfalls considered to be a possible hazard to road users?

D. MAINTENANCE

(a) Have the channels or outfalls been damaged by vehicles?

(b) Has surface flooding of the road been reported?

(c) Do the channels collect much grit and other debris?

(d) Are the outfalls easily clogged?

(e) How often are the channels and outfalls cleaned?

(f) Are there particular problems in cleaning the channels and outfalls?

(g) How could the use of surface channels be improved?

eg (1) different shape
(2) different size
(3) other : please specify

(h) Has the pavement drainage system caused any problems?

(i) If the road were to require an overlay in the future, how best could the surface channels be modified?

Appendix III

Advice Note on Design of Outfalls for Surface Water Channels

ADVICE NOTE

DESIGN OF OUTFALLS FOR SURFACE WATER CHANNELS

Draft dated December 1994

DESIGN OF OUTFALLS FOR SURFACE WATER CHANNELS

DRAFT DATED DECEMBER 1994

DECEMBER 1994

CONTENTS

TABLES

FIGURES

1. INTRODUCTION

1.1 Surface water channels for drainage of runoff from highways can be a suitable alternative to conventional kerbs and gullies or filter drains. Amongst other advantages, such as providing separate systems for drainage of surface and sub-surface water, they allow greater distances between outlets when compared with conventional gully systems.

1.2 Advice Note HA 37/88 (and Amendment No 1 dated March 1991) provides a method of determining the required spacing between outlets for surface water channels. The channel cross-falls should not normally be steeper than 1:5 but in very exceptional cases cross-falls of 1:4 are allowed. The maximum design depth of the channel is restricted to 150mm.

1.3 Rectangular channels and triangular channels deeper than 150mm can be used behind safety barriers. In these locations cross-falls exceeding 1:4 are allowed.

1.4 Flow rates in surface water channels are generally much higher than in equivalent kerb-and gully systems. Therefore, special designs of channel outfall are needed to obtain a satisfactory level of performance. In this Advice Note, the outfall is defined as the drainage system that collects and removes water from the surface water channels and conveys it to a downstream point of discharge. The transition section in the channel that collects the water and the set of gully gratings or the overflow weir that removes the water from the surface are collectively termed the outlet. The chamber below the outlet and the arrangements for conveying the water to a collector pipe, a soakaway or a watercourse are collectively termed the "outfall structures".

1.5 The designs of outlets recommended in this Advice Note were developed from laboratory tests. Details of the test data are given in HR Report SR 406, 1994.

2. SCOPE

2.1 This Advice Note describes suitable layouts for outlets from triangular and trapezoidal surface water channels and provides methods of designing each type according to the flow rate in the channel. Some general recommendations regarding the design of the outfall structures are also given in this Advice Note (Section 7).

2.2 The design methods enable the performance of the outlets to be assessed for channel-full conditions and for surcharging conditions when the flow may extend to the edge of the carriageway. The channel-full conditions are normally specified to correspond to storms with a return period of 1 year whereas the surcharged situation typically refers to storms with a return period of 5 years. It should be noted that surcharging is not allowed for channels built in the central reserve.

2.3 The design methods apply to symmetrical triangular channels with cross-falls of 1:5 and also to higher capacity channels with a trapezoidal cross-section.

2.4 High capacity channels are required for drainage of wide roads and long lengths with flat gradients. In such situations, trapezoidal cross-sections provide higher capabilities than triangular channels of the same depth and surface width. The base width of the recommended channels have been chosen to allow the use of certain standard sizes of gratings. Two geometries of trapezoidal channel are considered. They both have a base width of 0.300m and channel-full depth equal to 0.150m but are distinguished by cross-falls of 1:4.5 and 1:5. In order to promote self-cleansing conditions, the base of each channel has a cross-fall of 1:40 towards the verge (or central reserve). The two channel shapes can be modified at the outlet to accommodate a 0.450m wide grating in the invert with the sides of the channels locally steepened to slopes not exceeding the allowable limit of 1:4 (see Section 1.2).

2.5 Figures 1 and 2 show the cross-sectional shapes of the recommended channels. As shown in these figures, y_1 is the depth of the channel from the lower edge of the carriageway, y_2 is the depth of the channel from the upper edge of the carriageway, and y_3 is the overall depth of the surcharged channel. The allowable width of surcharging should not exceed 1m for hard-strips or 1.5m for hard-shoulders.

2.6 Three different geometries of outlet are recommended for each of the two types of channel considered. One is an in-line outlet, where the water is essentially collected symmetrically either side of the channel invert. Another type is an off-line outlet, where the channel is widened away from the carriageway and the outlet is off-set from the centreline of the channel. A third type of outlet, a weir outlet, is recommended for steep slopes (typically >1:50) where the water is made to curve towards a side-weir.

2.7 As described in Advice Note HA 37/88, Clauses 2.2 and 2.3, the longitudinal gradient of the channel may be zero at the upstream or downstream end of the channel but all intermediate points must have a positive slope towards the outlet.

2.8 This Advice Note does not cover the structural design of the outlets or of the flow-collecting chambers underneath the outlet gratings. However, diagrams of possible configurations of the chambers are included for illustrative purposes [see Appendix A].

3. FLOW CONDITIONS APPROACHING OUTLET

3.1 The flow rate to use in the design of the outlets should be calculated according to Advice Note HA 37/88 which adopts Manning's resistance equation:

$$Q = \frac{A\,R^{2/3}\,S^{1/2}}{n} \qquad (1)$$

where Q is the flow rate (m^3/s), A is the cross-sectional area of the flow (m^2), S is the longitudinal gradient of the channel (m/m) and n is the Manning roughness coefficient. The hydraulic radius R is defined by:

$$R = \frac{A}{P} \qquad (2)$$

where P is the wetted perimeter, ie the perimeter of the channel in contact with the water flow. Values of Manning's n are given in Table 2 of Advice Note HA 37/88.

3.2 If the longitudinal gradient of the channel is not uniform along its length, an equivalent value of the slope, S_e, should be used in the calculation of the flow rate. S_e should be evaluated as described in Section 8 of the Advice Note HA 37/88.

3.3 When checking for surcharged conditions, the flow rate, Q_s, to use in the design of outlets for triangular channels can be estimated from Figure 3. In this figure B_o and Q_o are respectively the surface width of the channel and the flow rate corresponding to channel-full conditions. Q_o is equal to the value of Q given by Equation (1) when A and R correspond to the design depth of flow, y_o, in the channel. The curve in Figure 3 is based on 1m width of surcharging of the carriageway at a cross-fall of 1:40. The value of Q_s/Q_o can be read off the curve and, with Q_o calculated using Equation (1), the value of Q_s can then be determined.

3.4 For the trapezoidal channels with cross-falls of 1:4.5 the channel-full flow is given by $Q_o=0.0271S^{1/2}/n$; for trapezoidal channels with cross-falls of 1:5 the channel-full flow is given by $Q_o=0.0290S^{1/2}/n$. For both trapezoidal channels the ratio between the flow rates corresponding to surcharged and channel-full conditions is 1.21, ie $Q_s = 1.21\,Q_o$.

4. TYPES OF OUTLET

4.1 Channel outlets can be defined as intermediate or terminal according to their position along a channel. Terminal outlets are located at low points along a length of channel and should be designed to collect practically all the flow carried by the channel. Intermediate outlets are located at points part-way along a length of channel where the flow rate of water from the road reaches the carrying capacity of the channel.

4.2 The design methods in this Advice Note are based on a minimum value of the waterway area (defined as the total area of openings) in relation to the plan area of the grating. If G is the width of the grating, this minimum waterway area is defined as $0.44G^2$. The efficiencies of outlets comprising gratings with bigger waterway areas and similar bar patterns will not be less than given by these methods. The laboratory tests, which are the basis of the present recommendations, were carried out with gratings of representative geometry (see Figure 4). Gratings with bar patterns consisting of longitudinal and transverse bars were also tested and their application is discussed in Section 5.4.

4.3 As mentioned in Clause 2.6, alternative designs of in-line and off-line outlets are recommended for each of the two types of channel. For triangular channels the in-line outlet recommended is generally more efficient than the off-line outlet but reasons for choosing between them will mainly depend on constructional aspects. Wherever possible, in-line outlets are preferable to off-line outlets since they require a smaller land take. However, in-line and off-line outlets are not suitable for steep channels where the high kinetic energy of the flow renders gratings less effective. In such situations the flow should be collected by curving it towards an off-line weir (see Section 6).

4.4 Triangular channels

4.4.1 The in-line outlet geometry recommended for this type of channel consists of pairs of gratings positioned on the side slopes of the channel (see Figure 5).

The number of pairs of gratings required will depend on the amount of flow in the channel (see Section 5). More than 3 pairs of gratings are considered to be uneconomical, and other measures should be taken to cope with higher flows (see Section 6).

The spacing between pairs of gratings should not be less than 1.7 G, where G is the width of the grating (see Figure 4). The size of the required gratings should be chosen so that the ratio of the width G over the depth of the channel y_1, is within the following limits:

$$4.5 \leq G/y_1 \leq 5.1 \qquad (3)$$

The lower limit corresponds to the minimum width of grating necessary to achieve the performance specified in Section 5. The upper limit corresponds to the widest grating that can be installed in the channel.

The length H of the gratings should be equal to or bigger than G.

The lower edge of each grating should be set as close as possible to the invert of the channel in order to maximise flow interception, ie distance e in Figure 5 should be minimised. Although they are not commercially available yet (1994), manufacturers may consider producing angled gratings specifically for this purpose to simplify installation. An in-line outlet with the grating set flat in the channel invert is not permissible because this layout would require cross-falls locally steeper than 1:4 (see Clause 1.2).

4.4.2 The recommended geometry of off-line outlet is shown in Figure 6. The number of gratings may vary from one to three depending on the amount of flow approaching the outlet (see Section 5). However, outlets formed by a single grating may have the disadvantage of being easily blocked by debris, particularly when the outlets are widely spaced. Consequently, a second grating would reduce the likelihood of local flooding of the road in those situations when the first grating is blocked. On the other hand, outlets including more than three gratings may not prove economical due to the space they require and the size of the flow collecting structure under the outlet. For these cases a weir outlet is recommended - see Section 6.

In this geometry the side slope on the road side is extended below the invert level of the channel to produce a ponding effect over the gratings which increases the efficiency of the outlet. A gradual transition between the channel and the outlet is essential to direct the flow smoothly towards the gratings.

Local cross-falls should not be steeper

than 1:4 and the spacing between gratings should not be less than 1.25G where G is the width of the gratings. The size of the gratings is determined by: $4.5 \leq G/y_1$.

4.5 Trapezoidal channels

4.5.1 The in-line outlet geometries recommended for the trapezoidal channels are shown in Figures 7 and 9. The gratings have nominal dimensions of 450mm x 450mm; the allowable width G of the grating should not exceed 450mm in this size of trapezoidal channel. The comments in Clause 4.4.2 regarding the number of gratings, the importance of a gradual transition and the local side slopes apply also to this case.

4.5.2 The off-line geometries recommended are shown in Figure 8 and 10. The dimensions of the gratings in this design are 610mm x 610mm. As for the in-line outlet, the comments in Clause 4.4.2 are also applicable to this case.

4.6 Terminal outlets

The requirement that surface water channels should not have any sides steeper than 1:4 applies also to the geometry of terminal outlets. When not protected by a safety barrier, surface water channels must therefore terminate with a smooth transition, without abrupt changes in level or width. Examples of recommended terminal outlets are shown in dashed lines in Figures 5 to 10. The terminal ramps should be built at a certain minimum distance from the grating furthest downstream. This reduces the probability of blockage of the gratings by debris since some of the debris will tend to accumulate in the area between the gratings and the terminal ramp. For in-line and off-line outlets in triangular channels, this distance should equal the grating width. For in-line and off-line outlets in trapezoidal channels the recommended distances are shown in Figures 7 to 10.

5. HYDRAULIC DESIGN OF OUTLETS

5.1 General procedure

The design procedure involves choosing the type of outlet (in-line, off-line or weir outlet) and the number of gratings needed to achieve the required performance. The geometry of each type of outlet is predetermined as described in Clauses 4.4 to 4.6 and illustrated in Figures 5 to 10. For triangular channels the size of the gratings is related to the size of the channel in accordance with Equation (3) or Clause 4.4.2.

The performance of an outlet should be determined for channel-full conditions but checks of the performance for surcharged flow conditions may also be carried out.

For intermediate outlets, design curves are presented which give the number of gratings needed to achieve the required performance of the outlet (Figures 11 to 22). For terminal outlets the number of gratings is obtained from Tables 1 and 2. For triangular channels the flow conditions are represented by a non-dimensional number so that the design procedure is valid for all sizes of channel; for the trapezoidal channels a non-dimensional number is not used because the two channels have a fixed size.

This design procedure is suitable only for channels with small to moderate slopes. In steep channels (typically >1:50) the procedure to adopt is described in Section 6.

5.2 Intermediate outlets

5.2.1 The hydraulic design of intermediate outlets is based on a number of curves (Figures 11 to 22) developed for channel-full and surcharged conditions. These curves show the variation of the efficiency of each outlet with the flow conditions.

In the curves for triangular channels, the flow conditions are represented by a non-dimensional number: F_o for channel-full and F_s for surcharged channel. The values of F_o and F_s are calculated respectively as:

$$F_o = \frac{28.6Q_o}{B_o^{2.5}} \qquad (4)$$

and

$$F_s = \frac{24.6Q_s}{B_1^{2.5}} \qquad (5)$$

where

Q_o is the approach flow corresponding to channel-full conditions (in m^3/s)

Q_s is the approach flow corresponding to surcharged conditions (in m^3/s)

B_o is the surface width of the flow for channel-full conditions (in m)

B_1 is the surface width of the flow in a surcharged channel neglecting the width of surcharge on the hard strip or hard shoulder - see Figures 1 and 2 - (in m).

For the estimation of Q_o and Q_s refer to Section 3. In Equation (4) the numerical constant is chosen so that F_o is equal to 1 when the flow in the channel is at critical depth for channel-full. The value of F_s is defined so that when there is surcharged flow it gives approximately critical flow in a composite channel.

In the curves for the trapezoidal channels (Figures 15 to 22), the flow rate is plotted on the x-axis, ie Q_o for channel-full and Q_s for surcharged conditions. A non-dimensional number is not used as the trapezoidal channels are of fixed size.

5.2.2 Values of efficiency are plotted on the vertical axis of the design curves. The efficiency of an outlet is defined as the ratio of the flow intercepted by the outlet, Q_i, to the total flow approaching it:

$$\eta_o = Q_i/Q_o \qquad (6)$$

$$\eta_s = Q_i/Q_s \qquad (7)$$

where η_o and η_s refer to channel-full and surcharged conditions, respectively.

5.2.3 Although efficiencies of 100% may be desirable, the resulting outlets may be large and costly; more economic designs can often be achieved by allowing a certain amount of flow to by-pass intermediate outlets. However, it is recommended that intermediate outlets operating under channel-full conditions should not be designed for efficiencies less than 80%.

5.2.4 The design charts for triangular channels (Figures 11 to 14) include curves for one, two and three gratings or pairs of gratings. The design charts for trapezoidal channels (Figures 15 to 22) include curves for only two and three gratings because it is recommended to adopt a minimum of two gratings for high capacity channels. The curves shown dashed were obtained by extrapolating the results of the laboratory tests using a conservative approach.

5.2.5 For the triangular channels, the designer should use the value of Q_o calculated as described in Section 3 to determine F_o defined by Equation (4). Having decided which type of outlet to adopt (in-line or off-line outlet), the number of gratings necessary to achieve the required efficiency is then read off the curves. Alternatively, the designer can check whether a particular outlet geometry or number of gratings is adequate for the approach flow. For the trapezoidal channels, the procedure for the triangular channels is used except that Q_o is introduced directly in the design curves so there is no need to determine the value of F_o.

5.2.6 It is recommended that outlets should normally be designed for channel-full conditions (ie, the 1-year return period event) but the designer may wish to check the performance for surcharged flow conditions. Figures 12, 14, 16, 18, 20 and 22, which correspond to a width of surcharging of 1m, should then be used.

5.2.7 For triangular channels, the minimum width of grating, G, required for an outlet is determined by Equation (3) or Clause 4.4.2; the length, H, should not be less than G. The designer should choose a size of commercially available grating that is not smaller than the calculated values of G and H and that provides a waterway area of opening between bars that is not less than required in Clause 4.2. For trapezoidal channels, the grating dimensions are given in Section 4.5.

5.3 Terminal outlets

5.3.1 The efficiency of a terminal outlet is generally higher than that of a similar intermediate outlet because of the effect of the end ramp. Also, a terminal outlet needs to be designed for an efficiency close to 100%, because any water by-passing the outlet may flow on to the verge or back on to the road.

5.3.2 For the design of terminal outlets, the first step is to calculate values of F_o and F_s for triangular channels, or Q_o and Q_s for the trapezoidal channels, as described in Clauses 5.2.1 and 3.4. The values of F_o or Q_o should then be compared with the limiting values given in Table 1 (for triangular channels) or Tables 2 and 3 (for the trapezoidal channels). The type of outlet selected should have a limiting value of F_o or Q_o that is not less than the calculated value. As for the case of intermediate outlets, a check may be carried out for surcharged conditions using the calculated values of F_s or Q_s.

The values presented in Tables 1, 2 and 3 for terminal outlets correspond to efficiencies of 97.5%. The small amount of by-passing that is permitted is considered acceptable for rare storm events.

5.4 Grating design

As mentioned in Clause 4.2, the design curves were based on tests carried out with gratings having a diagonal bar pattern. Comparing the performance of gratings equivalent in terms of overall size and waterway area, longitudinal bars are more efficient than diagonal bars, which in turn are more efficient than transverse bars.

However, longitudinal bar patterns are not allowed along the carriageway for two main safety reasons: 1) bicycle tyres may get trapped in the slots between the bars; and 2) longitudinal bars provide a lower skidding resistance than diagonal bars.

In the exceptional cases where surface water channels are built behind safety fences, gratings with longitudinal bars may be adopted. In such a case, the value of efficiency obtained from the relevant design curve for diagonal bars should be increased. If η_D is the efficiency corresponding to a diagonal bar pattern, the efficiency η_L corresponding to a longitudinal bar is approximately given by :

$$\eta_L = 0.5 + 0.5\,\eta_D \qquad (8)$$

The positioning of gratings with bars transverse to the direction of the flow is not recommended in any situation since it reduces the outlet efficiency considerably.

6. WEIR OUTLET

6.1 When the types of outlet illustrated in Figures 5 to 10 cannot guarantee the necessary level of flow-collecting efficiency (80% for intermediate and 97.5% for terminal outlets), a weir outlet is required. In this situation the water needs to be gently directed away from the road and discharged into a collecting channel in the verge by means of a side weir. For safety reasons a safety fence should be installed along the carriageway-side of the collecting channel. The recommended layout of the weir outlet is shown in Figures 23 to 25.

6.2 Triangular channels

The procedure to design a weir outlet for triangular channels is explained in the flow chart of Figure 26. In order to allow the high-velocity flow to be turned towards the side weir without spilling out on to the carriageway, it is necessary for the channel to be flowing only part full immediately upstream of the outlet. Therefore, once the need for a weir outlet is established, there are two possible options: 1) to drain the same area (ie keep the same flow) and increase the depth of the channel so that it flows part-full at the outlet; and 2) to keep the same channel size and drain a smaller catchment (ie reduce the flow).

In option 1 the designer may choose to adopt an increased channel size for the whole scheme, increase the channel size only in the drainage length under consideration, or just locally at the approach to the weir outlet. When the depth of the channel is increased just locally a gradual transition should be built to accommodate the change in the level of the channel invert. A minimum longitudinal slope of 2.5% is recommended for this transition. The depth of the channel approaching the weir outlet should be increased so that the local water depth under design conditions is 2/3 of the channel depth.

