Self-cleansing flow conditions for inverted siphons

R W P May

Report SR 559
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Contract

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Summary

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Inverted siphons are used when it is necessary to take flow in a gravity sewer, pumping main or open channel beneath a natural or man-made obstacle such as a river, estuary, highway or railway line. If the flow contains sediment, a key objective in the hydraulic design is to ensure that flow velocities in the rising leg of the siphon are sufficient to transport the material through the system without it depositing and causing blockages. In recent years, systematic research has enabled design equations to be developed for ensuring that self-cleansing conditions are achieved in sewers that are nearly horizontal. These equations show that the minimum velocity depends on the size of the pipe, the relative depth of flow, the concentration of the sediment and the size and density of the sediment particles.

The present study was carried out to produce similar information for sloping pipes in inverted siphons and so enable them to be designed more accurately taking account of the factors mentioned above. A programme of experimental research was therefore carried out at HR Wallingford using a test rig that could be adjusted to any angle between 0^{o} and 90^{o} to the horizontal. The pipes in the test rig had an internal diameter of 150mm and the overall height of the rig when in a vertical position was 3m. Tests were carried out using two sediment types – sand with a mean size of 0.78mm and gravel with a mean size of 4.3mm. Flow velocities in the tests were varied between 0.4m/s and 1.2m/s and volumetric sediment concentrations were between 10ppm and 400ppm. A special system was developed to inject sediment into the flow under pressure upstream of the start of the sloping section of test pipe.

Results from the tests were compared with a theoretical model of sediment transport that enabled the results to be generalised and linked to previous research on nearly-horizontal pipes. A design equation for predicting the limit of deposition in sloping pipes was developed from the results and used to produce tables for minimum self-cleansing velocities in inverted siphons for a wide range of pipe sizes and sediment conditions.

This report is divided into two main parts. The first part describes the experimental study and the method used to analyse the results and produce the design equation for the limit of deposition in sloping pipes. The second part of the report reviews existing data on inverted siphons and uses this information, together with the results of the experimental study, to provide general guidelines and recommendations for their hydraulic design. Particular attention is given to inverted siphons in gravity sewers because they tend to present the largest range of design problems.

Notation

A	Cross-sectional area of flow
A_S	Cross-sectional area of part of flow in which sediment particles are moving in flume traction
C	Hazen-Williams roughness coefficient of pipe
C_D	Drag coefficient of sediment particles
C_{DO}	Value of C_D for isolated particle
C_V	Volumetric sediment concentration [= Q_S/Q]
C_{VI}	Value of C_V in inclined pipe
C_{VO}	Value of C_V in horizontal or nearly horizontal pipe
D	Internal diameter of pipe
D_E	Internal diameter of circular pipe having same cross-sectional area as square or circular conduit
d	Representative particle size of sediment
d_{50}	Sediment size for which 50% of the sample is smaller by weight
e	Conversion factor between kinetic energy and potential energy of particles
F_D	Drag force on particle exerted by flow
F_R	Resistance force to movement of sediment particles in horizontal or nearly horizontal pipe
F_{RI}	Total down-slope force acting on particle moving up an inclined pipe
F_W	Immersed weight of particle
G	Numerical coefficient in Equation (A.26)
g	Acceleration due to gravity
H_F	Head loss due to frictional resistance of siphon pipe
H_P	Point head losses in pipe
H_T	Total head loss across siphon
i	Head loss gradient of flow
j	Ratio between effective flow volume occupied by sediment particle and actual volume of particle

Notation *continued*

K	Dimensional quantity defined by Equation (A.36)
KE	Kinetic energy of sediment particle
k_S	Equivalent sand roughness of pipe (in Colebrook-White formula, Equation (B.5))
L	Length
L_P	Length of pipe of given diameter
N	Number of sediment particles in length L
n	Manning roughness coefficient of pipe
P	Wetted perimeter of pipe
Q	Volumetric flow rate of water
Q_M	Maximum flow capacity of inverted siphon
Q_S	Volumetric transport rate of sediment
R	Hydraulic radius of pipe [= A/P]
r	Radius of curvature of pipe
S	Gradient of pipe (positive sloping downwards)
s	Specific gravity of sediment particles [$= \rho_S / \rho$]
U_S	Mean velocity of sediment particles
V	Mean cross-sectional velocity of flow in pipe [= Q/A]
V_T	Value of V for threshold of movement of individual sediment particles in nearly horizontal pipe
V_{TI}	Value of V_T in inclined pipe
V_1	Initial estimate of V
V_2	Improved estimate of V
W_S	Fall velocity of isolated particle in still water
y	Depth of water normal to pipe invert [y = D if pipe flowing full]
y_S	Depth of layer normal to pipe invert in which sediment particles are moving in flume traction

Notation *continued*

α_2 Shape factor of particle relating to its cross-sectional area

α_3 Shape factor of particle relating to its volume

β Ratio between average flow velocity experienced by sediment particles and mean flow velocity V

ΔPE Change in potential energy of sediment particle

ϕ Angle turned by bend

γ Angle of repose of sediment particles resting on other particles

μ Effective friction coefficient between sediment particles and wall of pipe

ν Kinematic viscosity of water

θ Angle of pipe to horizontal (positive upwards)

θ_E Effective value of θ corresponding to the point of limiting sediment deposition in a 90^o bend connected to a section of vertical pipe

ρ Density of water

ρ_S Density of sediment particles

σ Slope factor for total down-slope force in inclined pipe

ξ Non-dimensional coefficient for point head loss

ξ_B Value of ξ for bend

ξ_I Value of ξ for pipe inlet

ξ_O Value of ξ for pipe outlet

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1. INTRODUCTION

Inverted siphons consist of arrangements of pipes and chambers that are constructed to allow water to flow under gravity beneath natural or man-made obstructions. One type of situation in which they may be used is where the line of an existing sewer, gravity main, or natural watercourse will be cut by the construction of a new road or railway line at the same level. Another common type of application is where sewage flows collected by a gravity drainage system along one side of a river or estuary need to be conveyed to a treatment works on the other side.