In option 2 the design water depth in the channel is reduced to 2/3 of its original value.The spacing of the outlets therefore needs to be revised using Advice Note HA 37/88 before the design of the weir outlet can proceed. A similar situation can also occur if the increased channel depth required in option 1 exceeds the maximum allowable depth of the channel, as stated in Clauses 1.2 and 1.3. New values of the flow rate and of the non-dimensional number, F_o, defined by Equation (4) need to be calculated.

The value of F_o is introduced in Chart A of Figure 27 to give the value of L_w/y_1, where L_w is the total length of the weir outlet and y_1 is the depth of the channel immediately upstream of the outlet. The value of the angle θ (see Figure 24) can then be read off Chart B of Figure 27.

The total length of the weir outlet L_w is formed by a straight stretch, L_s, and a stretch, L_a, at an angle θ to the line of the channel. The value of L_a is determined by:

$$L_a = \frac{B_1}{\tan\theta} \qquad (9)$$

where B_1 is the surface width of flow neglecting the width of surcharge on hard-strip or hard-shoulder of the channel approaching the weir. L_s can be determined by the difference between L_w and L_a. The two stretches should be joined by a circular curve in plan with its upstream end at the mid-point of L_s (see Figure 24).

6.3 Trapezoidal channel with side-slopes 1:4.5

The procedure to design a weir outlet for the trapezoidal channel with side-slopes of 1:4.5 is explained in the flow chart of Figure 28. Once the need for a weir outlet is established, the design water depth should be reduced to 2/3 of its original value, ie 0.100m. The spacing of the outlets therefore needs to be revised using the Advice Note HA 37/88 before the design of the weir outlet can proceed.

The new value of flow rate, Q′, corresponding to the revised spacing is then introduced in Chart A of Figure 29 to give the total length of the weir, L_w. The value of the angle θ (see Figure 25) can then be read of Chart B of Figure 29. See last paragraph of Clause 6.2 for further definition of the geometry of the weir outlet.

6.4 Trapezoidal channel with side-slopes 1:5

The procedure to design a weir outlet for the trapezoidal channel with side-slopes of 1:5 is explained in the flow chart of Figure 30. Once the need for a weir outlet is established, the design water depth should be reduced to 2/3 of its original value, ie 0.100m. The spacing of the outlets therefore needs to be revised using Advice Note HA 37/88 before the design of the weir outlet can proceed.

The new value of flow rate, Q′, corresponding to the revised spacing is then introduced in Chart A of Figure 31 to give the total length of the weir, L_w. The value of the angle θ (see Figure 25) can then be read off Chart B of Figure 31. See last paragraph of Clause 6.2 for further definition of the geometry of the weir outlet.

7. GENERAL RECOMMENDATIONS ON DESIGN OF OUTFALL STRUCTURES

7.1 An outfall conveys water from one or more outlets in a surface water channel to a suitable discharge point. The design of an outfall may vary considerably depending on the general topography and nature of the ground, the layout of the road scheme and whether the water is discharged to a watercourse, a soakaway or a below-ground piped system.

7.2 A chamber or gully pot should be

located below or immediately adjacent to each outlet to collect sediment carried with the flow from the surface water channel. Standard circular gully pots have a limited hydraulic capacity and it is recommended that they should not be used for flow rates exceeding 5 l/s unless their suitability has been determined by test.

7.3 The plan shape of the chamber will be determined by the layout of the gratings forming the outlet. The invert of the outgoing pipe from the chamber should be set a minimum of 300mm above the bottom of the chamber to retain an adequate volume of sediment.

7.4 The invert level of the outgoing pipe should be chosen so that the water level in the chamber does not rise high enough to prevent flow discharging freely from the surface water channel into the outlet. For design, it is recommended that the water level in the chamber should be at least 150mm below the underside of the gratings when the outlet is receiving flow from the channel under surcharged conditions. The height Z (in m), of the water surface in the chamber above the invert of the outgoing pipe can be estimated from the equation:

$$Z = \frac{D}{2} + 0.23\,\frac{Q^2}{D^4} \qquad (10)$$

where D is the diameter of the pipe (in m) and Q is the flow rate (in m^3/s) in the chamber corresponding to surcharged conditions in the surface water channel. The gradient and diameter of the outgoing pipe should be determined from standard flow tables or resistance equation so that the pipe is just flowing full under surcharged conditions.

7.5 Provided the chamber below the outlet is designed to trap sediment, the outgoing pipe from the chamber may be connected directly to a collector pipe by means of a 45° Y junction without the need for a manhole at the junction position.

7.6 If a weir outlet is used (see Section 6), the collecting channel into which flow drops from the weir should be deep enough to allow the outlet to discharge freely when the surface water channel is flowing under surcharged conditions. The minimum depth, J (in m), of the channel invert below the level of the weir can be estimated from the equation:

$$J = 0.15 + 1.3\left(\frac{Q}{E}\right)^{2/3} \qquad (11)$$

where E is the width of the rectangular channel (in m) and Q is the design rate of flow (in m^3/s).The width of the channel should not be less than E = 0.5m.

7.7 It is recommended that the collecting channel below a weir outlet should discharge into a chamber with a removable cover in order to still the flow and allow sediment to be collected. The sizes of the chamber and the outgoing pipe should be determined in accordance with the general recommendations in Clauses 7.3 to 7.6.

8. SPACING OF OUTLETS WITH BY-PASSING

When by-pass flow is allowed in the design of an intermediate outlet, ie when efficiencies lower than 100% are adopted, the design of the channel downstream of the outlet is no longer directly covered by Advice Note HA 37/88. In this case the spacing of the outlets needs to be reduced in order to allow for the additional flow by-passing the upstream outlet. As an interim measure, it is recommended that the distance L between outlets, as determined in HA 37/88, should be reduced to ηL, where η is the adopted design efficiency of the upstream outlet.

9. OVERALL DESIGN OF SURFACE WATER CHANNEL SYSTEMS

In order to obtain the most cost-effective solution for a drainage system using surface water channels, the designer should consider the total cost of the channels and outlets together. In some cases, a design based on the longest possible spacings between outlets may not be the optimum solution. Shorter spacings will require more outlets but these may be smaller and cheaper; also, the shorter distance between outlets will allow use of smaller sizes of surface water channel. The effect on the total cost of allowing different amounts of by-passing at intermediate outlets should also be considered. For each option the relationship between channel size and required outlet spacing should be determined from Advice Note HA 37/88 (plus Amendment No 1), and the effect of allowing by-passing at intermediate outlets should be estimated according to Section 8.

10. WORKED EXAMPLES

10.1 Example 1

Design an intermediate in-line outlet in a triangular surface water channel having the following characteristics:

cross-falls	1:5
design flow depth	0.120m
longitudinal channel gradient	1:200 =0.005
Manning's roughness coefficient (average condition)	0.013

Adopt an efficiency of 100% for the outlets.

The flow in the channel is calculated from Equation (1) but first it is necessary to calculate the hydraulic radius R using Equation (2):

$$R = \frac{A}{P} = \frac{(1.2 \times 0.12)/2}{2(0.120^2+0.6^2)^{0.5}} = \frac{0.072}{1.224} = 0.0588\text{m}$$

The channel-full flow Q_o is then given by:

$$Q_o = \frac{0.072 \times 0.0588^{2/3} \times 0.005^{1/2}}{0.013} = 0.0592\text{m}^3/\text{s}$$

The flow Q_s for surcharging of 1m width of the hard-strip or hard-shoulder is determined from Figure 3. For B_o = 1.2m, Q_s/Q_o = 1.7 and therefore

$Q_s = 1.7 \times 0.0592 = 0.1006\text{m}^3/\text{s}$

Then calculate F_o using Equation (4):

$$F_o = \frac{28.6 \times 0.0592}{1.2^{2.5}} = 1.07$$

Calculate also F_s using Equation (5). It is first necessary to calculate B_1. For a carriageway cross-fall of 1:40, 1m of surcharging corresponds to 0.025m of water depth above the channel-full depth, ie a total depth of 0.145m. Therefore

$B_1 = 5\ (0.120 + 0.145) = 1.325\text{m}$

$$F_s = \frac{24.6 \times 0.1006}{1.325^{2.5}} = 1.22$$

Figures 11 and 12 are appropriate for the design of in-line intermediate outlets in triangular channels.

The designer should begin by considering channel-full conditions, which are described by Figure 11. Adopting an efficiency of 100%, Figure 11 shows the need for two pairs of gratings installed on the sloping sides of the channel. Figure 12 shows that two pairs of gratings are also satisfactory for surcharged conditions.

The size of the gratings (G is width and H is length) is calculated as described in Section 4.3.1:

$4.5 \le G/0.120 \le 5.1$

and $H \ge G$

Taking the smallest dimensions allowed gives

H = 0.120 x 4.5 = 0.540m = 540mm

H = G = 540mm

The designer should therefore choose from commercially available gratings, gratings with width and length not smaller than 540mm.

As shown in Figure 5, the longitudinal distance between the two pairs of gratings should be at least equal to 1.7 x 0.540 = 0.918m if a grating of width 0.540m is chosen.

10.2 Example 2

Design a terminal off-line outlet in a triangular surface water channel with the same characteristics as in Example 1.

The flow in the channel for channel-full conditions Q_o was calculated to be $0.0592m^3/s$ and for surcharged conditions Q_s was found to be $0.1006m^3/s$. The values of F_o and F_s were, respectively, 1.07 and 1.22.

Table 1 should be consulted for the design of terminal outlets in triangular channels. Checking first for channel-full conditions it can be seen that for an off-line outlet, one single grating would be able to intercept the flow satisfactorily (F_o = 1.1 in Table 1 is bigger than the calculated F_o = 1.07). However, when checking for surcharged conditions, Table 1 indicates the need for a minimum of two gratings (F_s = 0.95 in the table is smaller than the calculated F_s = 1.22). The designer is, in this case recommended to adopt two gratings not only to account for floods of higher return period but also for the possibility of partial blockage of the gratings by debris.

The size of the gratings is the same as in the previous example. The longitudinal distance between the two gratings should be at least equal to 1.25 x 0.540 = 0.675m. The total length of the outlet including one upstream transition of 2.02m should be equal to or bigger than 4.9m (see Figure 6 for details of the geometry).

10.3 Example 3

Design an intermediate off-line outlet in a trapezoidal surface water channel having the following characteristics:

cross-falls	1:5
design flow depth	0.150m
base width	0.300m
longitudinal channel gradient	1/333 =0.0030
Manning's roughness coefficient (average condition)	0.013

Adopt an efficiency for the outlets of 85%.

Use Equation (1) to calculate the channel-full flow Q_o:

$$Q_o = \frac{0.0290 \times 0.0030^{1/2}}{0.013} = 0.122m^3/s$$

The flow Q_s corresponding to surcharging of 1m width of the hard-strip or hard-shoulder is given by (see Clause 3.4):

$Q_s = 1.21 \times 0.122 = 0.148m^3/s$

Figures 21 and 22 are appropriate for the design of off-line outlets in the trapezoidal channel under consideration. For channel-full conditions, Figure 21 shows that for an efficiency of 85%, a minimum of 2 gratings is required. Using Q_s in Figure 22 to check for surcharged conditions, it can be seen that 2 gratings are still suitable.

The gratings for this trapezoidal channel have dimensions 610mm x 610mm and the layout of the outlet is the one shown in Figure 10 with two gratings only. The total length of the outlet is 3.62m, including two transitions 0.95m long.

10.4 Example 4

Design a suitable terminal outlet for a triangular surface water channel having the following characteristics :

cross-falls	1:5
design flow depth	0.120m
longitudinal channel gradient	1:25 = 0.04
Manning's roughness coefficient (average condition)	0.013

The flow in the channel is calculated using equation (1) as in Example 1 :

$A = 0.072m^2$
$R = 0.0588m$

$$Q_o = \frac{0.072 \times 0.0588^{2/3} \times 0.04^{1/2}}{0.013} = 0.168m^3/s$$

The value of F_o is calculated using equation (4) :

$$F_o = \frac{28.6 \times 0.168}{1.2^{2.5}} = 3.04$$

From Table 1 (and the flow chart in Figure 26) it can be seen that, because $F_o > 2.30$, neither an in-line or an off-line outlet is adequate and therefore a weir outlet is required. The design should proceed by following the flow chart in Figure 26. The depth of the present channel cannot be increased to 1.5 of its original depth (0.12m) because 1.5 x 0.12 = 0.18 > 0.150m which is the maximum allowable depth (see Clause 1.2). Therefore, it is decided to increase the depth of the channel to 0.150m so that, when flowing 2/3 full, the design flow depth is 0.100m.

The spacing of the outlets needs to be revised using Advice Note HA 37/88 for a design flow depth of 0.100m. If it is deepened only locally at the approach to the outlet, then there should be a transition at least 1.2m long.

A new value of the flow, Q, is obtained as follows :

$$A = \frac{1.00 \times 0.100}{2} = 0.05\text{m}^2$$

$$R = \frac{0.05}{2\ (0.5^2 + 0.1^2)^{0.5}} = 0.0490\text{m}$$

$$Q = \frac{0.05 \times 0.0490^{2/3} \times 0.04^{1/2}}{0.013} = 0.103\text{m}^3/\text{s}$$

$$F_o = \frac{28.6 \times 0.103}{1.0^{2.5}} = 2.95$$

For $F_o = 2.95$, Chart A of Figure 27 gives $L_w/B_1 = 3.8$, ie $L_w = (0.75 + 0.875) \times 3.8 = 6.2$m. From Chart B, $\theta = 21°$ and

$$L_a = \frac{1.625}{\tan 21°} = 4.23\text{m}$$

and

$$L_s = L_w - L_a = 1.97\text{m} \Rightarrow 2.0\text{m}$$

11. GLOSSARY OF SYMBOLS

A	Cross-sectional area of the flow
B_o	Surface width of flow for channel-full conditions
B_1	Surface width of flow in surcharged channel neglecting the width of surcharge on hard-strip or hard-shoulder
D	Pipe diameter
E	Width of rectangular collecting channel
e	Distance between lower edges of pairs of in-line gratings in triangular channels
F_o	Non-dimensional number for channel-full
F_s	Non-dimensional number for flow in surcharged channel
G	Width of gratings
H	Length of gratings
J	Minimum depth of collecting channel
L	Distance between outlets
L_w	Total length of weir outle
L_a	Angled stretch of weir outlet
L_s	Straight stretch of weir outlet
n	Manning roughness coefficient
P	Wetted perimeter of channel
Q	Flow rate
Q_o	Approach flow for channel-full conditions
Q_i	Flow intercepted by outlet
Q_s	Approach flow for surcharged conditions
Q'	Flow rate for revised design flow depth
R	Hydraulic radius of channel

S	Longitudinal gradient
S_e	Value of equivalent longitudinal slope
y_o	Design flow depth
y_1	Depth of the channel from the lower edge of the carriageway
y_2	Depth of the channel from the upper edge of the carriageway
y_3	Overall depth of surcharged channel
Z	Head of water above pipe invert
η	Efficiency
η_o	Efficiency of outlet for channel-full conditions
η_D	Efficiency of outlet for gratings with diagonal bar pattern
η_L	Efficiency of outlet for gratings with longitudinal bar pattern
η_s	Efficiency of outlet for surcharged conditions
θ	Angle of weir outlet

12 REFERENCES

1. Advice Note HA 37/88 "Hydraulic Design of Road-Edge Surface Water Channels", Department of Transport, 1988.

2. Amendment No 1 to HA 37/88, Department of Transport, 1991.

3. Highway Construction Details, Department of Transport, 1987.

4. HR Report SR 406 "Surface Water Channels and Outfalls: Recommendations on Design", HR Wallingford, 1994.

13. ENQUIRIES

TABLES

TABLE 1 - Triangular channels
Limiting values of F_o and F_s for terminal outlets

TYPE OF OUTLET	No OF GRATINGS (OR PAIRS OF GRATINGS) 1	2	3
IN-LINE OUTLET :			
Channel full (F_o)	0.95	2.0	2.3
Surcharged (F_s)	0.80	1.8	2.1
OFF-LINE OUTLET :			
Channel full (F_o)	1.1	1.3	1.8
Surcharged (F_s)	0.95	1.2	1.6

TABLE 2 - Trapezoidal channel with cross-falls of 1:4.5
Limiting values of Q_o and Q_s for terminal outlets

TYPE OF OUTLET	No OF GRATINGS 2	3
IN-LINE OUTLET :		
Channel full (Q_o) m^3/s	0.088	0.13
Surcharged (Q_s) m^3/s	0.028	0.046
OFF-LINE OUTLET :		
Channel full (Q_o) m^3/s	0.13	0.22
Surcharged (Q_s) m^3/s	0.16	0.22

TABLE 3 - Trapezoidal channel with cross-falls of 1:5
Limiting values of Q_o and Q_s for terminal outlets

TYPE OF OUTLET	No OF GRATINGS 2	3
IN-LINE OUTLET :		
Channel full (Q_o) m^3/s	0.077	0.11
Surcharged (Q_s) m^3/s	-	-
OFF-LINE OUTLET :		
Channel full (Q_o) m^3/s	0.11	0.18
Surcharged (Q_s) m^3/s	0.14	0.20