A typical inverted siphon is shown in Figure 1 and is made up of the following elements:

- An inlet chamber receiving flow from an upstream sewer, culvert or natural channel.
- A length of downward-sloping pipe.
- A length of pipe constructed level or nearly level beneath the obstruction.
- A length of upward-sloping pipe.
- An outlet chamber discharging the flow to a downstream sewer, culvert or natural channel.

Although widely accepted, the term "inverted siphon" is something of a misnomer because no siphoning or negative pressures need to develop in the pipes in order for the system to operate. In fact the pipes in an inverted siphon run full under positive pressure, with the flow being produced by the building up of a head difference between the water levels in the inlet and outlet chambers. The term is used because the shape of an inverted siphon is opposite to that of a "true siphon" in which flow can pass over an obstruction if air is first removed from the pipes (so as to allow the development of negative pressures).

Large numbers of inverted systems have been built in the UK and worldwide, particularly for sewerage systems where gravity drainage flows need to be taken from one side of a river to the other. For major systems, the structures can be large and expensive to build but often there is no viable alternative to their use. Design of an inverted siphon requires solution of a range of difficult hydraulic and practical problems, and poorly designed siphons can be very troublesome in operation. The most serious problems tend to occur with siphons in sewerage systems because the sediment carried by the flow may be liable to deposit in the upward-sloping pipes and thereby cause blockages. .

Existing design practice for inverted sewer siphons is based on the use of assumed values of minimum flow velocity that need to be achieved in the pipes to prevent blockages. These values do not appear to have been determined systematically, and they do not take account of variations in factors such as pipe size, inclination of pipe, sediment load, particle size, particle density, etc. Recent research on sediment movement in pipes, including studies carried out at HR Wallingford, enabled CIRIA Report 141 (Ackers, Butler & May, 1996) to be produced giving design recommendations for minimum self-cleansing velocities and gradients in gravity sewers. The objective of the present project was to develop equivalent results for sediment movement in upward-inclined pipes and thereby to produce improved guidelines for the design of inverted siphons. The project was funded by the Construction Directorate of the Department of the Environment, Transport and the Regions (DETR), as part of its Partners in Innovation programme, and by HR Wallingford.

Chapter 2 of this report describes the overall scope of the study and the methodology adopted. This is followed by Part A (Chapters 3 to 5) which gives full details of the experimental work carried out at HR including the design of the test rig, the measurements made and the method of analysis. Part B of the report (Chapters 6 and 7) uses the information from the experiments and other sources to provide general guidelines for the hydraulic design of inverted siphons.

2. METHODOLOGY

2.1 Scope

The overall objective of the project was to produce guidelines for the hydraulic design of inverted siphons. The means used to meet this objective were:

(1) Collection of data on previously constructed inverted siphons.

(2) Review of existing design criteria and recommendations.

(3) Design and construction of a test rig at HR for studying flow conditions in inverted siphons at laboratory scale.

(4) Carrying out of experiments in test rig.

(5) Analysis of experimental results and collation of data from steps (1) and (2).

(6) Preparation of report containing design guidelines and data from study.

As explained in Chapter 1, the primary factor affecting the design and operation of inverted siphons is the possibility of blockage by sediment. Sewerage systems operating under gravity tend to experience the most difficult combination of problems because of:

- high sediment loads with wide variations in sediment type, size and density;
- wide range of flow rates between dry weather conditions and peak storm events;
- correspondingly wide range of flow velocities;
- need for head losses through siphon to be minimised.

In this study it was therefore decided to concentrate mainly on the case of inverted siphons for sewers, since guidelines covering their design can also be adapted to apply, as required, to less severe situations such as sewer pumping mains and gravity systems carrying clear-water flows.

2.2 Questionnaire

As part of the data collection exercise on existing inverted sewer siphons, it was decided to send a postal questionnaire to UK Sewerage Undertakers, Agent Authorities and design organisations who were thought likely to have particular interests in the subject. The information that was requested covered items such as the numbers of siphons in use, problems in operation and design requirements for new siphons (eg minimum flow velocities, slopes of pipes, etc). A copy of the questionnaire is contained in Appendix 1.

After approval by DETR, the questionnaire was sent to thirteen organisations but only one of them (a large Sewerage Undertaker) provided a completed return. One organisation gave a limited amount of information by telephone, and another answered that it dealt with the construction of few new inverted siphons and that each was designed uniquely.

Information from the completed questionnaire is given in Appendix 1. The respondent estimated that the Sewerage Undertaker had between 30 and 50 inverted sewer siphons and that the only serious problems were caused by blockages. Details provided verbally by another Sewerage Undertaker indicated that it had of the order of 300 sewer siphons. Many of these were single-pipe designs which had proved troublesome due to blockages. The Sewerage Undertaker was therefore undertaking a programme to upgrade them to a twin-barrel pattern.

During the study HR obtained data on two large inverted siphons recently constructed in Paris. The information is included in Section A2.15 of Appendix 2.

Although the postal survey did not produce the expected amount of information, it did confirm that a significant numbers of inverted sewer siphons are in operation in the UK and that blockages are the major cause of problems.