FIGURES

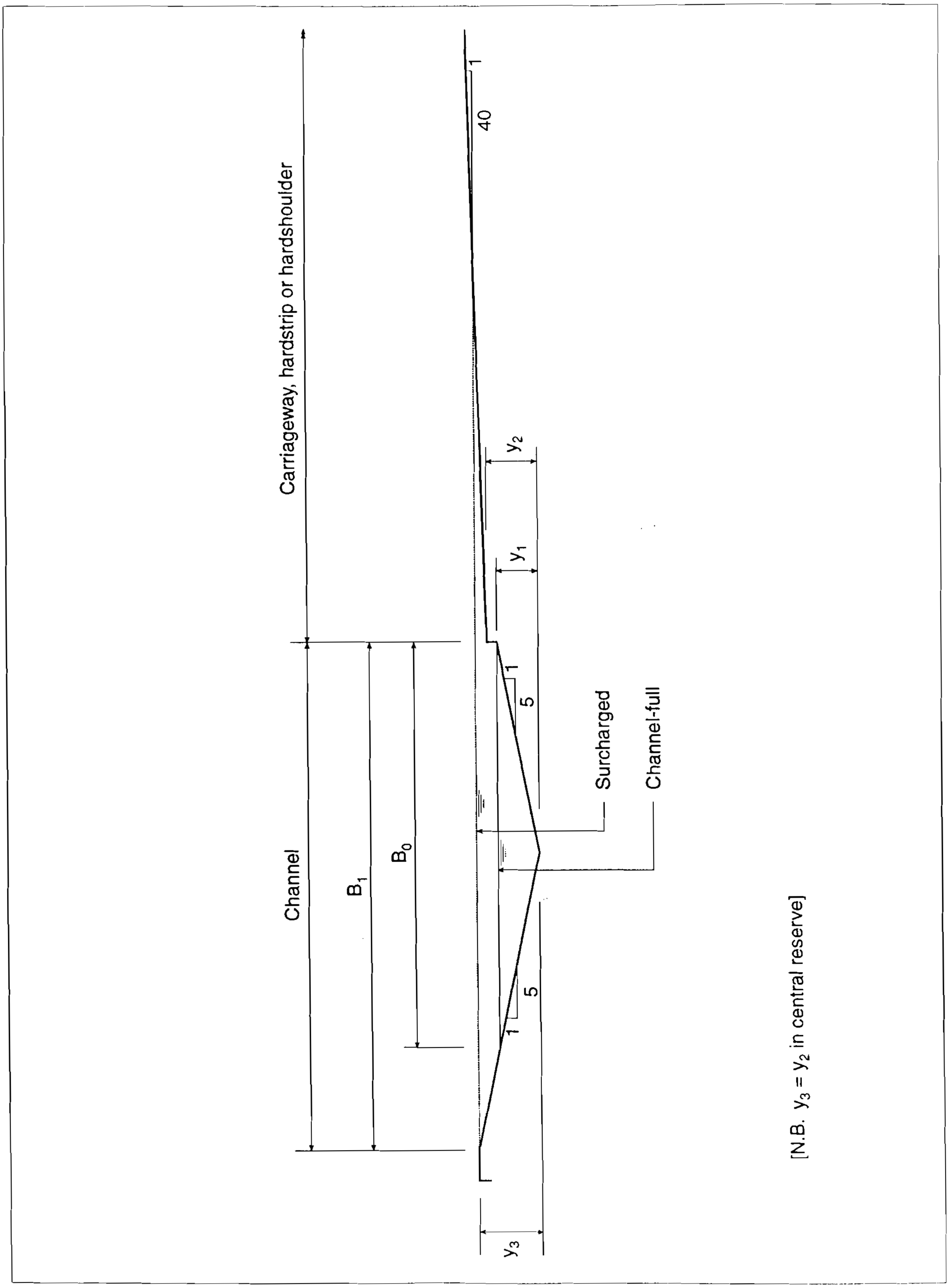


Figure 1 Cross-sectional shape of triangular channel

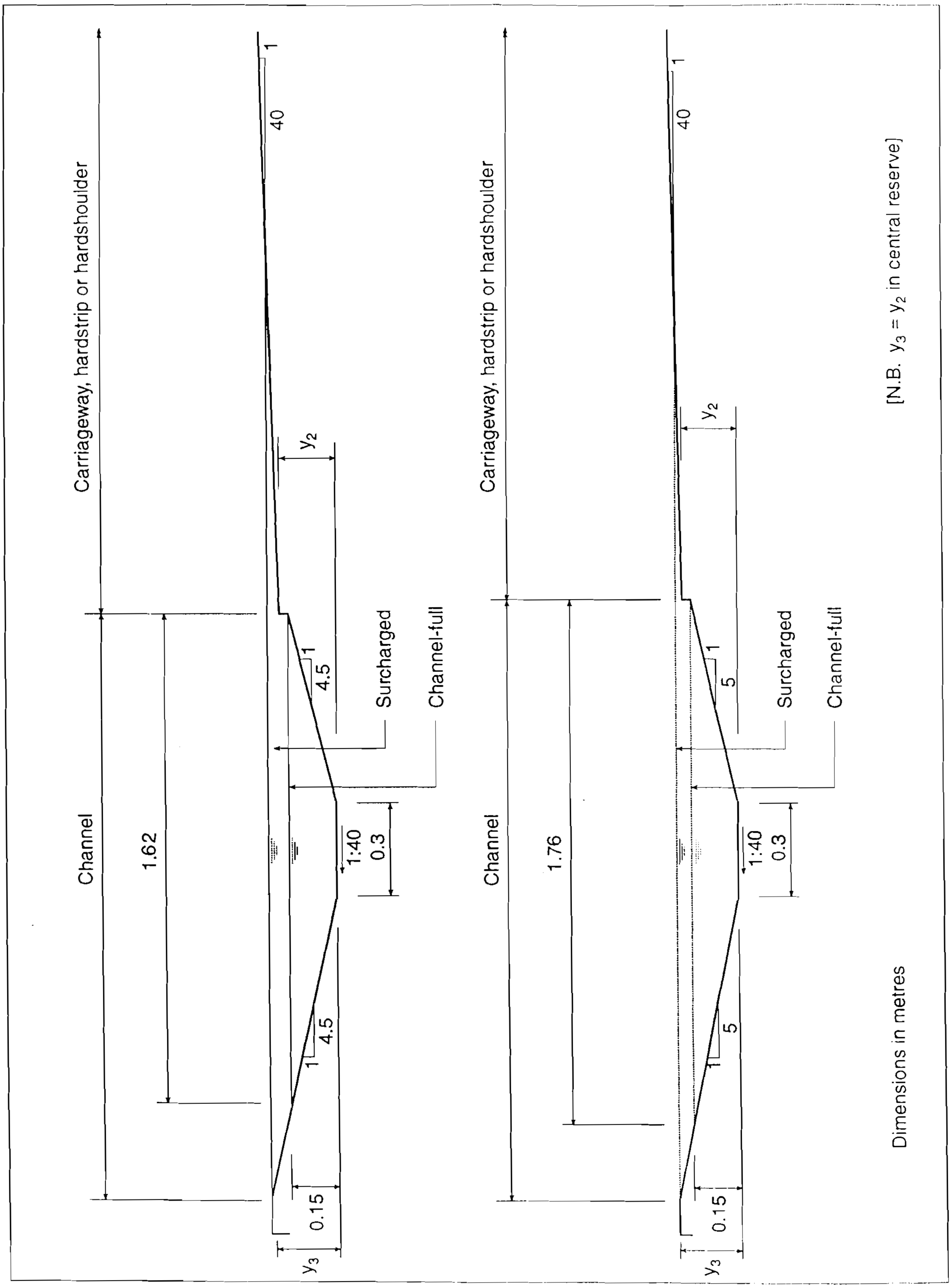


Figure 2 Cross-sectional shape of trapezoidal channels

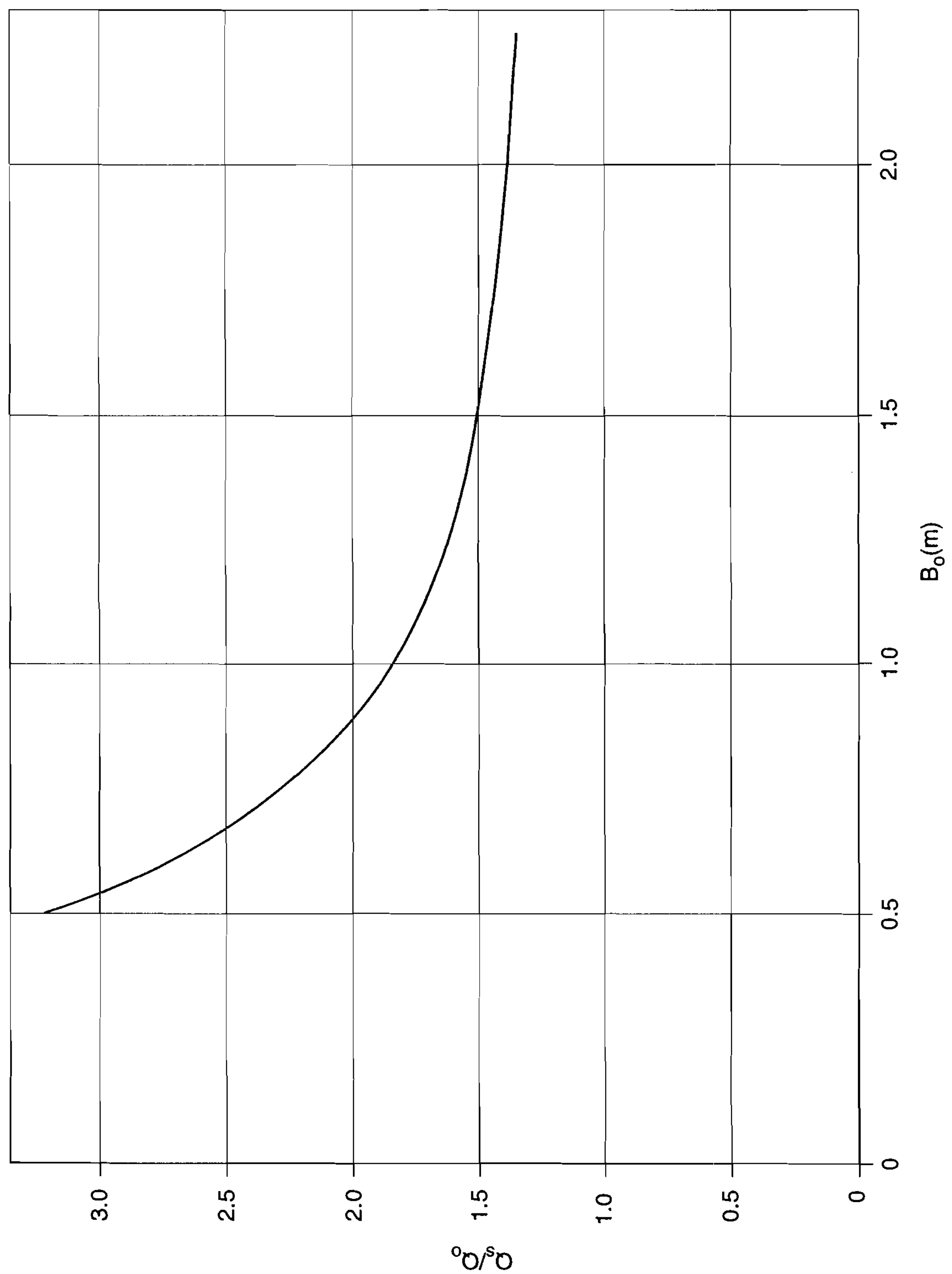


Figure 3 Relationship between surcharged and channel-full flows Triangular channel