Part A – HR Experimental Study

3. DESIGN OF EXPERIMENTS AND TEST RIG

3.1 Introduction

The movement of sediment in upward or downward sloping pipes has been studied previously by researchers such as Robinson & Graf (1972) in connection with the hydraulic transportation of materials by pipeline. However, in these types of application, flows are pumped at high velocities (of the order of 5m/s or more) in order to keep the particles in suspension and achieve high transport rates with volumetric sediment concentrations typically in the range $C_V = 10^4$ to 10^5 ppm. Conditions are very different in gravity sewers where peak flow velocities are normally only about 0.5m/s to 1.5 m/s. At these velocities the heavier inorganic sediments are transported only as bedload with volumetric concentrations that are typically in the range $C_V = 10$ to 100 ppm. Therefore, the information from the earlier studies on the transportation of materials by pipeline cannot safely be extrapolated to the case of inverted sewer siphons. This lack of applicable data was the reason why the present project was defined around a programme of new experimental work on sediment movement in sloping pipes.

Recent studies on sediment transport in nearly horizontal pipes, including research at HR Wallingford (see May, 1993), have shown that the minimum velocity needed to prevent the build-up of sediment in a sewer depends on a range of factors such as the size of the pipe, the depth of flow, the concentration of the sediment, and the size and grading of the sediment. Results of these studies were reviewed in CIRIA Report 141 (Ackers, Butler & May, 1996) and used to produce design recommendations on minimum self-cleansing velocities and gradients for gravity sewers. For the present project, it was considered important that the new experimental work should be carried out in a similar way to previous studies on nearly horizontal pipes to help ensure consistency and enable the effects of slope to be clearly identified. Also, it was decided that the flow conditions in the experiments should be chosen so as to produce sediment concentrations covering the range of design values recommended in CIRIA Report 141 for coarser sediments normally transported as bedload (ie, C_V = 4ppm to 80ppm for particle sizes ≥ 0.6mm).

3.2 Definition of self-cleansing conditions

In order to enable inverted siphons to be designed in a more scientific and systematic way, it is necessary to define the term "self-cleansing" in terms of the required level of hydraulic performance. Since flow rates and sediment loads in gravity sewer systems can vary very considerably with time, it is unrealistic to expect to be able to design an inverted sewer siphon so that no deposition ever occurs under any possible flow condition. The key requirement is to ensure that, averaged over a suitable period of time (eg one day), sediment will be able to be transported through the siphon at the same rate that it approaches the siphon without any long-term build up of sediment deposits and without the sediment having a significant adverse effect on the flow capacity of the siphon.

Sediment can be transported along a pipe in various ways:

- in suspension within the flow;
- in so-called "flume traction" with particles moving individually in contact with the invert of the pipe;
- as separated slow-moving dunes; or
- as a surface layer of particles moving over a continuous deposit of stationary material.

Fine silt particles and low-density organic materials can be transported quite easily in suspension by the flow velocities that typically occur in gravity sewers; however, uneconomically steep gradients would be needed to achieve this mode of transport for the heavier inorganic fractions such as medium/coarse sand and gravel. It is not normally desirable to allow significant stationary deposits to form in sewers because their high surface roughness can reduce the flow capacity of the pipes very considerably. The two main options are therefore to design sewerage systems so that the heavier inorganic fractions can be transported in flume traction or as separated dunes without the formation of continuous deposits.

Previous studies (eg May, 1982 and May et al, 1989) have demonstrated that the transition from flume traction to transport with separated dunes typically produces an increase in head loss of the order of 5% to 15%. Also, in the case of an inverted sewer siphon, it is desirable to minimise the quantity of sediment present in the system at any given time. This is because during periods of very low flow there is a danger that sediment being transported up the sloping leg of the siphon might stop moving and slide back down to the lowest point in the system. For given flow conditions, the speed of particles travelling in flume traction is considerably higher than that of particles travelling in separated dunes. It, therefore, follows that the amount of material that might potentially cause a blockage in a siphon is very much greater if separated dunes are the normal mode of transport.

For these reasons, it was decided in this project to define the requirement for self-cleansing in terms of the ability of the flow to transport sediment through a siphon in flume traction at a rate corresponding to the average sediment load approaching the siphon. The most critical point for application of this criterion will normally be the upward sloping leg of the siphon. The experimental measurements described in this part of the report were therefore concentrated on determining the flow conditions applying in sloping pipes at the limit of deposition, ie at the point of transition from flume traction to transport with separated dunes.

3.3 Key features of test rig

A new test rig needed to be constructed to allow the experiments to be carried out with pipes at different slopes. The two key problems that had to be solved in the design were:

- Finding a simple way of varying the slope of the pipe between 0^o and 90^o without requiring major changes to the pipe fittings or the support structure.

- Finding a way of injecting sediment at an accurate and controllable rate into a pipe flowing full and under pressure.

A conventional solution to the problem of changing the slope would have been to fit a different pair of bends at either end of the sloping section of pipe for each of the required gradients. However, as the bends needed to have a long radius of curvature (to prevent disturbance of the flow in the sloping section) and to be transparent (to allow observation of the sediment movement), it would have been expensive to have had more than a few different bend angles manufactured. Another, and more significant, problem associated with the conventional solution is that the discharge point and the position of the supporting framework at the top end of the sloping pipe would have had to be changed each time the pipe slope was altered. These factors would have made it time-consuming to change from one slope to another and would have tended to limit flexibility in the way the test programme was carried out.

The solution adopted was to construct the length of test pipe in the form of a loop with three straight sides and having a rotational joint at either end (see Figure 2). This layout enabled the loop to be rotated about an axis located at floor level and coinciding with the line of the “missing” fourth side of the loop. Thus, tests with a horizontal pipe could be carried out with the loop laid sideways and parallel to the floor. For tests with a vertical pipe, the loop could be raised to the vertical position. Each rotational joint consisted of two serrated-face stub flanges with stainless steel backing rings. This configuration enabled the flanges to be rotated relative to each other and then clamped together by means of bolts so as to enable the pipe loop to be set at any angle of inclination between 0^o and 90^o. The horizontal section of pipe at the top of the loop was supported in its appropriate position by a small movable gantry. An additional advantage of this design was that the point of discharge of the system was the same for all angles of inclination of the pipe loop.

As shown in Figure 3a, the overall height of the rotatable pipe loop when vertical was 3.0m measured between the centrelines of the pipes at the top and bottom of the loop. The internal diameter of the pipes was 150mm. A bend with a long radius (r = 1.0m to centreline of pipe) was used at the upstream end in order to minimise disturbance to the flow entering the upward sloping leg of the loop, which had a straight length of 1.89m. Both these sections of pipe were constructed using transparent PVC to allow unrestricted observation of sediment movement in the test section. The sections of pipe used further downstream were of grey uPVC.

The other key problem to be solved was the method of injecting sediment at an accurate rate into the flow upstream of the rotatable pipe loop containing the transparent test section. If, for example, a vibrating-screw injector had been used to supply the sediment under gravity from above, this would have required mounting the injector more than 3m above floor level when the pipe loop was in its vertical position. Also, if the sediment had been conveyed to floor level through a tube (necessarily full of water), there would have been a danger of the tube becoming clogged and of its not supplying sediment to the flow at the intended rate.

To avoid these problems an innovative solution was developed using a piston to push sediment into the pipe from below against the hydrostatic pressure of the water. To accommodate the piston arrangement, a fixed vertical pipe loop was constructed upstream of the rotatable pipe loop (see Figures 2 and 3b). A short stub pipe was attached to the underside of the pipe in the top of the fixed loop so as to allow installation of a removable tube containing sediment. Sediment was pushed upwards from the tube into the flow by a piston driven by a variable-speed electric motor. The tube was 650mm long with an internal diameter of 68mm and was sealed into the stub pipe by means of a screw coupling. The tube, piston and electric motor were mounted on a trolley that could be jacked up and down to allow the tube to be withdrawn from the stub pipe and re-filled with sediment. The length of horizontal pipe to which the tube was attached had an internal diameter of 139mm but its cross-sectional area was reduced by fitting a 22mm deep block in the soffit of the pipe. The purpose of the block was to produce significantly higher flow velocities in this part of the test rig so that injected sediment would be carried rapidly downstream and help ensure that the first point at which sediment would deposit in the system would be in the test section of 150mm diameter transparent pipe. A triangular deflector block (15mm high and 75mm long) was also installed in the invert of the 139mm diameter pipe just upstream of the position of the tube in order to shelter the point of sediment injection and prevent flow diving into the tube and causing non-uniform erosion of the sediment. The 139mm pipe was of transparent PVC to allow the behaviour of the injected sediment to be observed.

Other features of the test rig were straightforward and are shown in Figure 2. A recirculating system was used with water being drawn from an open sump by a pump with a capacity of about 28 l/s. At the downstream end of the rotatable pipe loop, the flow discharged into a hopper containing a fine gauze screen to retain the sediment while the clean water was returned to the sump. The flow rate was measured by an electromagnetic pipe flow meter. Air bleed valves were fitted in the tops of the fixed and rotatable pipe loops to allow air to be removed during the filling of the system. A tee-connection was also installed between the two pipe loops so that any air bubbles travelling along the soffit of the main pipe could be intercepted and drawn off via a length of flexible hose.

4. PROGRAMME OF EXPERIMENTS

4.1 Test conditions

To determine the effect of pipe slope on the limit of sediment deposition, tests were carried out with the rotatable pipe loop set at the following angles (θ) to the horizontal: 0^o, 15^o, 22.5^o, 30^o, 37.5^o, 45^o, 60^o and 90^o.

Two different types of non-cohesive sediment were used in the tests:

- Coarse sand – with a mean particle size of d_{50} = 0.78mm and a specific gravity of s = 2.62.
- Fine gravel – with a mean particle size of d_{50} = 4.3mm and a specific gravity of s = 2.63.

Grading curves of the two sediment types are shown in Figures 4a and 4b.

Flow velocities in the tests were chosen so as to produce values of the volumetric sediment concentration covering the range between $C_V = 1.0x10^{-5}$ and $C_V = 4.0x10^{-4}$ (ie 10 ppm and 400 ppm respectively). As a result, the mean flow velocity in the 150mm diameter transparent pipes was varied between about V = 0.4m/s and 1.2m/s depending on the pipe slope and sediment type.

4.2 Experimental procedure

The type of sediment injection system developed for the study (see Section 3.2) required a special technique for accurately determining the concentration of sediment in the flow within the transparent test section.

The first step involved calibrating the speed of movement of the piston in the 68mm supply tube against the dial setting of the electric motor; two different gear ratios were available in order to cover the range of piston speeds needed for the tests. Knowing the specific gravity of the sediment particles and the average bulk density of the sediment in the tube therefore enabled an estimate to be made of the dial setting required to produce a given rate of sediment injection for a particular test. The test was then carried out at this estimated dial setting but the actual injection rate achieved was determined independently as follows. Before the test, the supply tube was first filled with water and a known weight of dry sediment dropped into the water until the tube was filled with sediment to the required level. This technique helped remove air from the voids between the particles and aided achievement of a uniform compaction of the material. The dry weight (and volume) of the sediment particles per unit length of tube was therefore accurately known. During the test, the time taken for the piston to travel a measured distance along the supply tube was recorded so that the actual volumetric rate of sediment injection (Q_S, in units of m^3/s of solid sediment) could be calculated precisely. This injection rate could be assumed to be equal to the sediment transport rate through the test section because the design and operation of the system ensured that the sediment particles would be transported rapidly through the upstream pipes without accumulating to form stationary deposits. Knowing the flow rate of water (Q, in m^3/s) from the reading of the electromagnetic flow meter enabled the volumetric sediment concentration in the test section to be calculated from $C_V = Q_S/Q$, where C_V is non-dimensional.