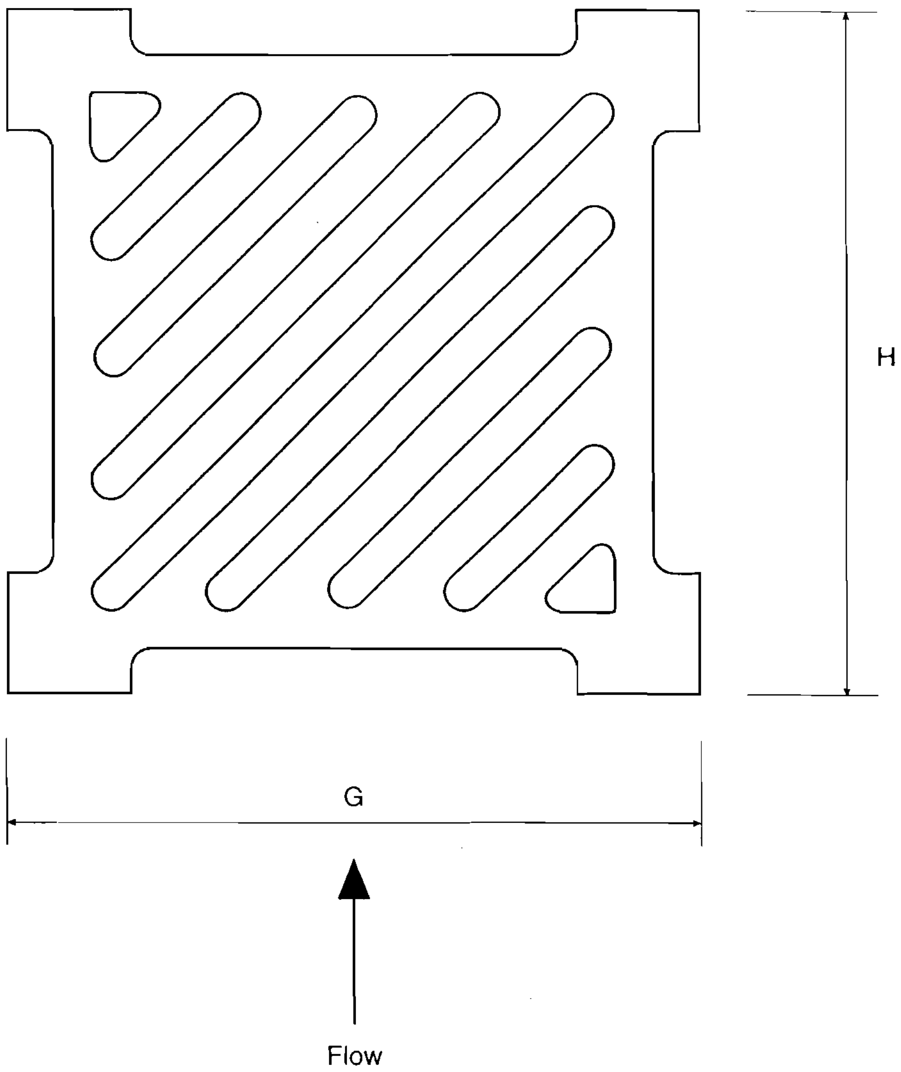


Figure 4 Typical bar pattern of gratings

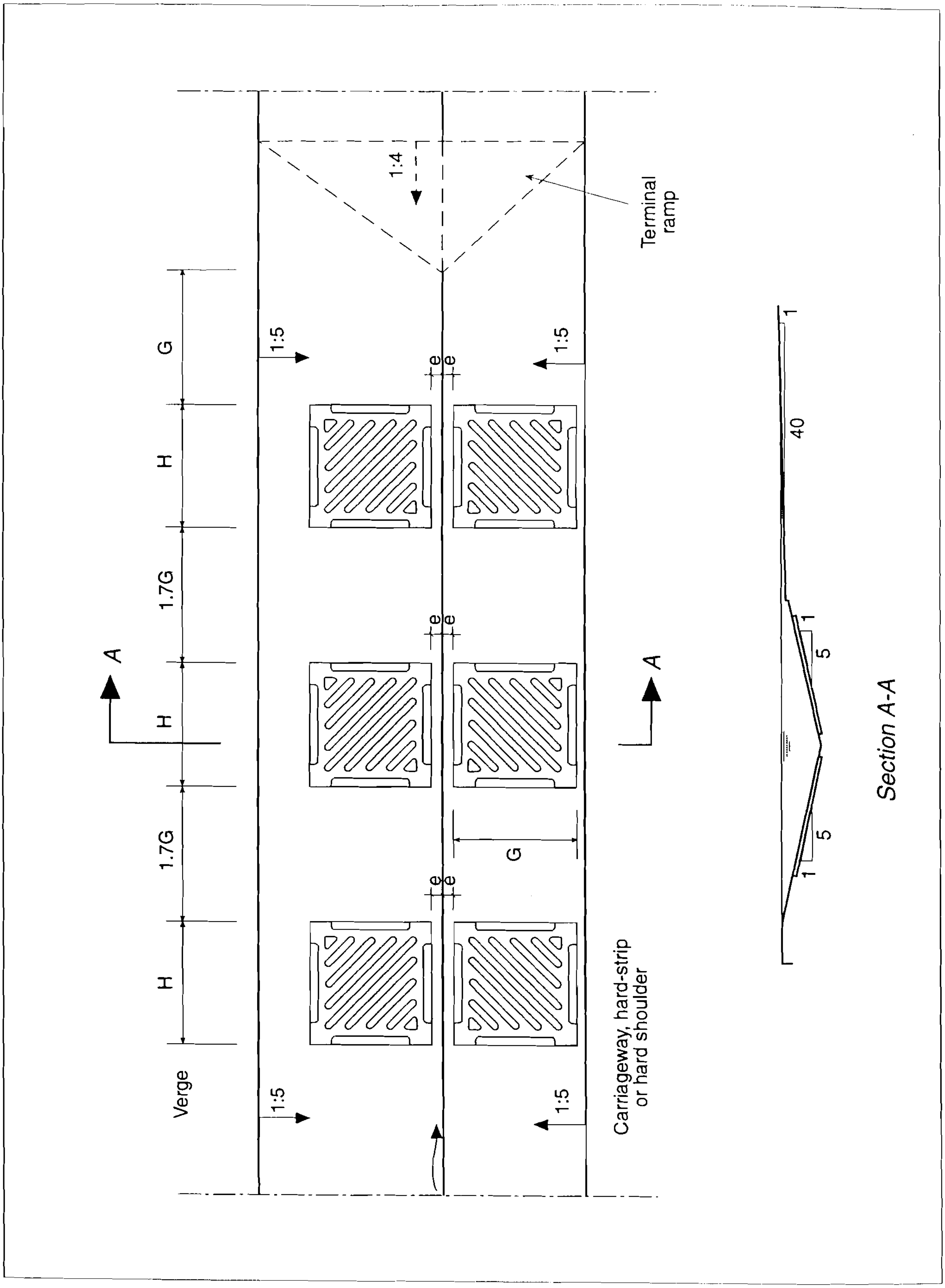


Figure 5 Triangular channel In-line outlet

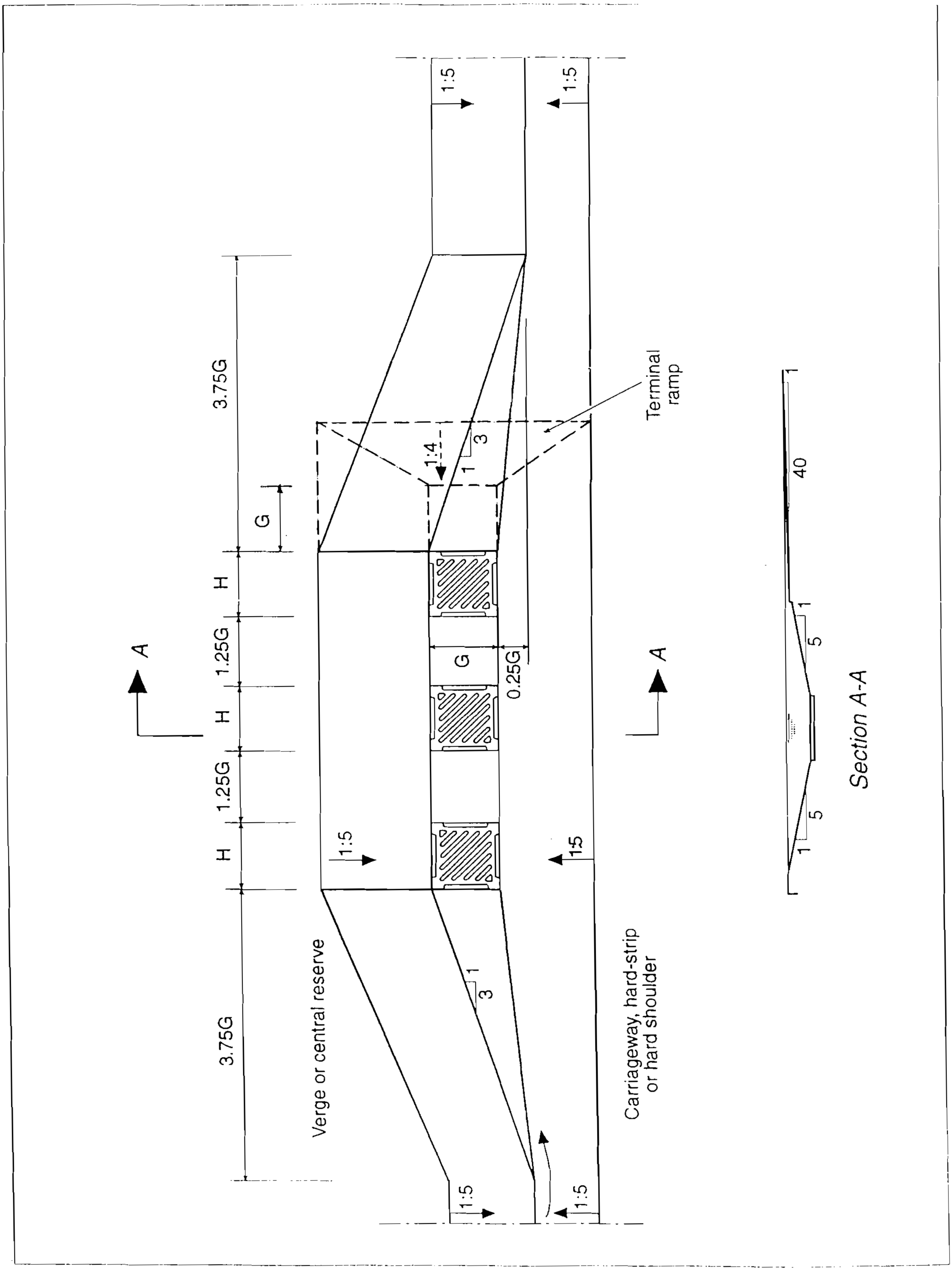


Figure 6 Triangular channel Off-line outlet

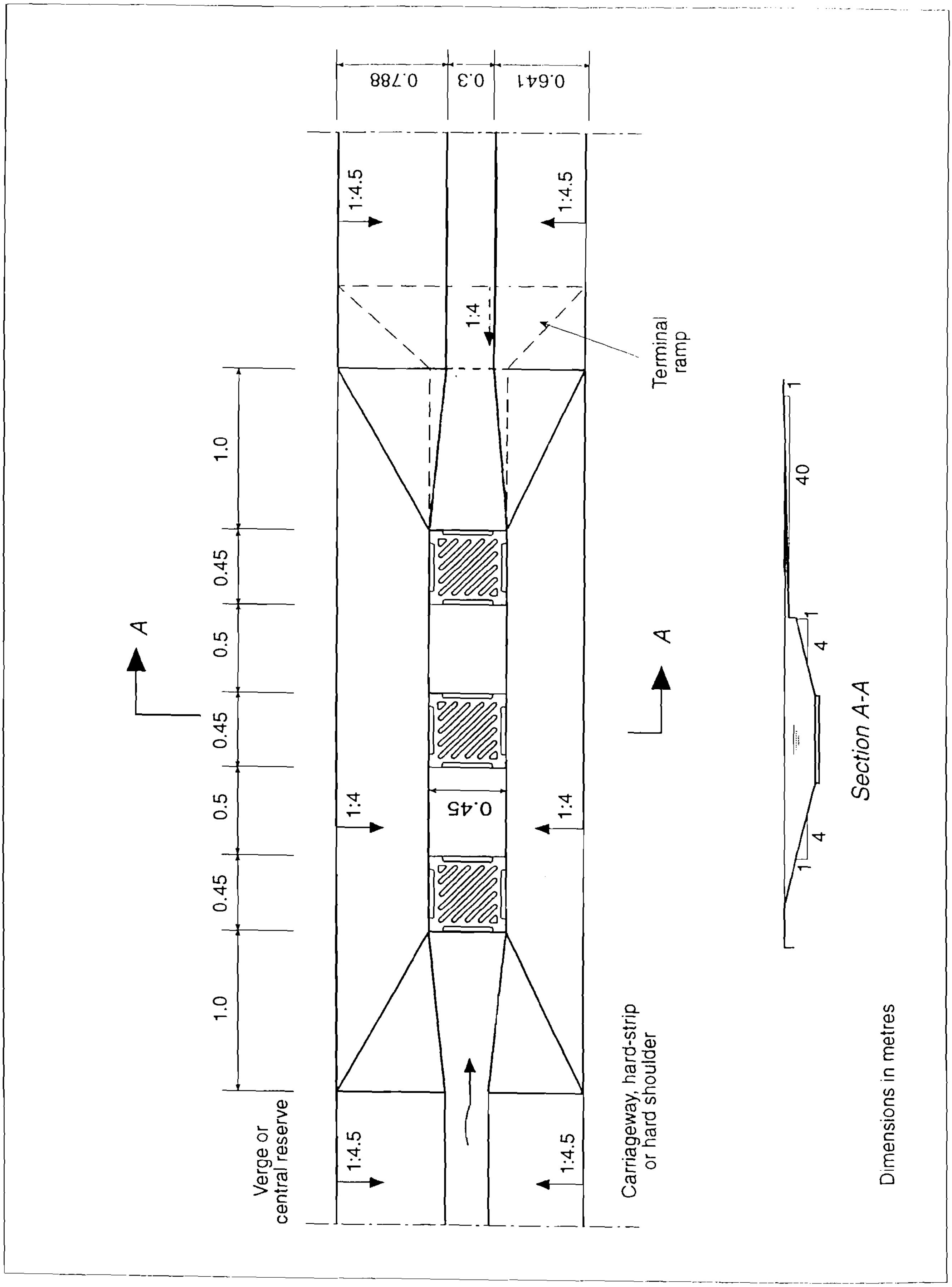


Figure 7 Trapezoidal channel with cross-falls of 1:4.5 In-line outlet

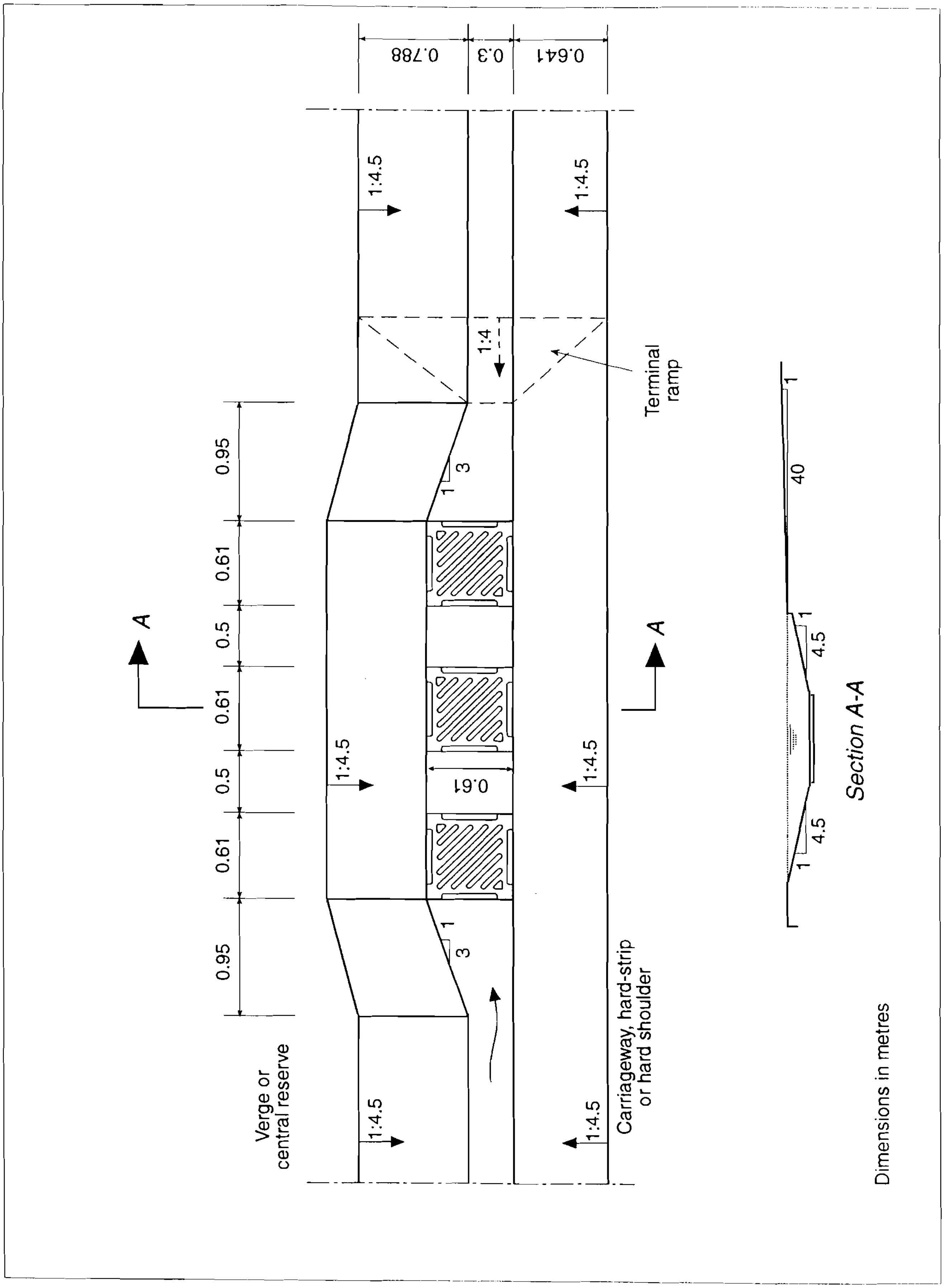


Figure 8 Trapezoidal channel with cross-falls of 1:4.5 - Off-line outlet

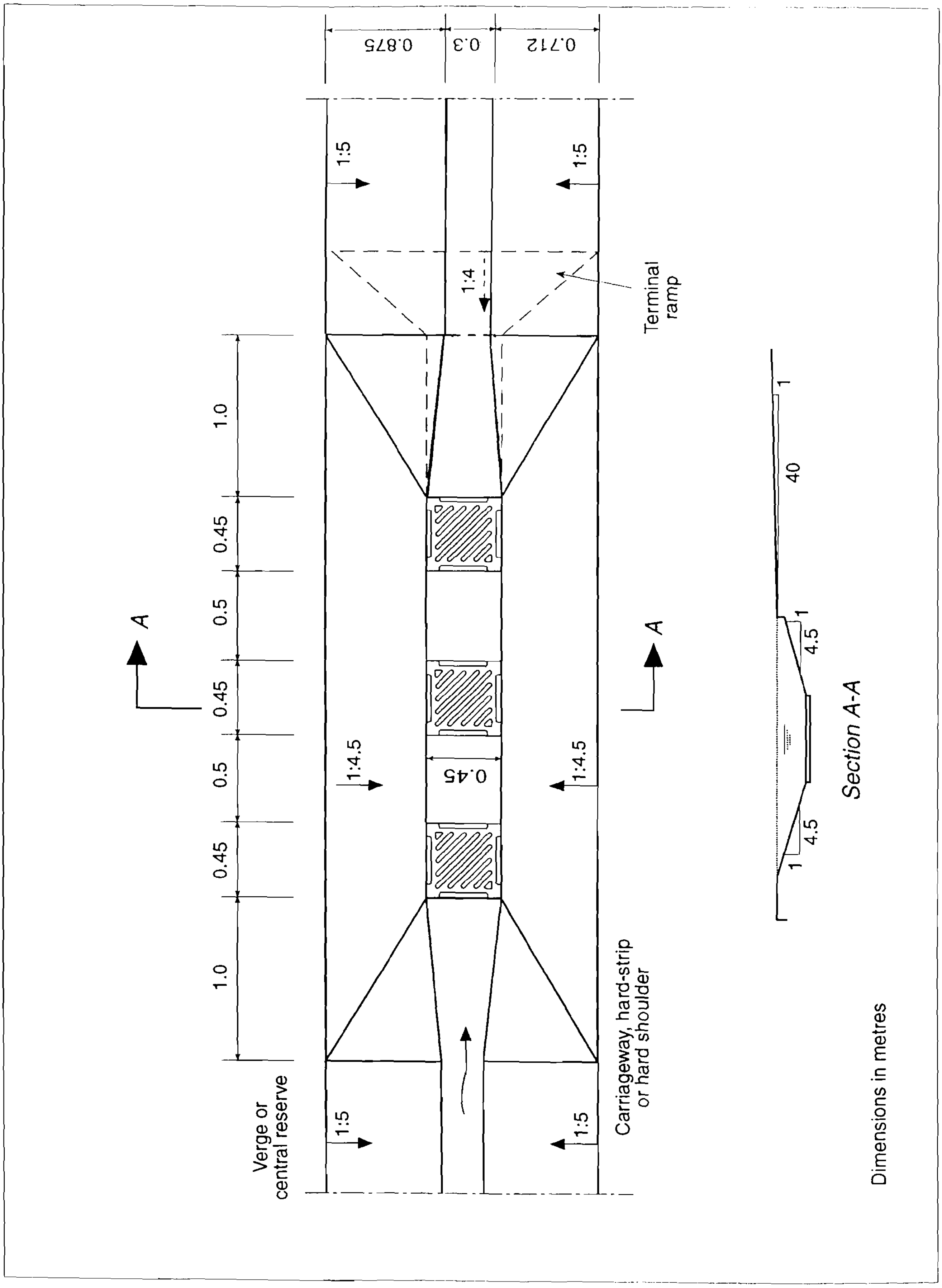


Figure 9 Trapezoidal channel with cross-falls of 1:5 In-line outlet

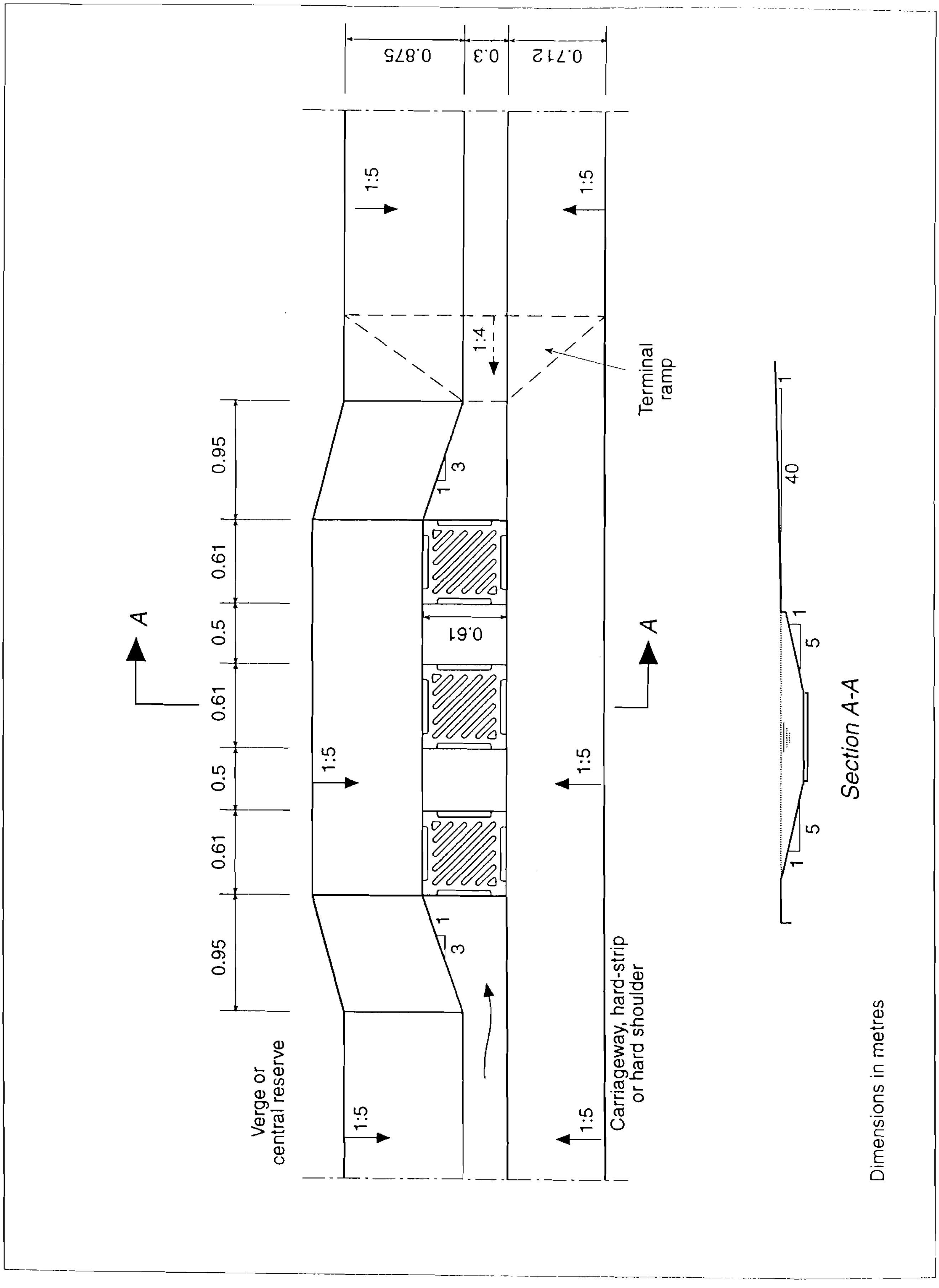


Figure 10 Trapezoidal channel with cross-falls of 1:5
Off-line outlet

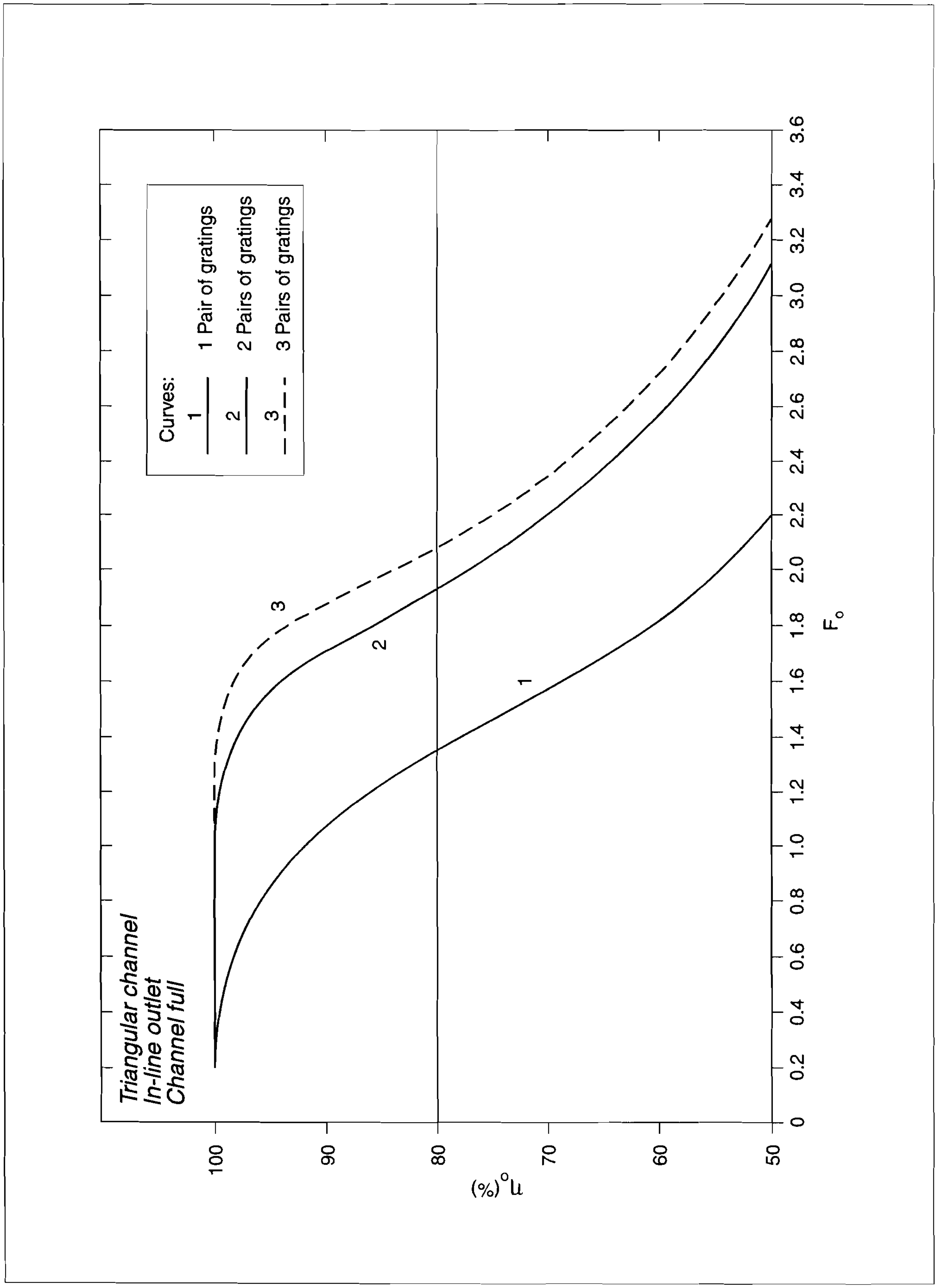