Experience of using the equipment showed that it was best not to completely fill the sediment supply tube to its top so that some time would elapse after the start of a test before the piston began to push sediment up into the flow. If the tube were completely filled, the top few centimetres of sediment tended to be rapidly eroded by the flow, causing the sediment concentration to exceed the required value and possibly leading to the unwanted formation of deposits in the pipes upstream of the test section (see above).

During a test the following procedure was followed when determining the limit of deposition in the section of sloping transparent pipe. First, a target value for the flow velocity in the test section was decided and an estimate made of the likely sediment concentration at the limit of deposition (based on predictions or the results of previous tests). The required sediment injection rate corresponding to this concentration was then calculated and used to set an appropriate speed for the movement of the piston. This speed was kept constant throughout the test, and the actual value of the sediment injection rate determined accurately using the procedure described above. The flow rate from the pump was initially set to be considerably above the value corresponding to the target velocity so as to ensure that the injected sediment was transported through the pipes rapidly (in flume traction) without danger of any deposition occurring. Once the injection rate of sediment was seen to have reached a steady state (by observation through the transparent 139mm and 150mm diameter pipes), the flow rate of water from the pump was gradually reduced until sediment particles moving up the inclined pipe were seen to coalesce and form small, slowly moving deposits on the invert of the pipe. As explained in Section 3.2, this change in the mode of sediment transport is considered to be a suitable criterion for defining the limit of deposition in inverted siphons. To check that the limit had been correctly identified, the flow rate was then increased slightly to make sure that the deposits would disperse and that the sediment would resume travelling as individual particles in flume traction. Fine adjustments to the flow rate could be made using a small by-pass valve, installed in parallel with the main control valve, and this enabled the limit of deposition to be determined with good precision.

As mentioned in Section 3.1, an important objective of the study was to link the new results for sloping pipes with the large body of data already existing for nearly horizontal pipes. For this reason, tests were carried out with the rotatable pipe loop in a horizontal position in order to check that the new test rig gave similar results to those obtained in earlier studies. The limit of deposition was determined using the same technique as that described above.

Test with the pipe in the vertical position showed that there were two different types of limiting flow condition. By using the by-pass valve, small adjustments could be made to the flow rate to vary the upward or downward speed of movement of the sediment particles in the vertical pipe. Thus, it was possible to determine the flow velocity just needed to keep all the particles moving upwards. The value of this first limiting velocity was approximately the same as the fall velocity of the sediment particles in still water. However, the flow rate was not sufficient to prevent the formation of a significant sediment deposit in the long-radius bend connected to the bottom of the vertical pipe. This deposit occurred because the flow velocity was not high enough to produce a direct transition from bed-load transport entering the bend to suspended-load transport in the vertical pipe. Sediment at the surface of the deposit was transported forwards by the flow, with part of it being carried upwards into the vertical pipe and part being retained by a recirculating eddy formed at the downstream end of the deposit. The size of the partial blockage grew until the local increase in velocity was sufficient to produce an upward rate of sediment transport that matched the rate entering the bend. Also, the position of the deposit was not constant but fluctuated with time, slowly moving up the bend until it reached a point where it suddenly slid down again, the process then repeating itself in a cyclic fashion.

Based on these observations, it was concluded that the formation of this type of sediment deposit would be undesirable in an inverted siphon because it would produce a significant increase in head loss and cause a permanent accumulation of material at the lowest point in the system. It was, therefore, decided to define the limiting flow condition for a siphon with a vertical upward leg as being the minimum velocity needed to enable a direct transition, without formation of a sediment deposit, from bed-load transport in the connecting bend to suspended-load transport in the vertical pipe.

4.3 Test results

The results of the tests to determine the limiting flow conditions at different pipe slopes are given at the end of Part A of this Report in Table A.1 for the coarse sand (mean particle size d_{50} = 0.78mm and specific gravity s = 2.62) and in Table A.2 for the fine gravel (d_{50} = 4.3mm and s = 2.63). Experimental data provided are: the angle of inclination (θ) of the test pipe to the horizontal; the limiting mean flow velocity (V) in the 150mm diameter test pipe, calculated as discharge divided by area of flow; and the corresponding volumetric sediment concentration (C_V) which, as described in Section 4.2, was defined as the volumetric flow rate of sediment particles (Q_S) divided by the volumetric flow rate of water (Q). As is clear from the nature of the experiments, the test pipe was flowing full of water under all test conditions.

Other information contained in Tables A.1 and A.2 relate to the analysis of the data which is described in Chapter 5.

The modes of sediment transport observed in the tests are shown in Plates 1 to 6. In Plate 1 the flow velocity is well above the value at the limit of deposition and the sediment is travelling up the invert of the sloping pipe as individual particles; this mode of transport is termed "flume traction" (see Section 3.2). In Plate 2 the flow velocity is just above the limit of deposition and the particles are travelling closer together but still individually. Only a small reduction of velocity is then necessary to reach the limit of deposition; turbulent fluctuations in the flow cause the particles to come together to form single-layer deposits as shown in Plate 3; if the limit has not quite been reached, the deposits will only be temporary and will be dispersed again by the flow. At a slightly lower flow velocity, the deposits become permanent and travel slowly up the invert of the pipe as a series of small distinct dunes (see Plate 4). If the flow velocity is reduced further while keeping the sediment supply rate constant, the dunes can become considerably larger, as shown in Plates 5 and 6.

5. ANALYSIS OF DATA

5.1 General approach

As explained above, a key objective of the study was to produce guidelines on self-cleansing conditions for sewer siphons that are an extension of, and consistent with, established results for nearly horizontal pipes and sewers. As part of the work carried out for CIRIA Report 141 " Design of sewers to control sediment problems" (Ackers, Butler & May, 1996), a new design equation for predicting the limit of sediment deposition in pipes was developed by May using data from seven separate experimental studies. This limit corresponds to the transition from transport in flume traction to transport with separated dunes, as described in Section 3.2. The recommended relationship between the flow velocity and the sediment concentration in a horizontal or nearly horizontal pipe at the limit of deposition is given by the equation:

$$C_{VO} = 3.03 \times 10^{-2} \left(\frac{D^2}{A} \right) \left(\frac{d_{50}}{D} \right)^{0.6} \left(1 - \frac{V_T}{V} \right)^4 \left(\frac{V^2}{g\,(s-1)\,D} \right)^{3/2} \qquad \text{(A.1)}$$

where:

- C_{VO} is the volumetric sediment concentration (non-dimensional) and defined as the volumetric transport rate of sediment, Q_S (in m^3/s), divided by the volumetric flow rate of the water, Q (in m^3/s) ;
- D is the internal diameter of the pipe (in m);
- A is the cross-sectional area of the flow (in m^2);
- d_{50} is the mean particle size of the sediment (in m);
- V is the mean velocity (in m/s) of the water in the pipe (= Q/A);
- V_T is the threshold value of V (in m/s) at which individual particles of the sediment will first begin to move along the invert of a horizontal or nearly horizontal pipe;
- g is the acceleration due to gravity (= $9.81 m/s^2$);
- s is the specific gravity of the sediment particles (non-dimensional).

The numerical value of the coefficient 3.03×10^{-2} assumes that C_{VO} is expressed as a non-dimensional concentration (eg 50×10^{-6}) and not, for example, as a percentage (ie 0.005) or in parts per million (ie 50).

The threshold velocity, V_T , can be determined from the equation:

$$V_T = 0.125 \sqrt{g\,(s-1)\,d_{50}} \left(\frac{y}{d_{50}} \right)^{0.47} \qquad \text{(A.2)}$$

where y is the flow depth in the pipe (in m).

It was decided to use the above equations as the starting point for the analysis of the present data for the limit of sediment deposition in inclined pipes. The general format of Equation (A.1) was based on a theoretical analysis by May (1982) in which the limit of deposition was considered in terms of the time-averaged balance between the drag forces exerted on the sediment particles by the flow and the frictional resistance between the particles and the invert of the pipe. Although the analysis was not able to describe the full complexity of the factors involved, it did suggest particular relationships between the main parameters which were found to be valid when actual data from experiments were analysed. For the present study, it was decided to repeat the theoretical analysis for the case of sediment moving in an inclined pipe in order to identify the key differences compared with the case of a nearly horizontal pipe. The results of the new analysis, which is described in Section 5.2, were then used to determine how Equations (A.1) and (A.2) might best be modified to apply to the case of an inclined pipe. The comparison between the new versions of the equations and the experimental data in Tables A.1 and A.2 (see Section 4.3) is described in Section 5.3.

5.2 Theoretical analysis for limit of sediment deposition

5.2.1 Nearly horizontal pipes

For present purposes, a "nearly horizontal pipe" is defined as one in which the component of particle weight acting parallel to the pipe invert is small enough to be neglected in comparison with the forces exerted on the particle by the flow and the frictional resistance of the bed. Gravity sewers and drains laid at gradients of the order of 1/50 or flatter would normally be considered as being "nearly horizontal".

In order to identify the key factors affecting the limit of sediment deposition, consider the case shown schematically in Figure 5a. The circular pipe has a diameter of D, the water depth is y and the corresponding cross-sectional area of flow is A. The volumetic flow rate of water is Q and its mean velocity is V. Sediment particles are being transported by the flow as individual particles in flume traction and are contained in a layer that extends to a height y_S above the invert of the pipe. The portion of the flow in which the sediment is moving has a cross-sectional area of A_S and the average velocity of the water in this area is βV; due to the shape of the velocity distribution in a pipe, the value of β is normally less than 1. The mean size of the particles is d_{50} and they have shape factors α_2 and α_3 such that the cross-sectional area of an individual particle is $\alpha_2 d_{50}^2$ and its volume is $\alpha_3 d_{50}^3$; the specific gravity of the sediment is s. The average forward speed of the particles is U_S and the average volume occupied by each particle within the water layer of thickness y_S is j times the volume of the particle, ie $j\alpha_3 d_{50}^3$.

The sediment particles experience fluctuating lift and drag forces due to the flow but, averaged over time, the lift forces have no net effect because the particles remain within the layer close to the invert of the pipe. Therefore, on a time-averaged basis, the immersed weight of the particles must be transferred to the invert of the pipe either by direct contact or by exchange of vertical momentum with other particles or the fluid. Due to the relative movement between the particles and the pipe invert, the particles will experience a frictional resistance from the invert proportional to the reaction force supporting the weight of the particles; the factor of proportionality will be termed the effective friction coefficient, μ. The driving force that keeps the particles in motion is the drag exerted on them by the water. In order for a drag force to be generated, the sediment particles must be moving, on average, at a somewhat slower velocity than the surrounding water, ie $U_S < \beta V$.

Considering the time-averaged force on an individual particle, the frictional resistance, F_R , is proportional to its immersed weight, ie:

$$F_R = \rho\, g\, \mu\, (s - 1)\, \alpha_3\, d_{50}^3 \qquad \text{(A.3)}$$

where ρ is the density of the water. The drag force, F_D, due to the flow of the water is given by:

$$F_D = \frac{1}{2} \rho\, C_D\, \alpha_2\, d_{50}^2\, (\beta V - U_S)^2 \qquad \text{(A.4)}$$

where C_D is the drag coefficient of the particle. For time-averaged equilibrium:

$$F_R = F_D \qquad \text{(A.5)}$$

so Equations (A.3) and (A.4) can be combined to give:

$$U_S = \beta\, V - \left[2\, g\, (s - 1) \frac{\alpha_3}{\alpha_2} d_{50} \frac{\mu}{C_D} \right]^{1/2} \qquad \text{(A.6)}$$

which can be written in the form:

$$U_S = \beta\, (V - V_T) \qquad \text{(A.7)}$$

where V_T is the value of the mean flow velocity in the pipe at the threshold of movement of the sediment. The effective value of threshold velocity is therefore given by:

$$V_T = \frac{1}{\beta} \left[2\, g\, (s - 1) \frac{\alpha_3}{\alpha_2} d_{50} \frac{\mu}{C_D} \right]^{1/2} \qquad \text{(A.8)}$$