Figure 11 Design curves. Triangular channel - In-line outlet Channel full

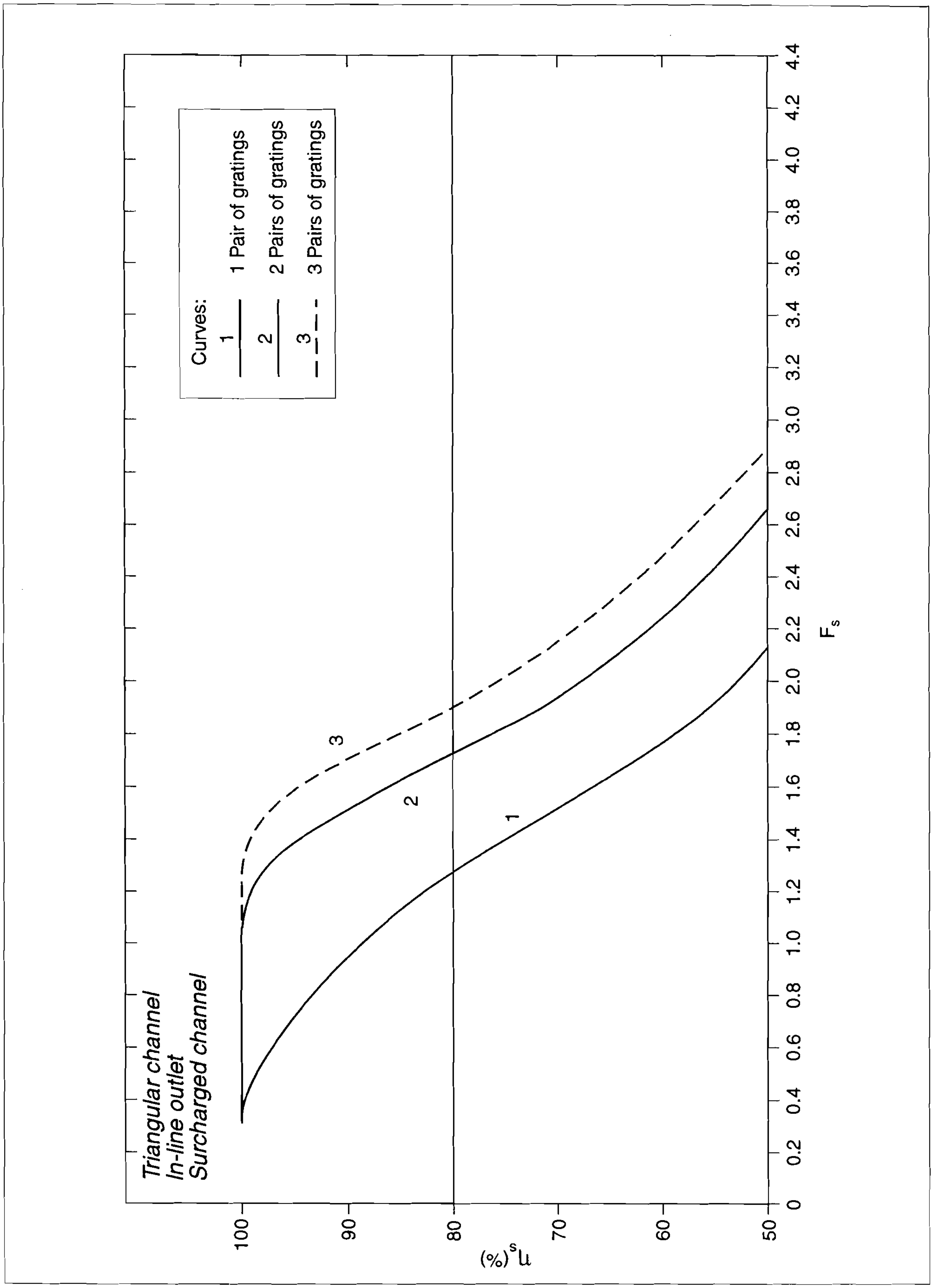


Figure 12 Design curves. Triangular channel - In-line outlet Surcharged channel

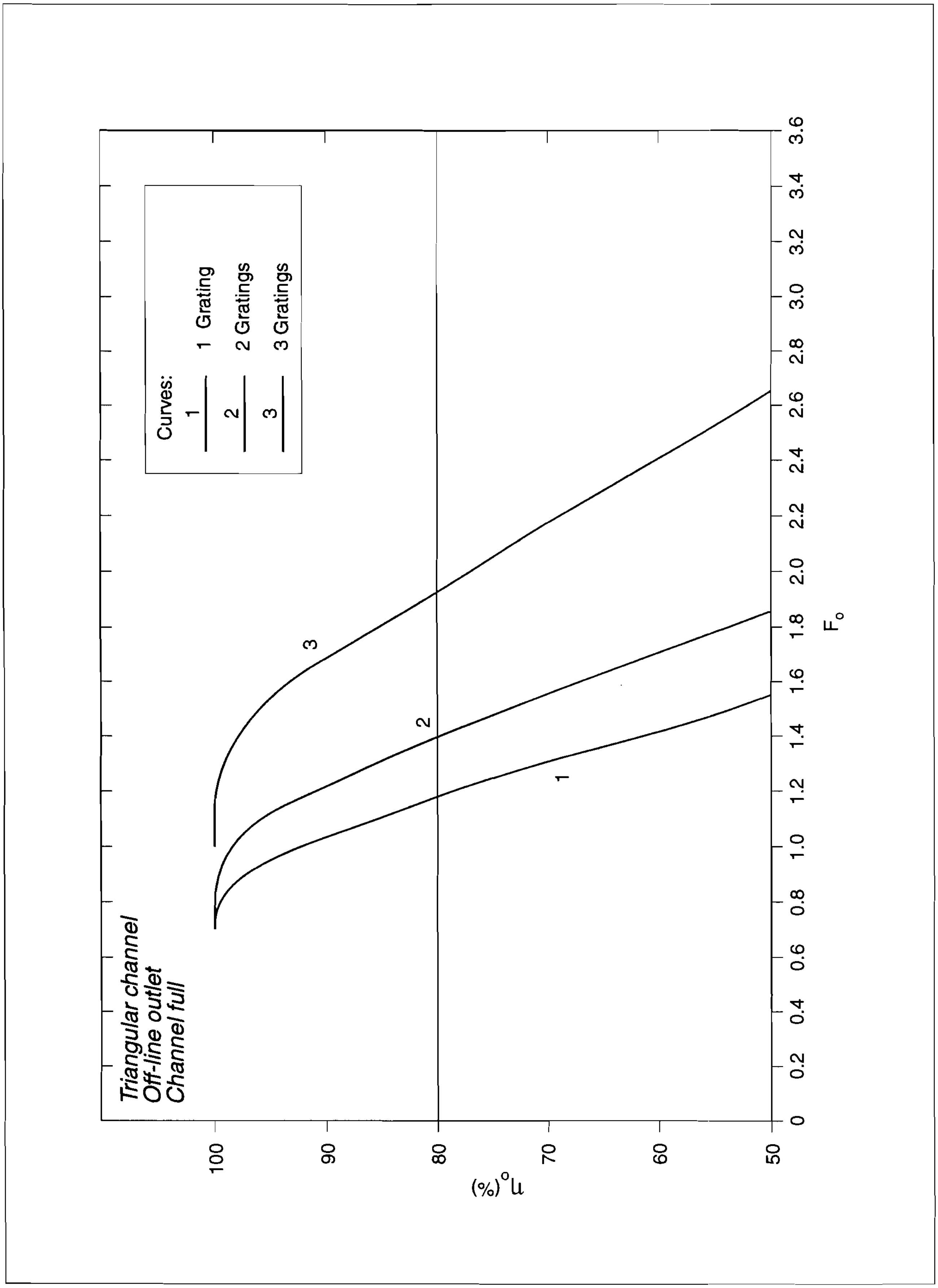


Figure 13 Design curves. Triangular channel - Off-line outlet Channel full

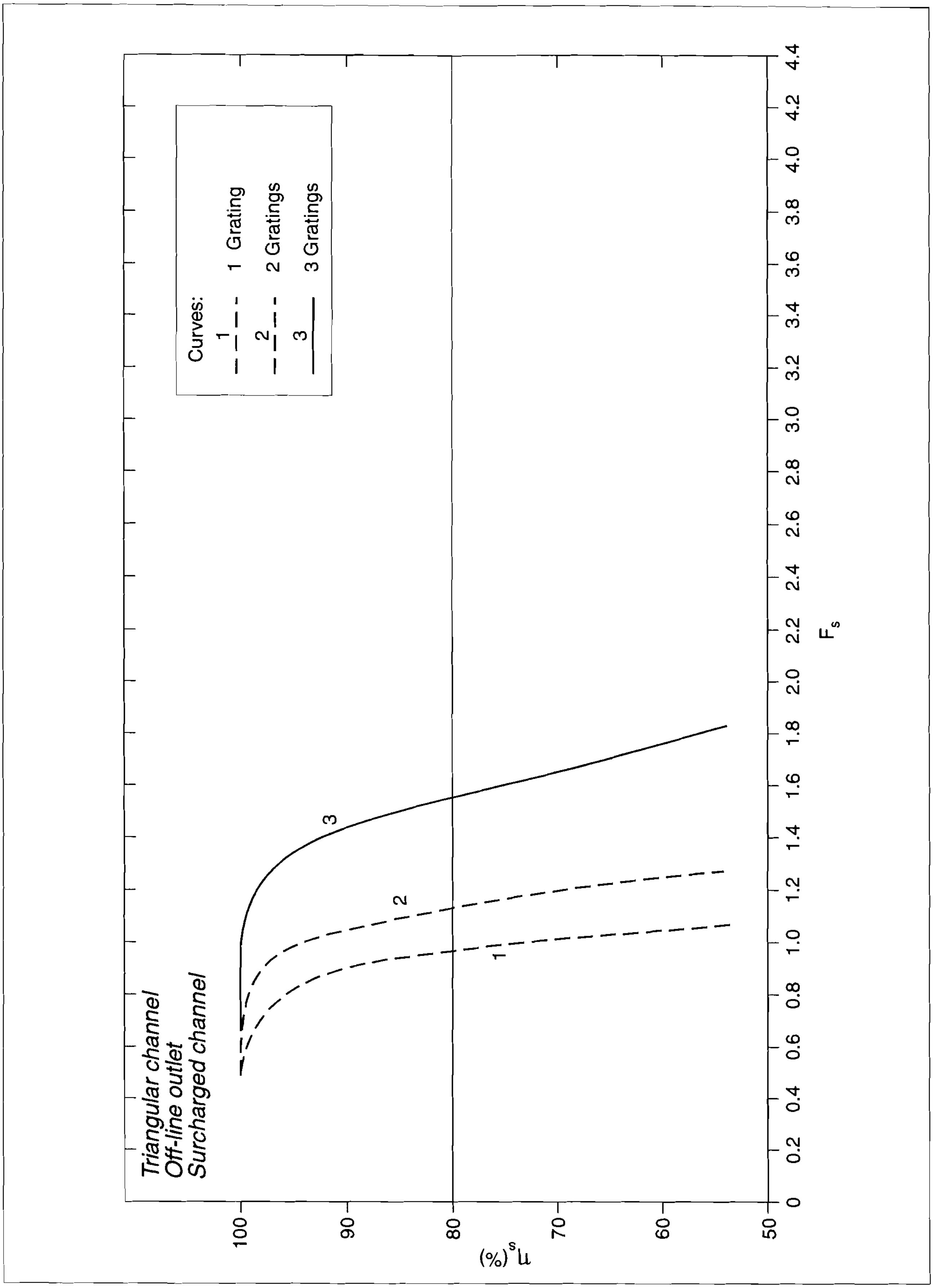


Figure 14 Design curves. Triangular channel - Off-line outlet Surcharged channel

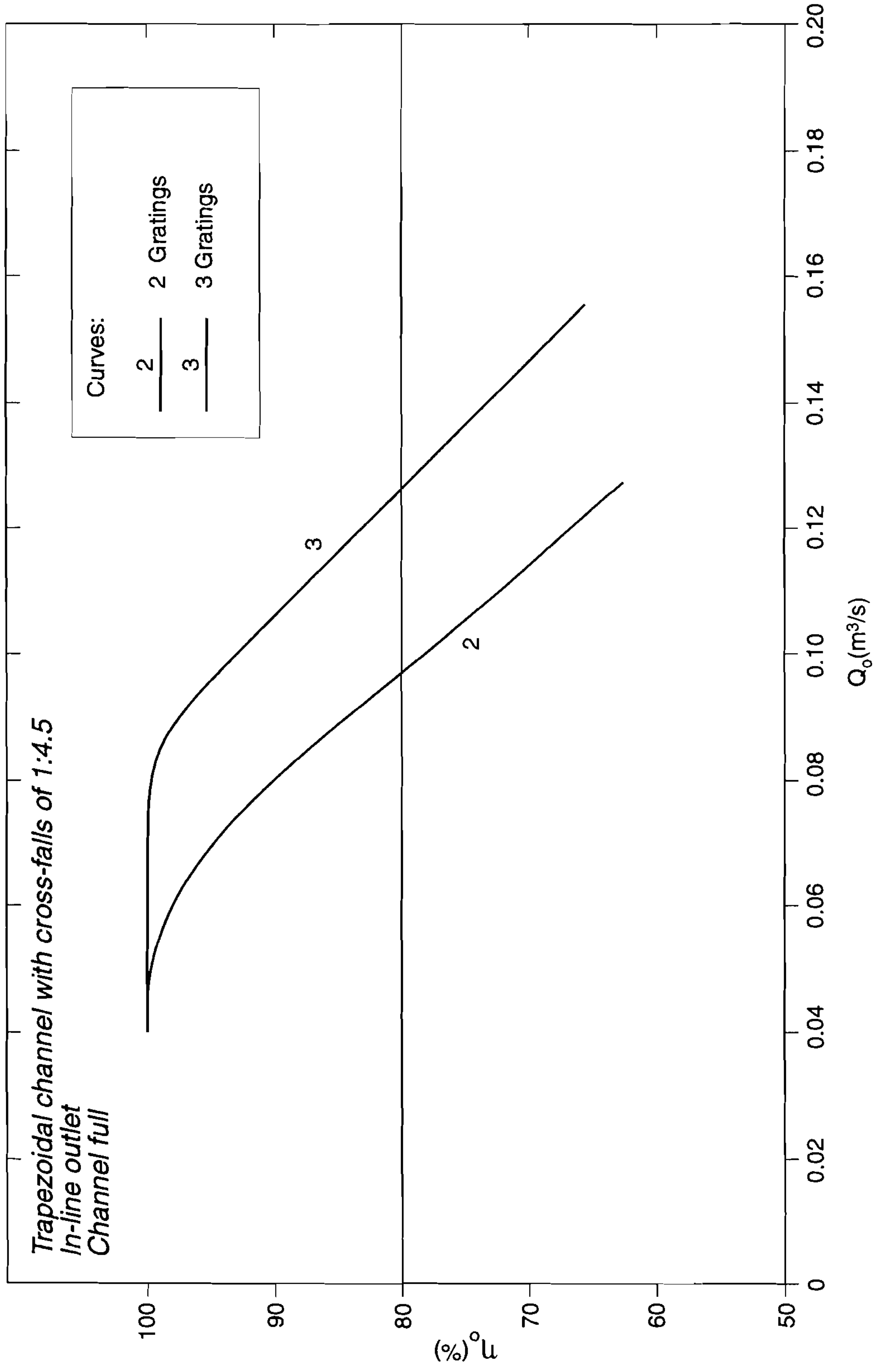


Figure 15 Design curves. Trapezoidal channel with cross-falls of 1:4.5 In-line outlet. Channel full

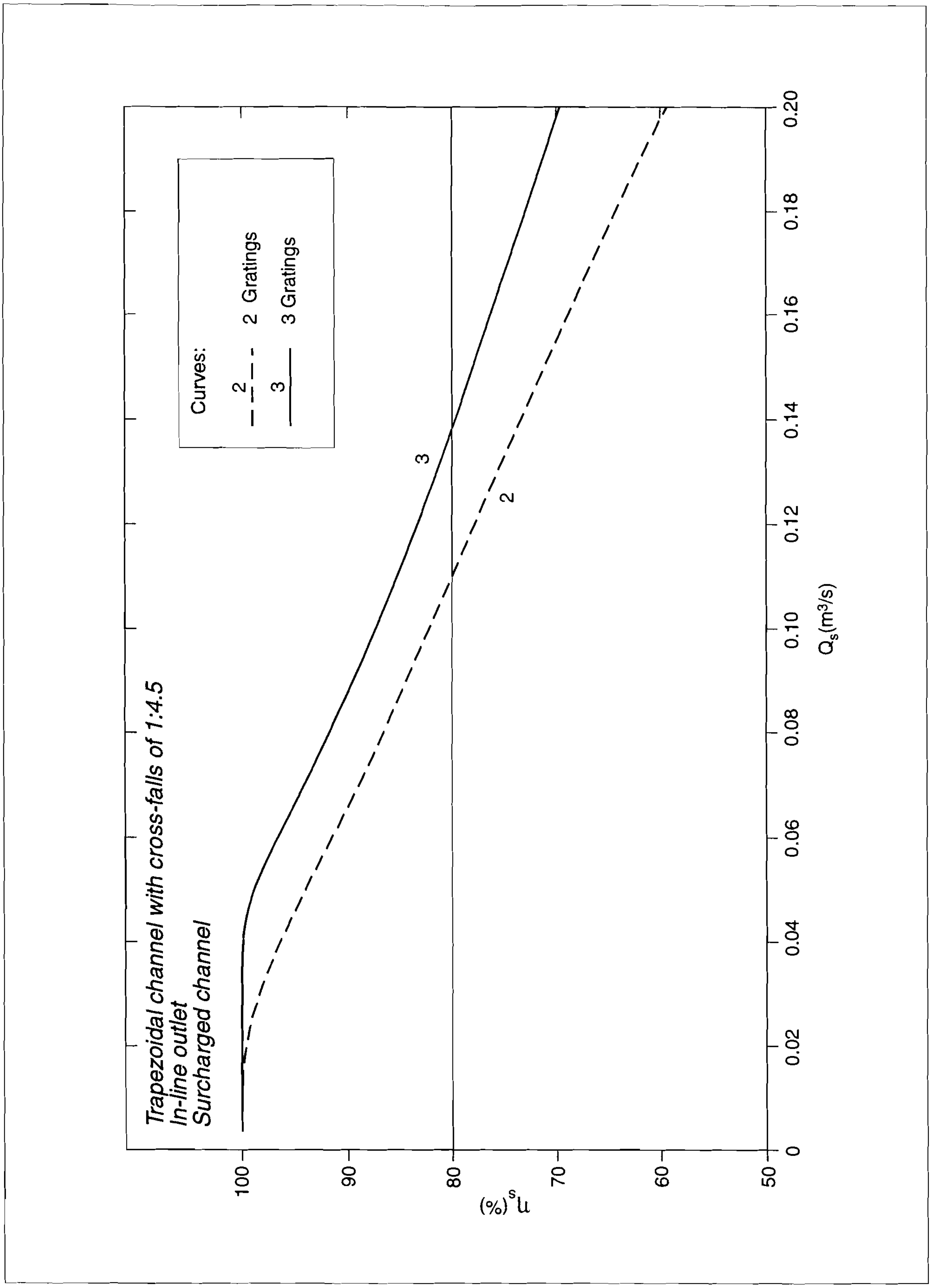


Figure 16 Design curves. Trapezoidal channel with cross-falls of 1:4.5 In-line outlet. Surcharged channel

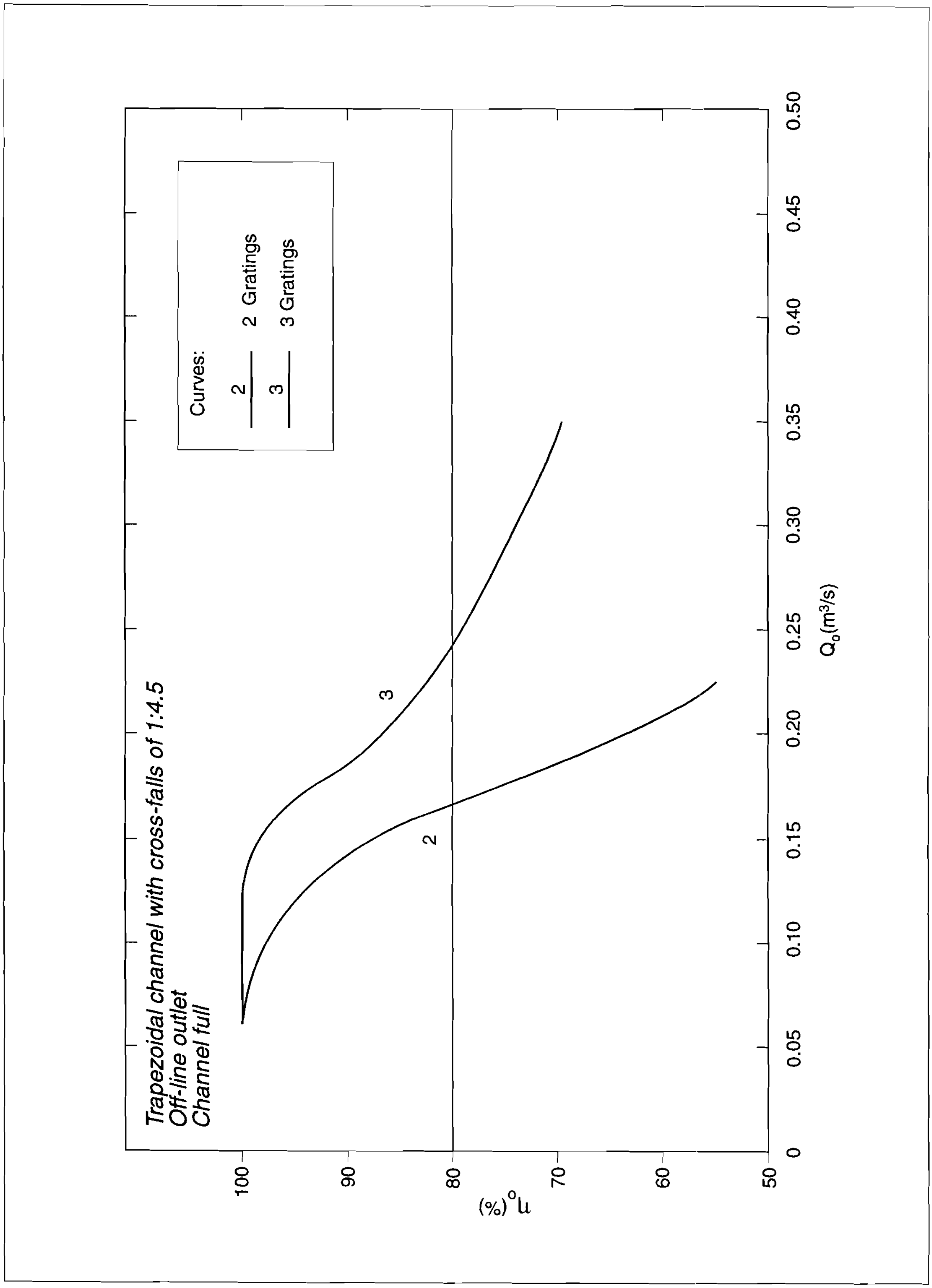


Figure 17 Design curves. Trapezoidal channel with cross-falls of 1:4.5 Off-line outlet. Channel full

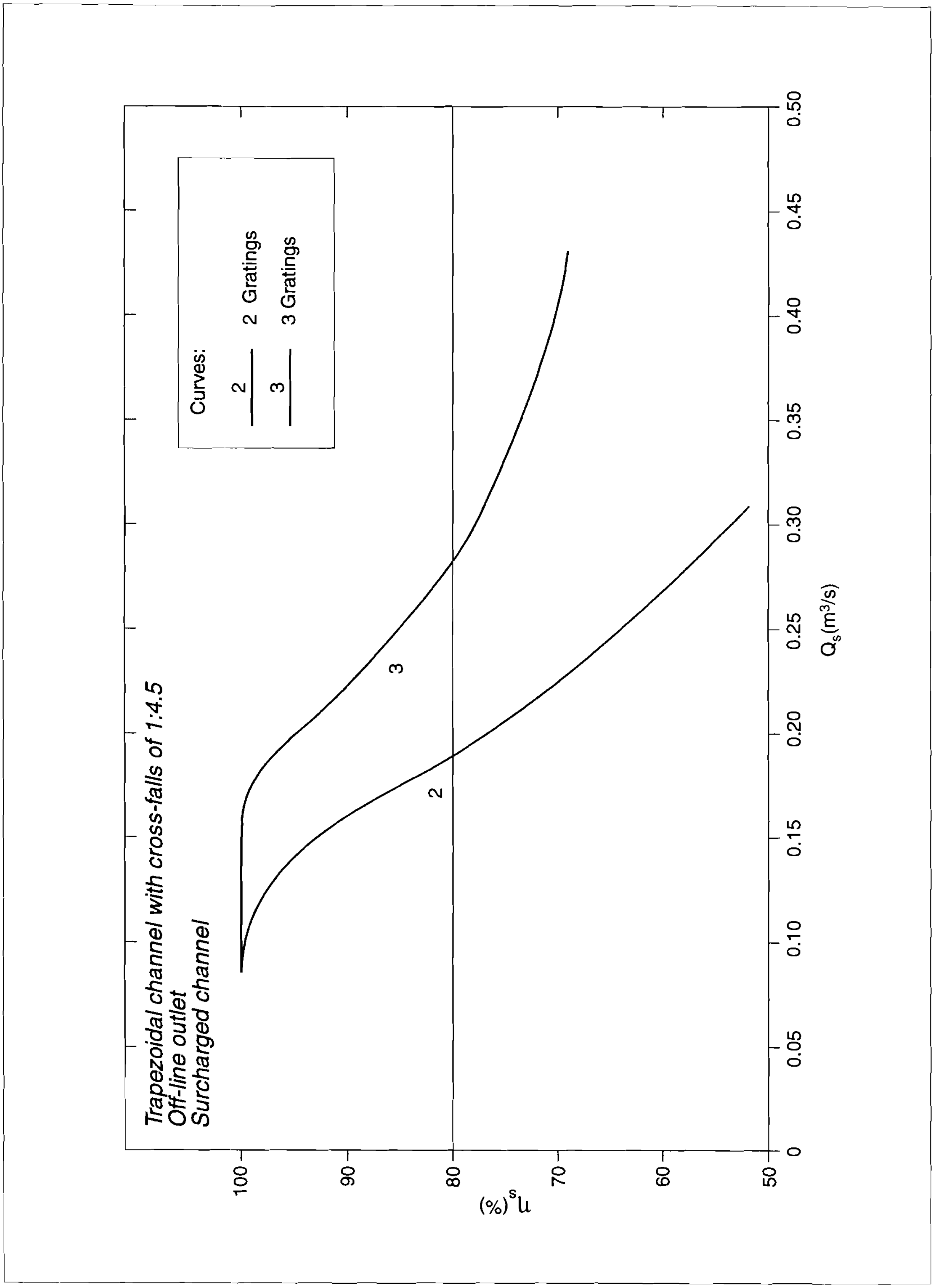


Figure 18 Design curves. Trapezoidal channel with cross-falls of 1:4.5 Off-line outlet. Surcharged channel

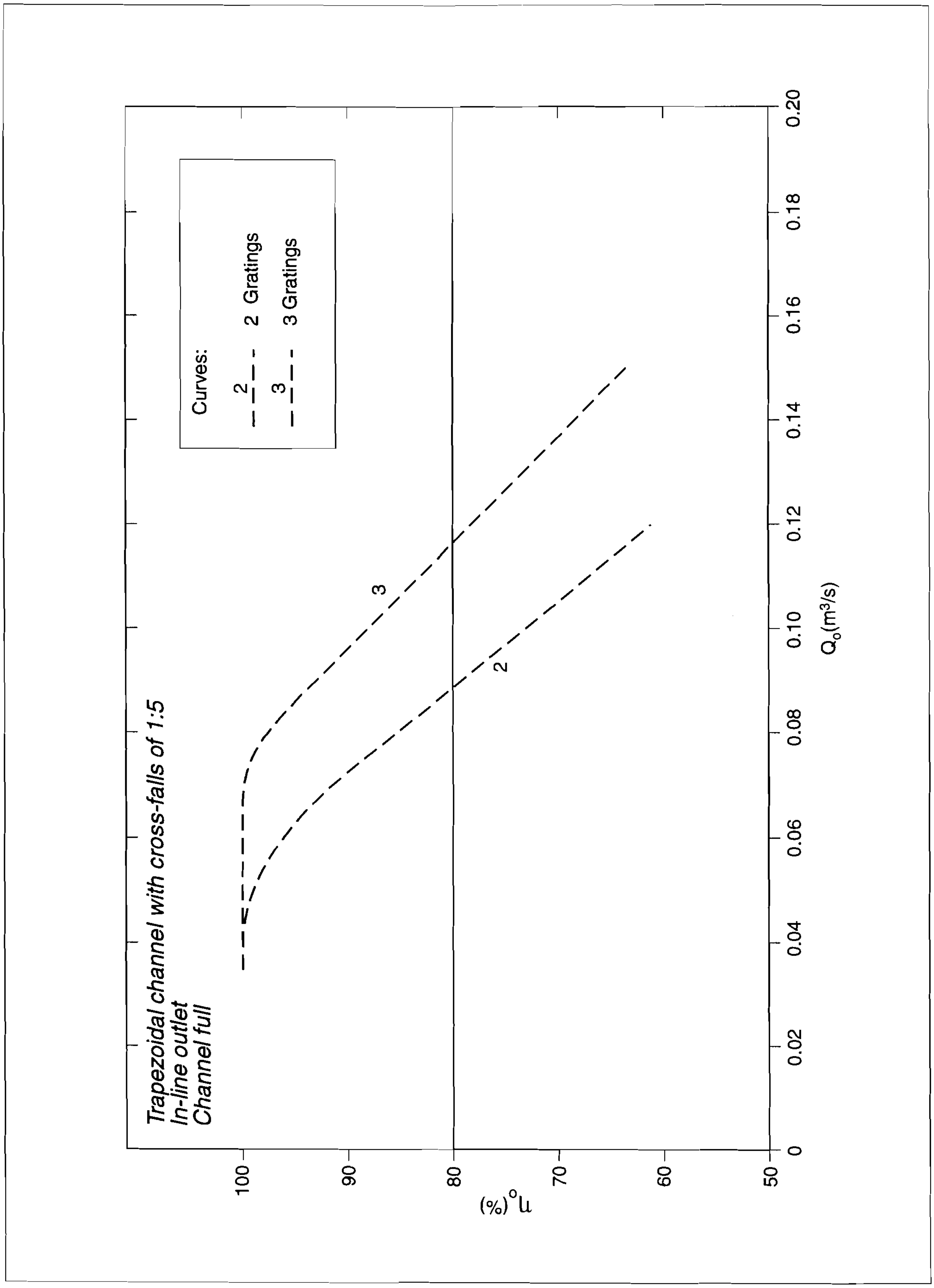


Figure 19 Design curves. Trapezoidal channel with cross-falls of 1:5 In-line outlet. Channel full

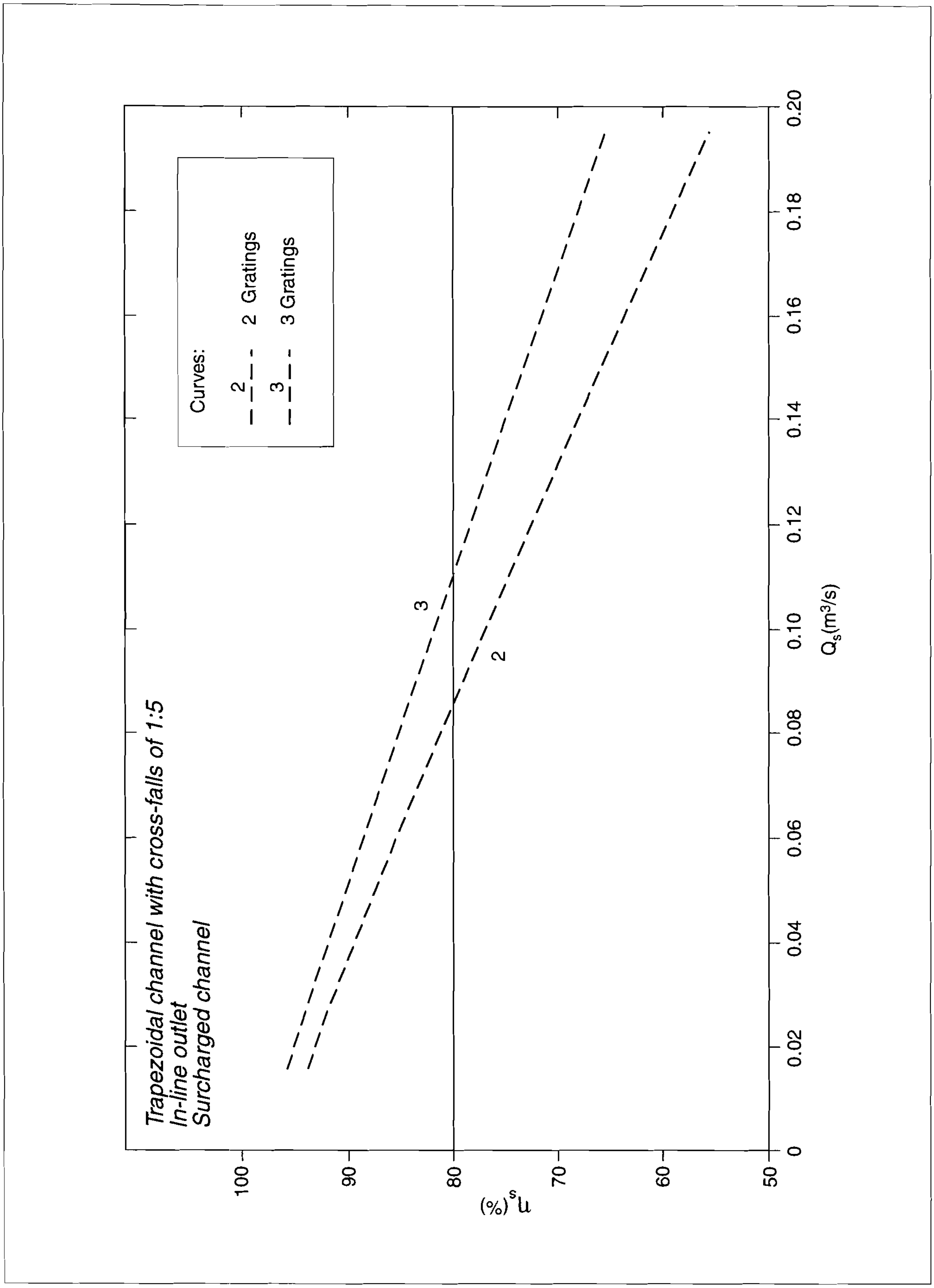


Figure 20 Design curves. Trapezoidal channel with cross-falls of 1:5 In-line outlet. Surcharged channel

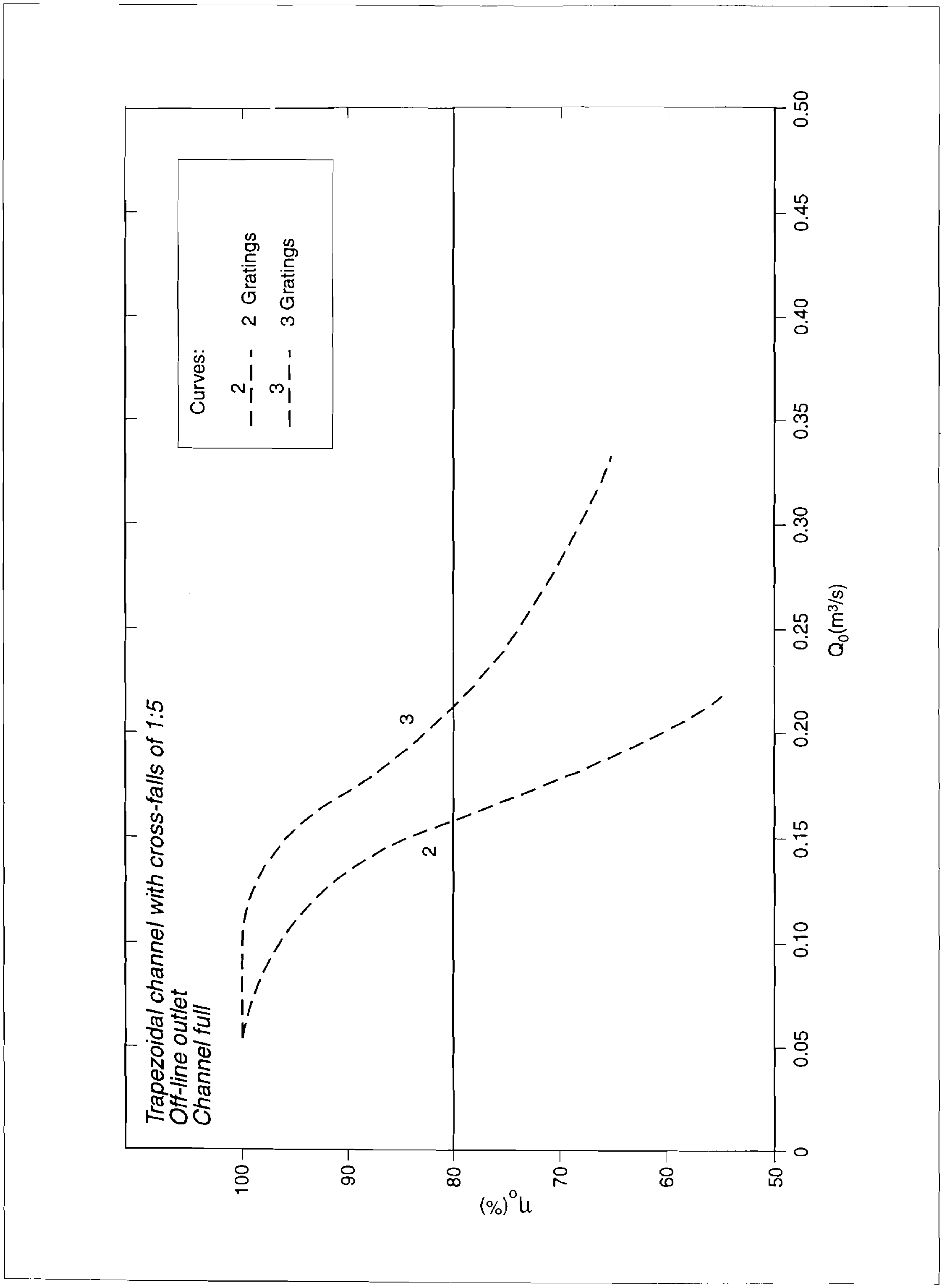


Figure 21 Design curves. Trapezoidal channel with cross-falls of 1:5 Off-line outlet. Channel full