Consider now a length, L, of pipe containing N number particles moving at an average velocity of U_S. In time L/U_S , a total of N particles each of volume $\alpha_3 d_{50}^3$ passes the downstream end of the pipe length. Therefore, the volumetric transport rate of the sediment is given by:

$$Q_S = \frac{N\, \alpha_3\, d_{50}^3\, U_S}{L} \qquad \text{(A.9)}$$

The volumetric sediment concentration is defined as:

$$C_{VO} = \frac{Q_S}{Q} \qquad \text{(A.10)}$$

and since Q = AV, it follows that:

$$C_{VO} = \frac{N\, \alpha_3\, d_{50}^3\, U_S}{A\, L\, V} \qquad \text{(A.11)}$$

The total volume of water occupied by the particles moving in flume traction in the length L of the pipe is $A_S L$ (see Figure 5), and by definition this is also equal to N times the effective volume $j\alpha_3 d_{50}^3$ occupied by an individual particle. Therefore:

$$N = \frac{A_S\, L}{j\, \alpha_3\, d_{50}^3} \qquad \text{(A.12)}$$

Substituting in Equation (A.11) then gives:

$$C_{VO} = \frac{1}{j} \left(\frac{A_S}{A} \right) \left(\frac{U_S}{V} \right) \qquad \text{(A.13)}$$

The height, y_S , to which the sediment particles are able to rise above the invert of the pipe will tend to depend on the speed at which they are travelling. It would therefore be expected that there should be some relationship between the average kinetic energy, KE, of a particle:

$$KE = \frac{1}{2} \rho\, s\, \alpha_3\, d_{50}^3\, U_S^2 \qquad \text{(A.14)}$$

and the increase in potential energy, ΔPE, produced by raising a particle through a vertical distance, y_S, against the immersed weight of the particle:

$$\Delta PE = \rho\,(s-1)\, g\, \alpha_3\, d_{50}^3\, y_S \qquad \text{(A.15)}$$

If a proportion e of the kinetic energy can be converted into potential energy, it follows from Equations (A.14) and (A.15) that:

$$y_S = \frac{es}{(s-1)} \left(\frac{U_S^2}{2g} \right) \qquad \text{(A.16)}$$

The relationship between the height, y_S , and the corresponding flow area, A_S , below this level is determined by the pipe geometry. If y_S is relatively small compared with the diameter D (eg, $y_S/D < 0.2$), it can be shown (see May, 1982) that:

$$\frac{A_S}{D^2} \approx \frac{4}{3} \left(\frac{y_S}{D} \right)^{3/2} \qquad \text{(A.17)}$$

Substituting for y_S from Equation (A.16) and for U_S from Equation (A.7), it can be shown that:

$$\frac{A_S}{D^2} = \frac{4}{3} \left[\frac{es}{(s-1)} \right]^{3/2} \left[\frac{\beta^2\,(V-V_T)^2}{2\,g\,D} \right]^{3/2} \qquad \text{(A.18)}$$

Using this result in Equation (A.13) leads finally to:

$$C_{VO} = \left[\frac{\sqrt{2}}{3\,j}\, \beta^4\, (e\,s)^{3/2} \right] \left[\frac{D^2}{A} \right] \left[1 - \frac{V_T}{V} \right]^4 \left[\frac{V^2}{g\,(s-1)\,D} \right]^{3/2} \qquad \text{(A.19)}$$

This equation applies generally to the case of a circular pipe in which sediment is being transported in flume traction. At low sediment concentrations or well above the limit of deposition, the particles are far apart and interact very little with each other. However, as the limit is approached, the particles come closer together and start to shield each other from the flow. Put another way, as the value of the effective volume factor, j, in Equation (A.12) decreases, so does the effective drag coefficient, C_D , of the particles. Since the driving force exerted by the flow reduces, the particles slow down and they move closer together (see Equation (A.8)); this further reduces the value of C_D and so increases the tendency for them to coalesce to form deposits. Previous analysis by May (1982) investigated the relationship between the values of j and C_D , and showed that, for a given set of conditions, there is always a maximum possible rate of sediment transport in flume traction; if the sediment load exceeds that rate, there will be a sharp transition to movement with a deposited bed. The theoretical model, therefore, suggests that there is a limiting value of the factor j in Equation (A.19) corresponding to the limit of deposition. The value of j will depend on the particular conditions and is difficult to predict theoretically (as also are the values of β and e). However, the model is useful because it demonstrates the likely relationships between the major parameters such as the pipe size (D), the mean flow velocity (V) and the proportional flow depth (via the ratio A/D^2). Although the sediment size (d_{50}) does not appear explicitly in Equation (A.19), it affects the threshold velocity V_T (via Equation (A.8)) and can also influence the limiting values of e and j.

As explained in Section 5.1, the analysis of data for the limit of deposition carried out for CIRIA Report 141 (Ackers, Butler & May, 1996) led to the development of a new design equation for nearly horizontal pipes that was generally consistent with the above theoretical model; this can be seen from comparison of Equations (A.1) and (A.19).

5.2.2 Inclined pipes

The analysis described in Section 5.2.1 is now repeated for the case of sediment being transported up an inclined pipe in flume traction. Only those factors which distinguish the case from that of a nearly horizontal pipe are highlighted.