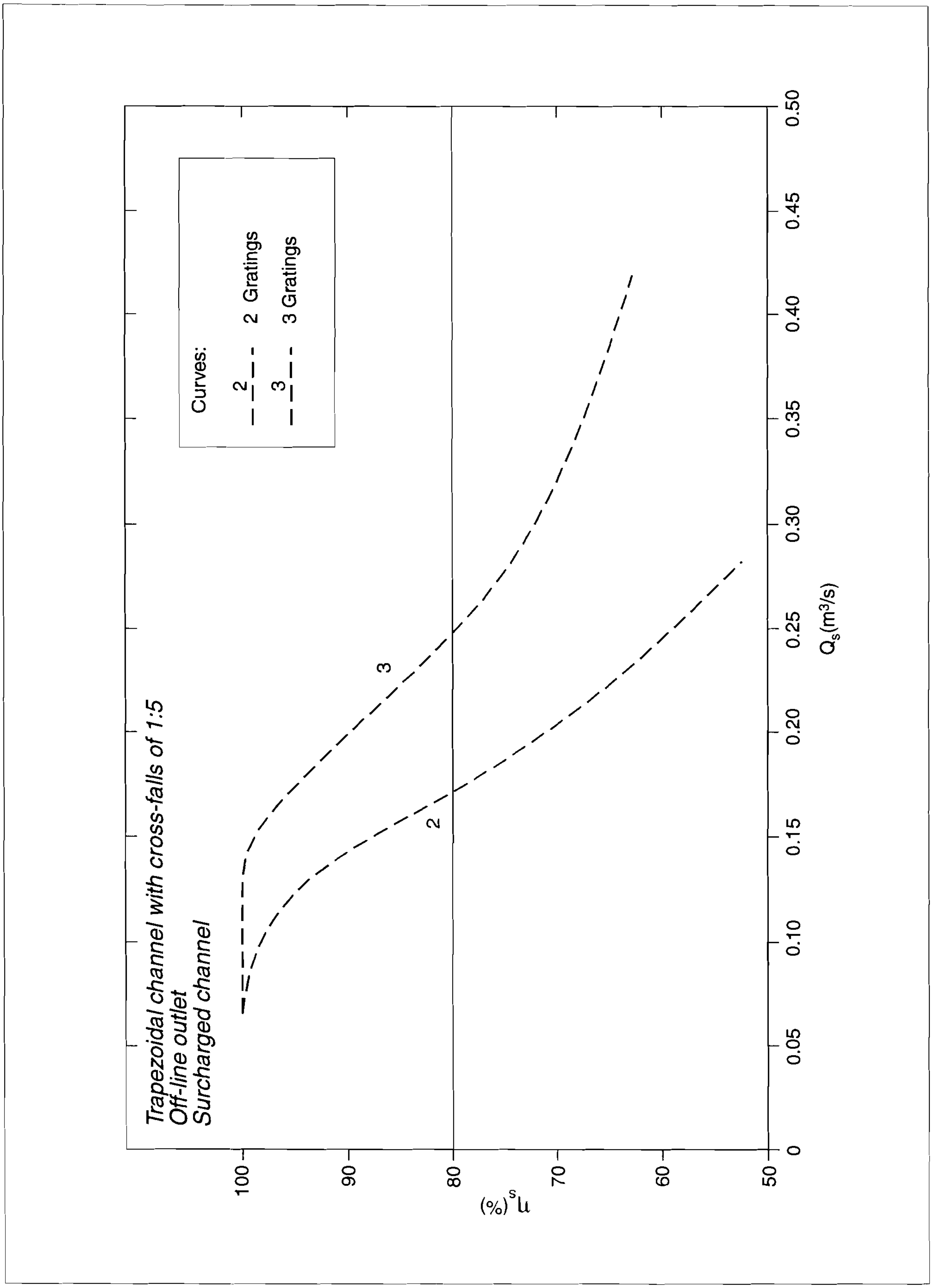


Figure 22 Design curves. Trapezoidal channel with cross-falls of 1:5 Off-line outlet. Surcharged channel

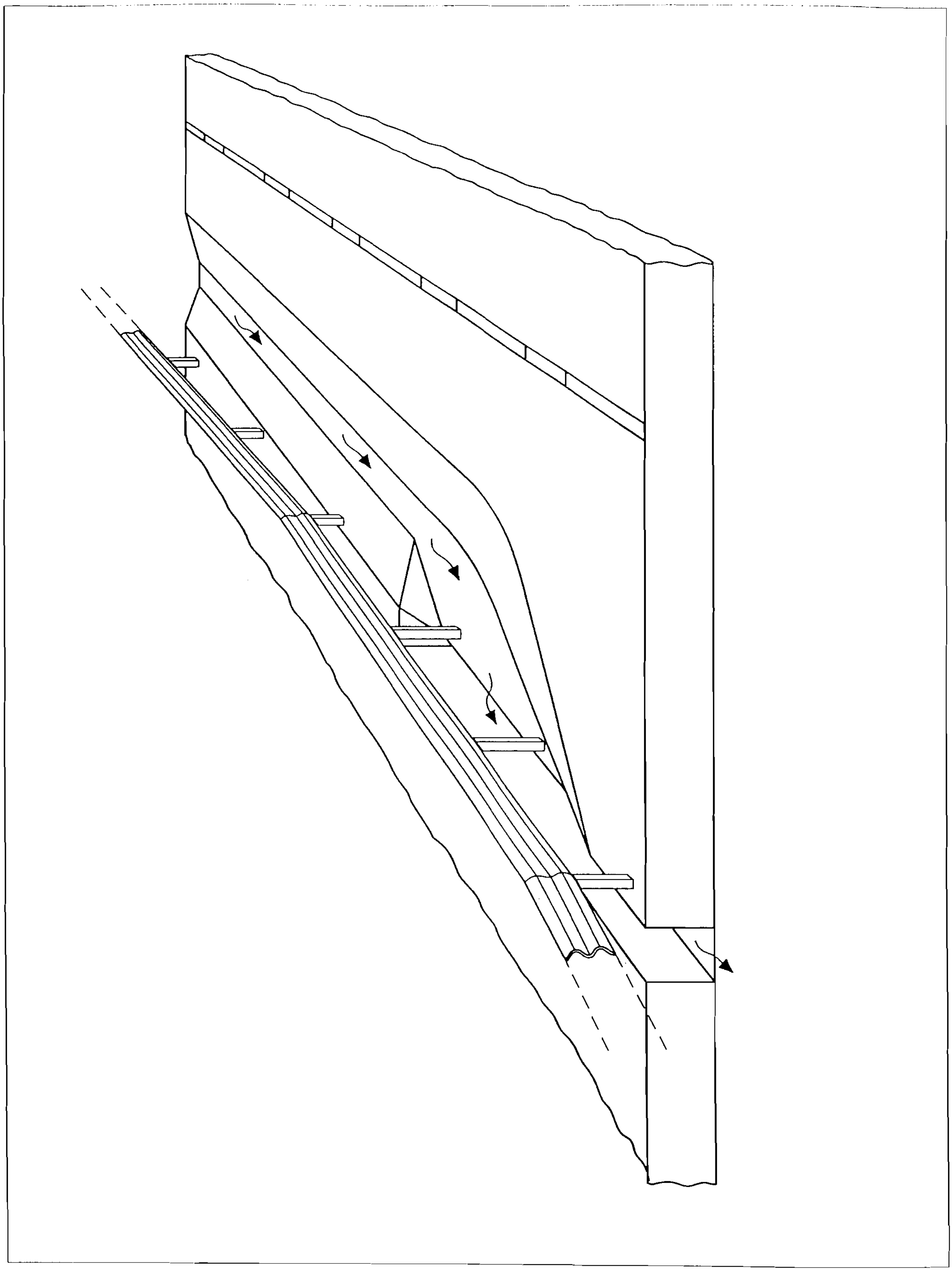


Figure 23 Isometric view of weir outlet indicating possible location of safety fence

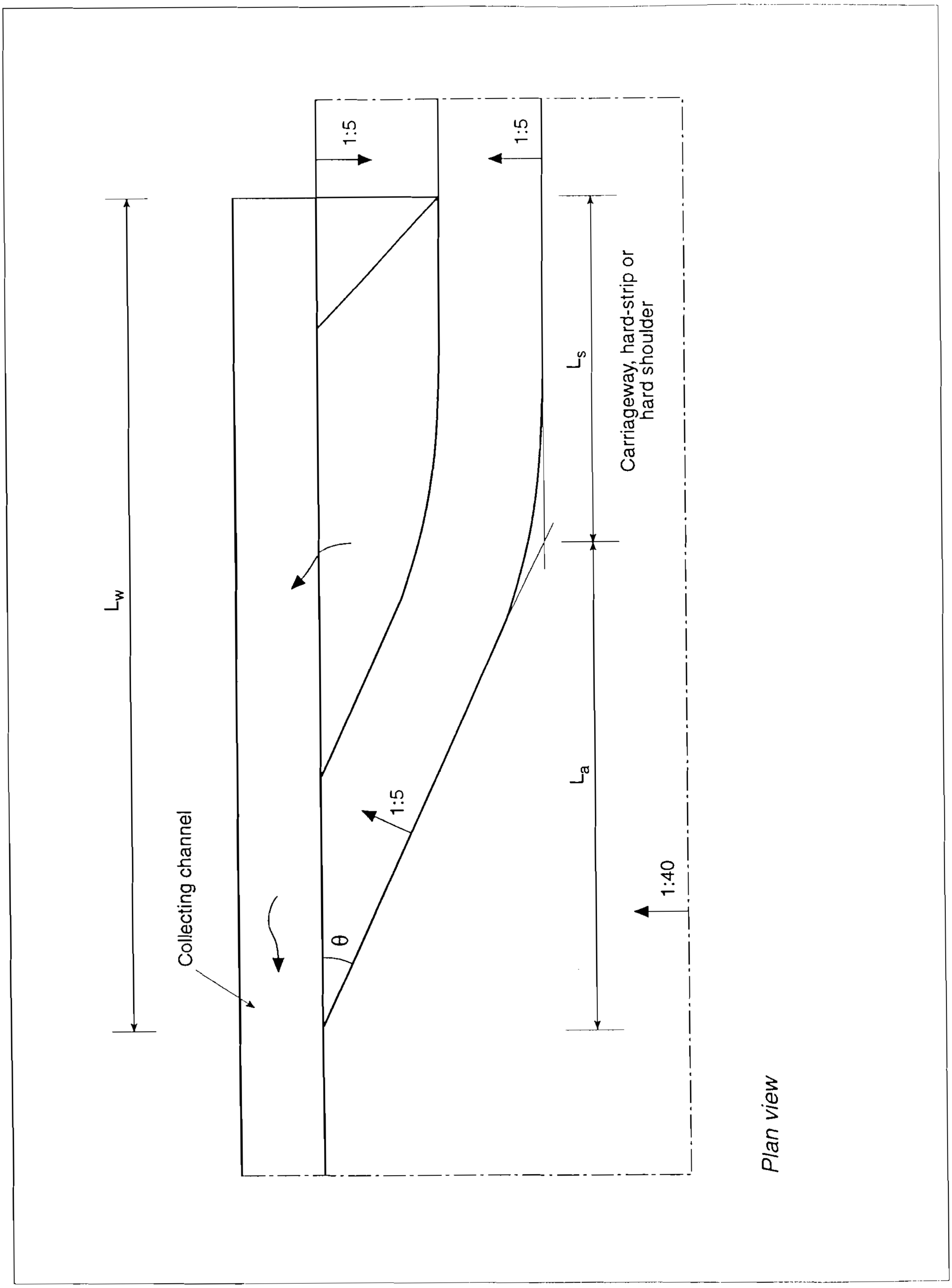


Figure 24 Example of triangular channel with weir outlet

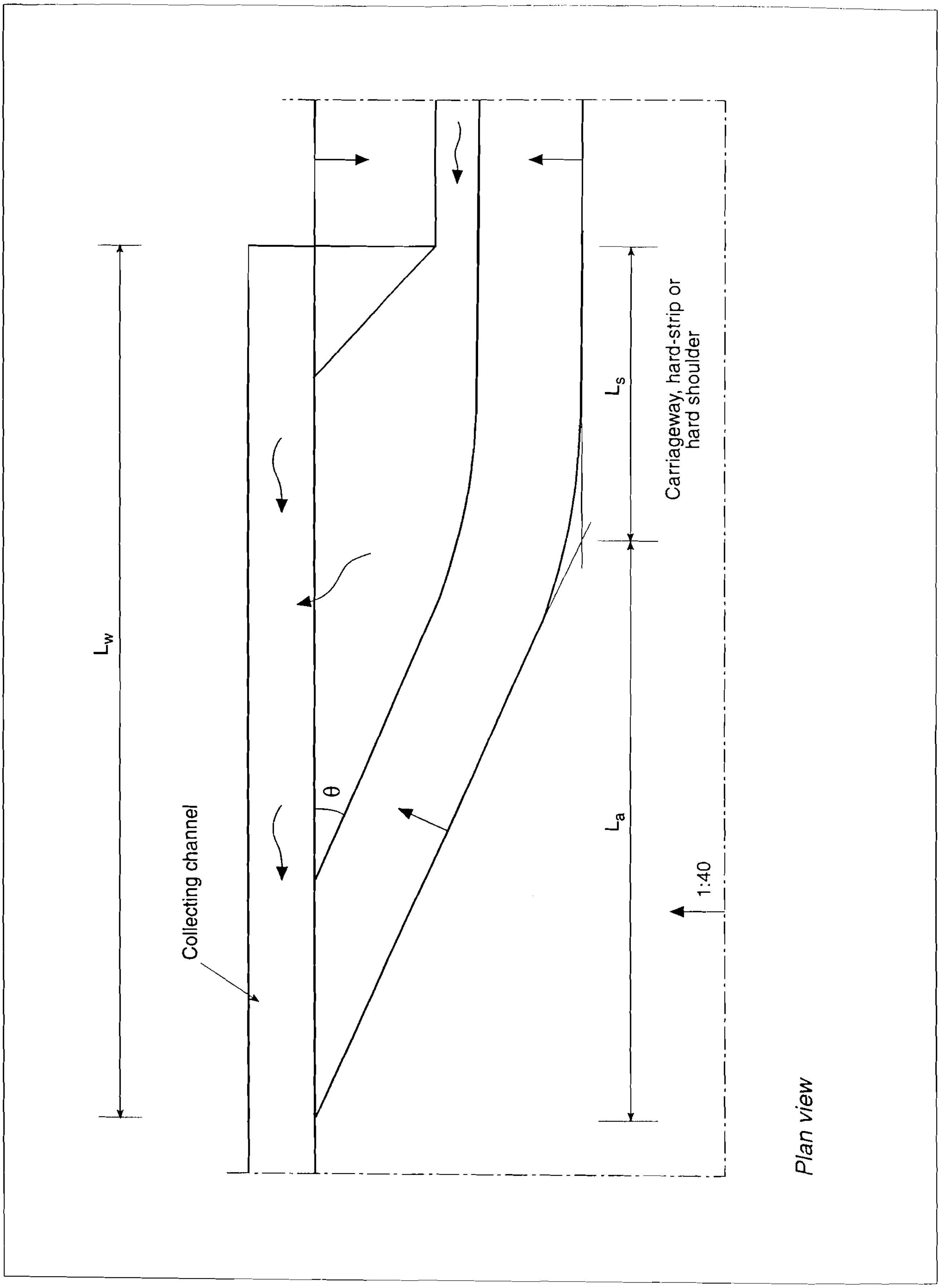


Figure 25 Example of trapezoidal channel with weir outlet

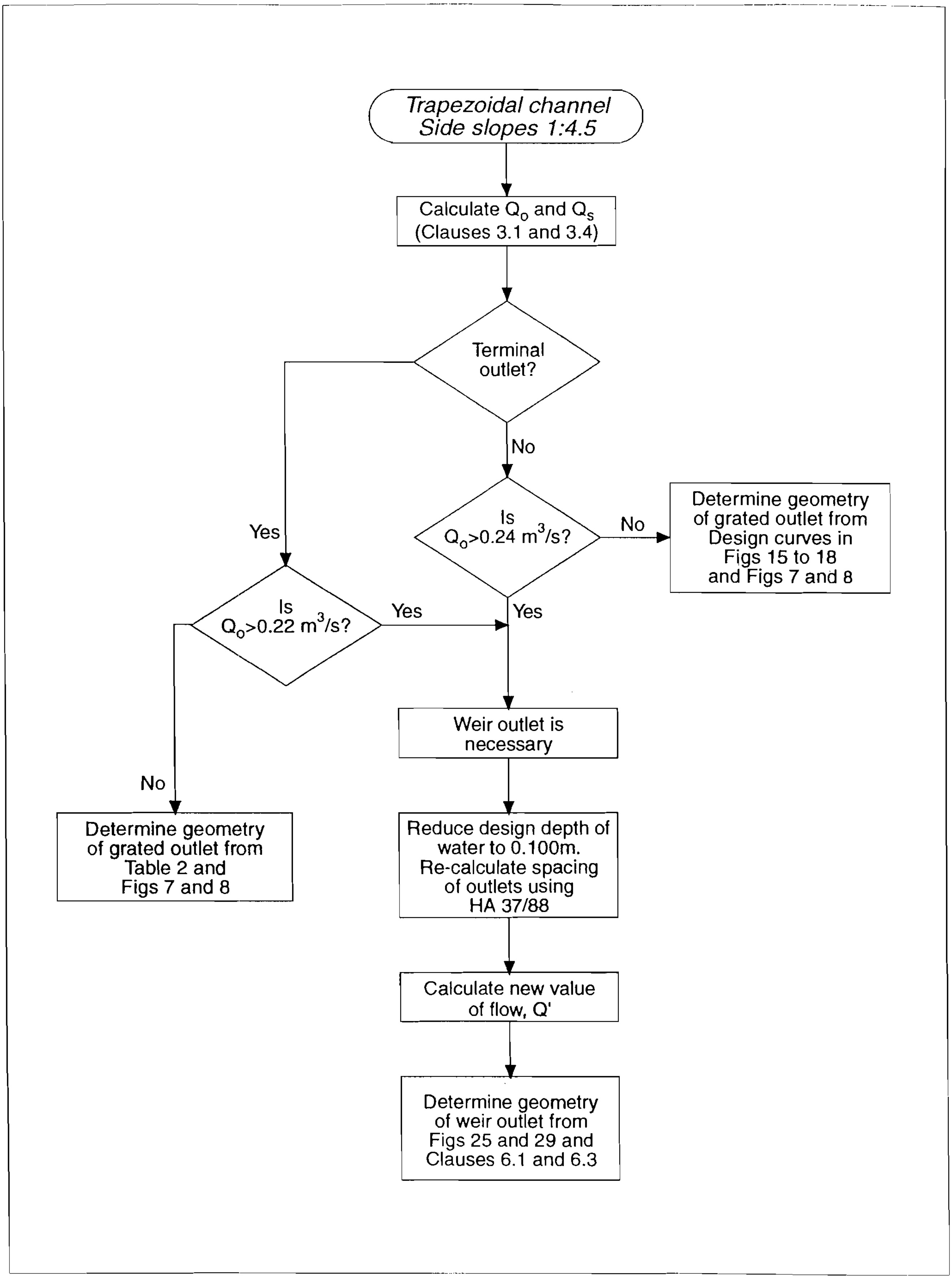
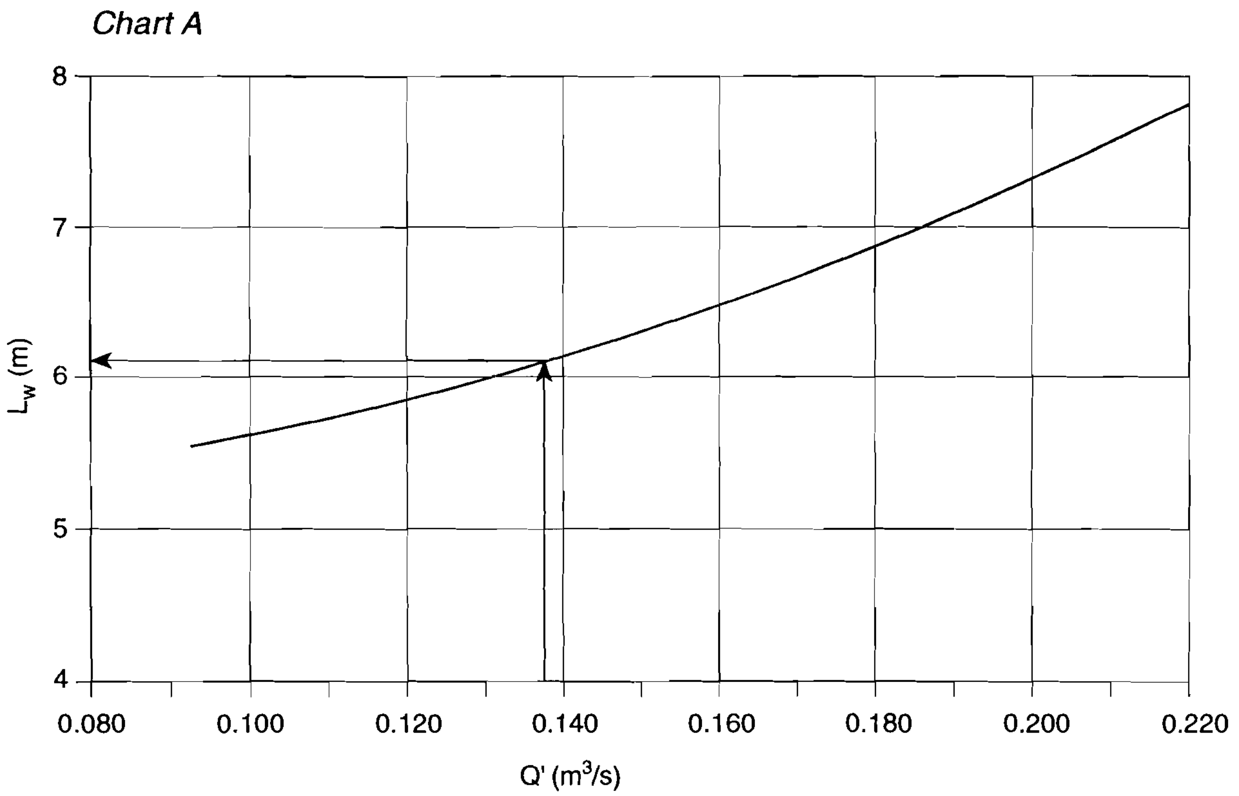


Figure 28 Flow chart for design of weir outlet in trapezoidal channel 1:4.5



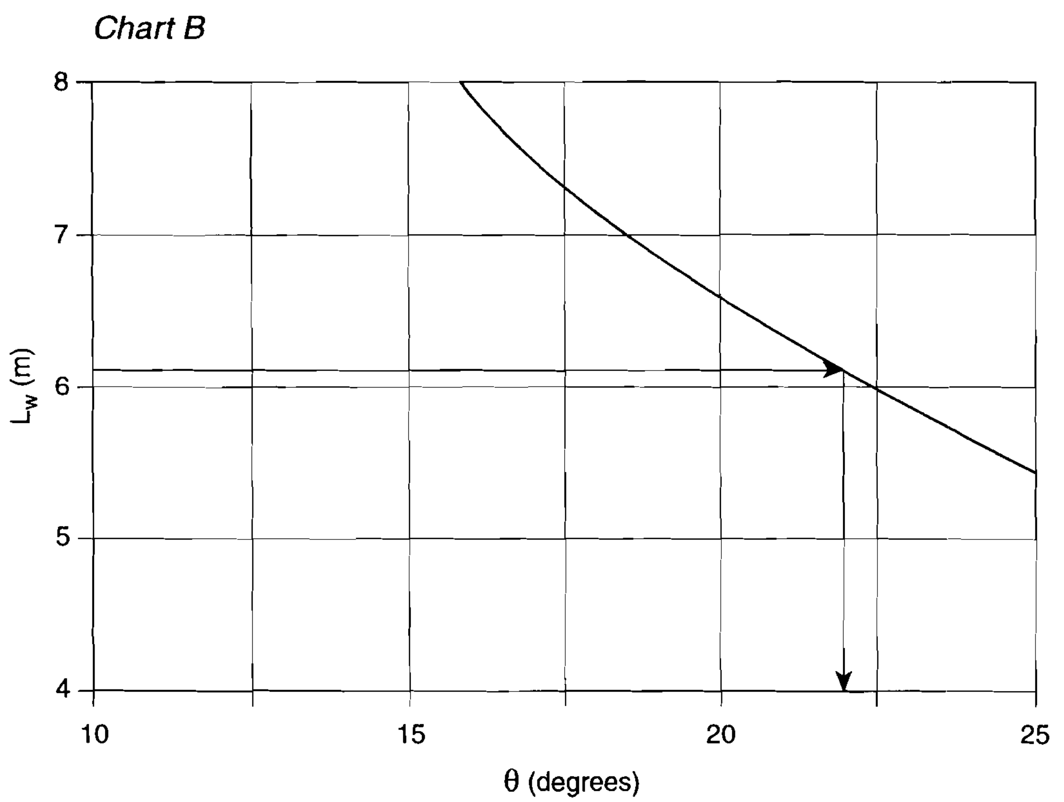


Figure 29 Geometric characteristics of weir outlet in trapezoidal channel 1:4.5

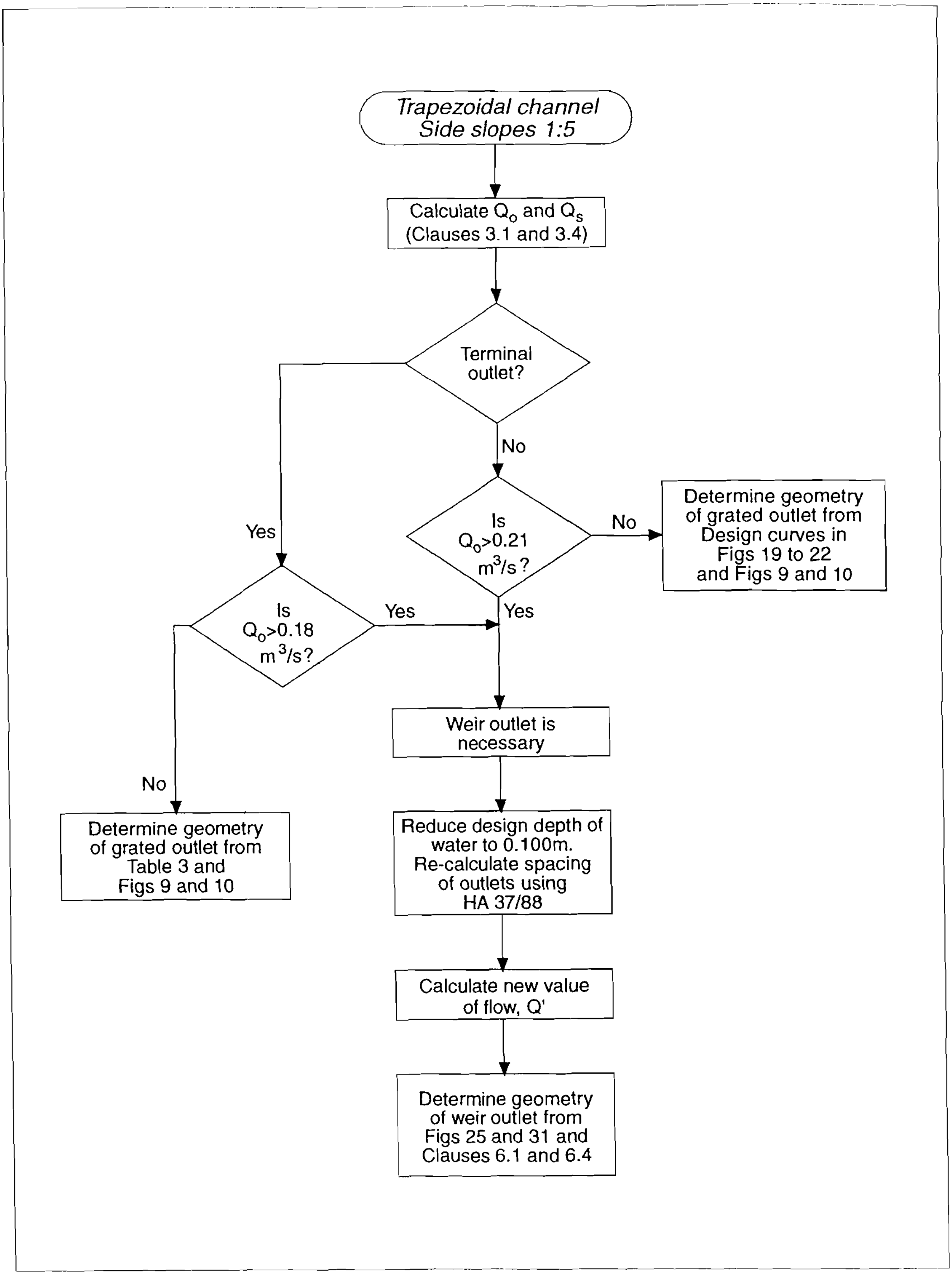


Figure 30 Flow chart for design of weir outlet in trapezoidal channel 1:5

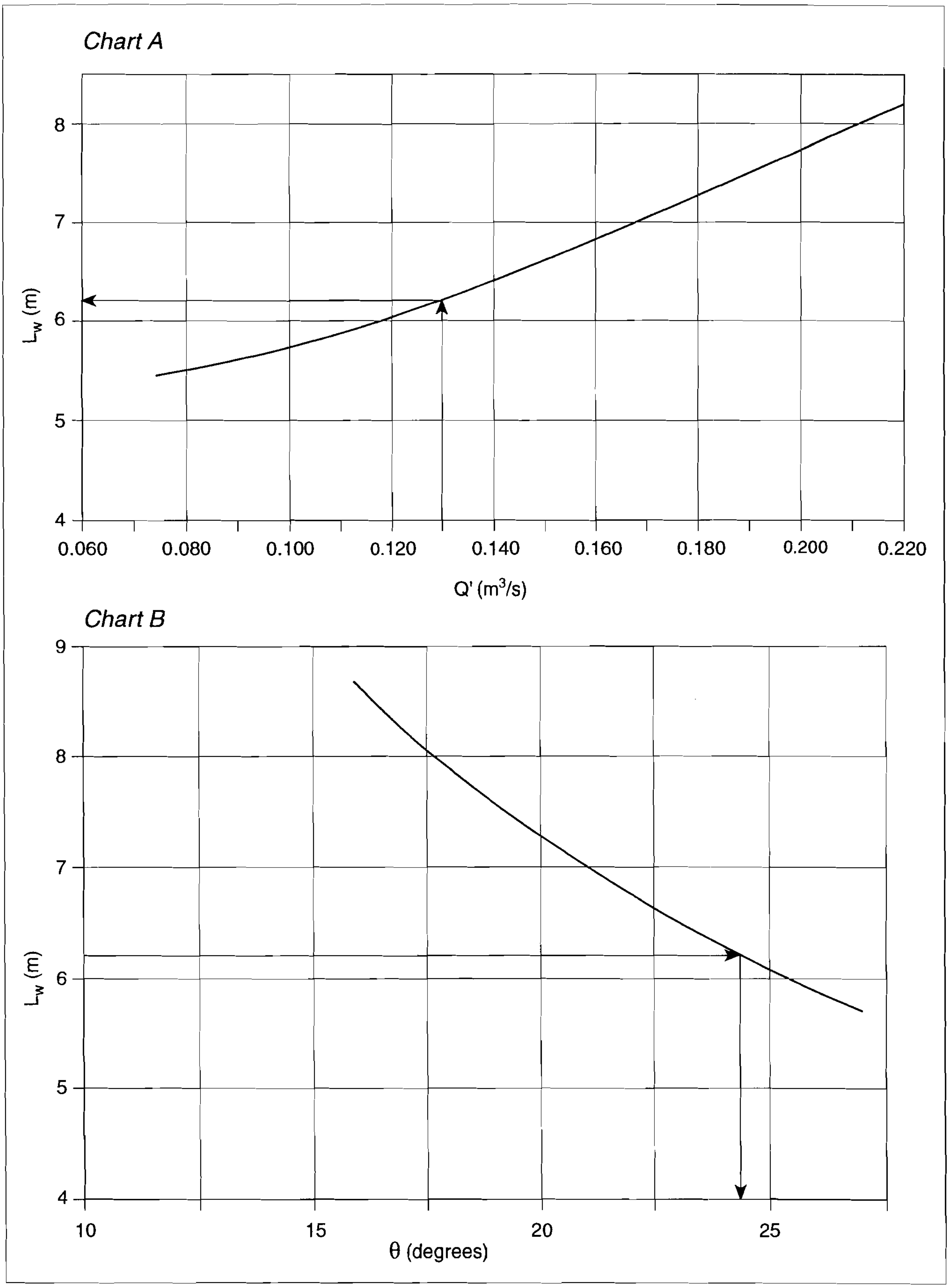


Figure 31 Geometric characteristics of weir outlet in trapezoidal channel 1:5

APPENDIX A

Examples of flow-collecting chambers

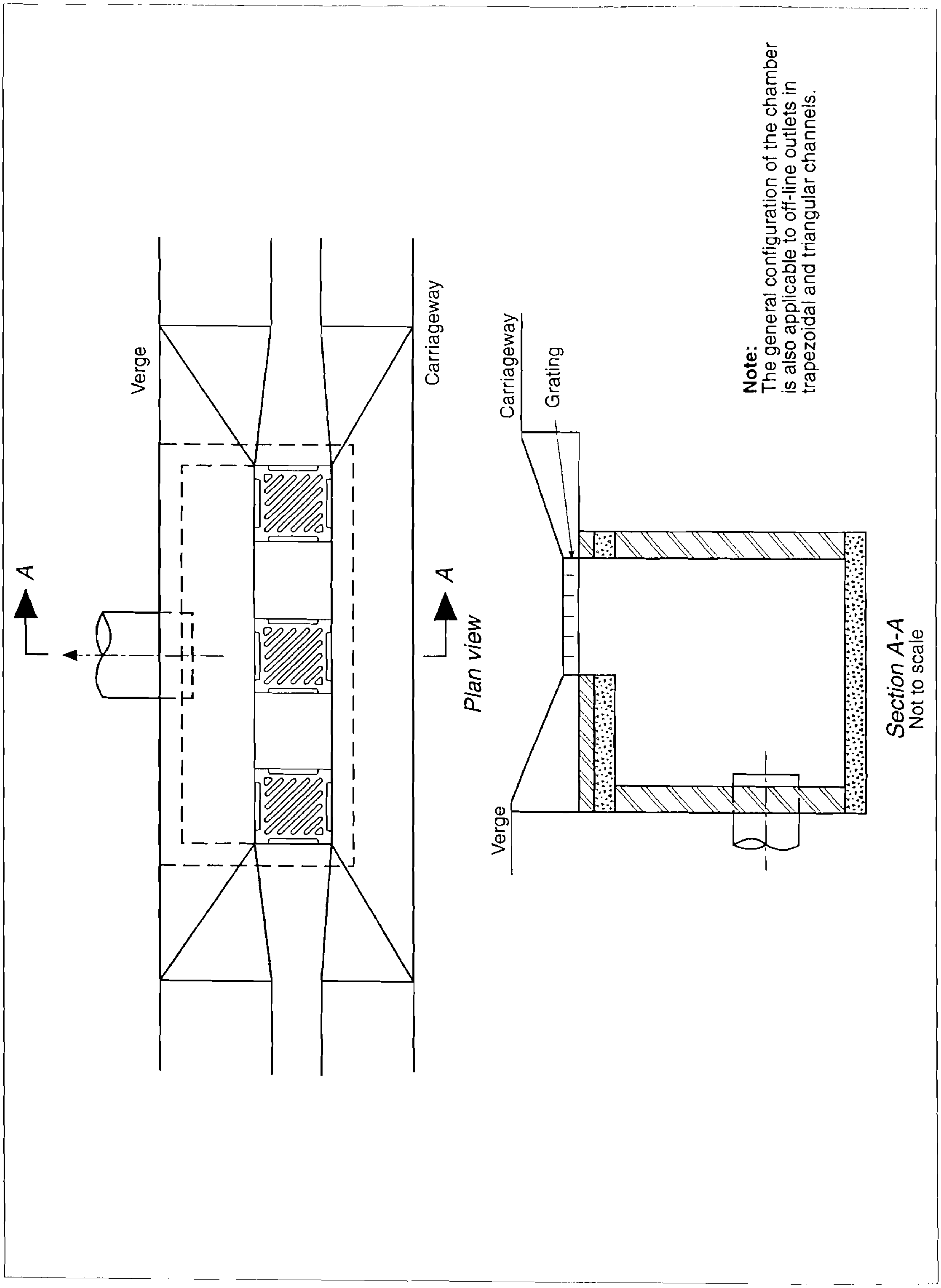


Figure A1 Example of collecting chamber for in-line outlet in trapezoidal channel

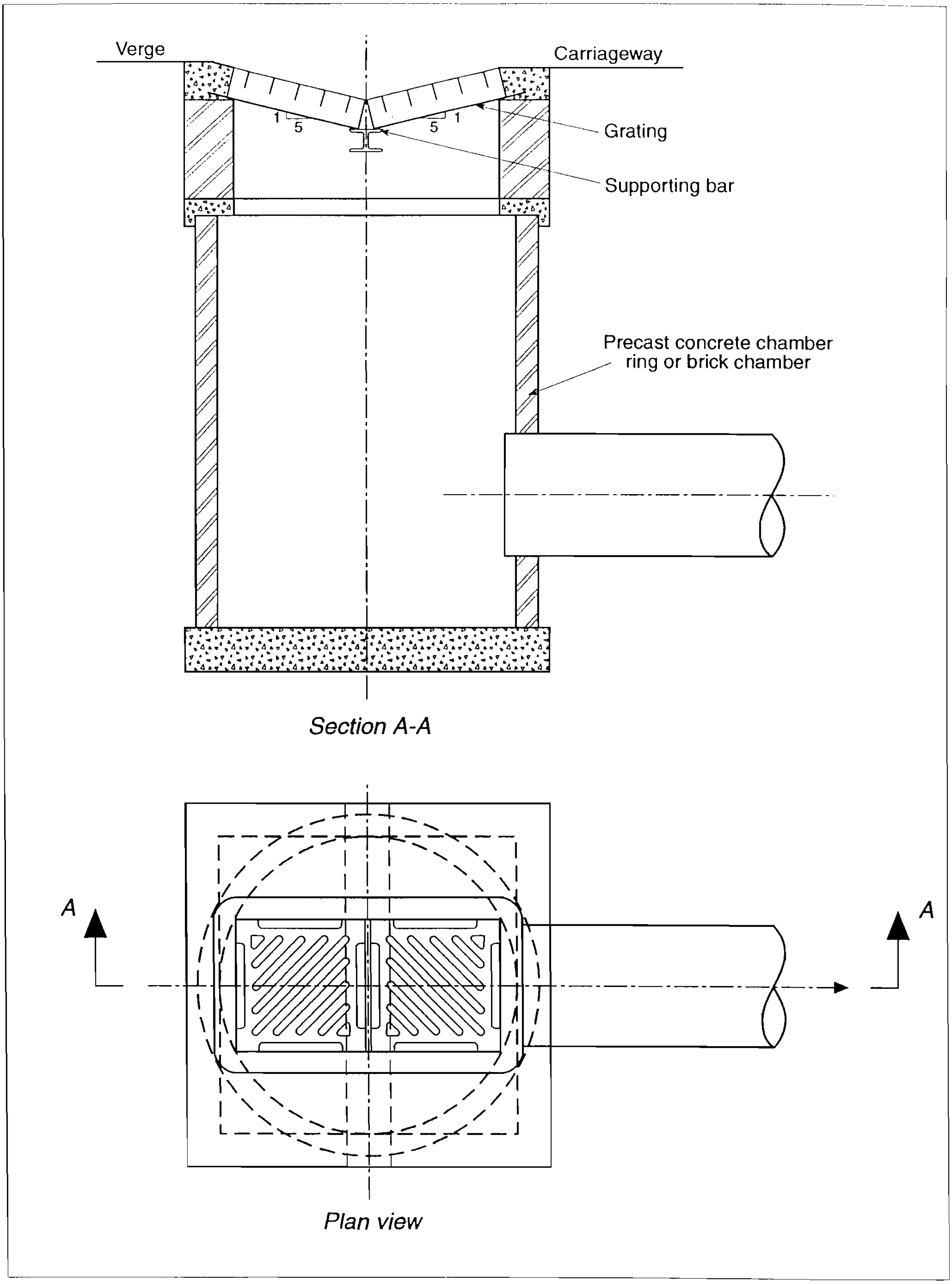


Figure A2 Example of collecting chamber for in-line outlet in triangular channel