As shown in Figure 5b, the pipe is assumed to be inclined upwards in the direction of flow at an angle of θ to the horizontal. As a result, the drag force exerted on a sediment particle by the flow now has to balance the component of the immersed weight of the particle acting down the pipe as well as the frictional resistance provided by the pipe wall. Also, compared with the nearly-horizontal case, the value of the frictional resistance is altered because only a component of the weight of the particle is supported by the normal reaction from the pipe invert. By resolving forces, it can be shown that the total down-slope force, F_{RI} , acting on a sediment particle in an inclined pipe is given by:

$$F_{RI} = \rho\, g\, (s-1)\, \alpha_3\, d_{50}^3 \left\{ \sin\theta + \mu \cos\theta \right\} \qquad \text{(A.20)}$$

Comparing this with the resistance force, F_R , in a nearly horizontal pipe (see Equation (A.3)), it is useful to define the slope factor σ as:

$$\sigma^2 = \frac{F_{RI}}{F_R} = \frac{\left\{ \sin\theta + \mu \cos\theta \right\}}{\mu} \qquad \text{(A.21)}$$

Repeating the analysis for the time-averaged balance between the drag force and the resistance force described in Section 5.2.1, it can be shown that Equation (A.7) for the speed of movement of the sediment particles becomes in the case of an inclined pipe:

$$U_S = \beta\, (V - \sigma\, V_T) \qquad \text{(A.22)}$$

where V_T is the effective threshold velocity of the sediment in a nearly horizontal pipe, as given by Equation (A.8).

Equation (A.13) for the volumetric sediment concentration holds equally for the case of an inclined pipe, but a modification is required to the relationship between KE and PE that determines the distance, y_S, to which the particles can rise above the invert of the pipe. Since y_S is measured normal to the pipe invert but the potential energy depends on the vertical change in height, Equation (A.16) needs to be altered to:

$$y_S = \frac{es}{(s-1)\cos\theta} \left(\frac{U_S^2}{2g} \right) \qquad \text{(A.23)}$$

The analysis then proceeds in a similar way to that in Section 5.2.1 and gives the following result for the volumetric sediment concentration, C_{VI} , in an inclined pipe for particles travelling in flume traction:

$$C_{VI} = \left[\frac{\sqrt{2}}{3\,j}\, \beta^4 (e\,s)^{3/2} \right] \left[\frac{D^2}{A} \right] \left[1 - \frac{\sigma\, V_T}{V} \right]^4 \left[\frac{V^2}{g\,(s-1)\, D \cos\theta} \right]^{3/2} \qquad \text{(A.24)}$$

Comparing this result with Equation (A.19), it can be seen that, for similar flow conditions, the theoretical model suggests that the sediment concentration, C_{VI} , in an inclined pipe should be related to the corresponding value, C_{VO}, in a nearly horizontal pipe by:

$$\frac{C_{VI}}{C_{VO}} = \left[\frac{V - \sigma V_T}{V - V_T}\right]^4 \left[\frac{1}{\cos\theta}\right]^{3/2} \tag{A.25}$$

Thus, in principle, it might be expected that CIRIA Equation (A.1) for the limit of deposition could be modified to apply to inclined pipes if written in the new form:

$$C_{VI} = G\left(\frac{D^2}{A}\right)\left(\frac{d_{50}}{D}\right)^{0.6}\left[1 - \frac{\sigma V_T}{V}\right]^4 \left[\frac{V^2}{g\,(s-1)\,D\cos\theta}\right]^{3/2} \tag{A.26}$$

where the coefficient G may vary with pipe slope but has a value of $G = 3.03\text{x}10^{-2}$ in the case of a horizontal pipe. Since inverted siphons always flow full, the quantity (D^2/A) in Equation (A.26) can be replaced by a constant value of $4/\pi$. The factor σ can be expected to be predicted by Equation (A.21), with V_T being given by Equation (A.2), which for the case of a pipe flowing full can be written in the form:

$$V_T = 0.125\sqrt{g\,(s-1)\,d_{50}}\left(\frac{D}{d_{50}}\right)^{0.47} \tag{A.27}$$

The suitability of Equation (A.26) is evaluated in Section 5.3 from the experimental data described in Section 4.3.

5.2.3 Vertical pipes

As described in Section 4.2, there are two possible definitions of the limiting flow condition in a vertical pipe. The first is the velocity just needed to ensure upward movement of all the sediment particles in a straight section of pipe. (Note that at slightly lower velocities there will still be a net transport of sediment but with the upward movement of particles near the centre of the pipe being partly counterbalanced by the downward motion of particles near the walls). The second limiting value is the velocity required to prevent formation of a sediment deposit in the 90^o bend connected to the bottom of the vertical pipe. The second definition is the one adopted for this study and will normally always be more severe than the first definition.

The theoretical analysis given in Section 5.2.2 for inclined pipes can be expected to remain reasonably valid for sediment moving in a 90^o vertical bend provided the particles are not travelling in suspension and that the radius of curvature of the bend is not so tight as to generate flow separation or strong vertical accelerations. Therefore, it is possible that data relating to the second definition of the limiting flow conditions (ie formation of a deposit within the 90^o bend itself) could fit the pattern given by Equation (A.26) but with an effective value of pipe angle θ somewhat less than 90^o.

In the case of the first definition (ie maintenance of upward movement of particles in the vertical pipe), the theoretical model developed in Section 5.2.2 can be modified as follows. Assuming the particles are uniformly distributed over the cross-section of the pipe, the average drag force exerted on an individual particle is given by:

$$F_D = \frac{1}{2}\rho\, C_D\, \alpha_2\, d_{50}^2\, (V - U_S)^2 \tag{A.28}$$

where U_S is the average upward velocity of the particle. This is balanced by the immersed weight of the particle F_W given by:

$$F_W = \rho\, g\, (s-1)\, \alpha_3\, d_{50}^3 \tag{A.29}$$

so that:

$$U_S = V - \left[2\, g\, (s-1) \frac{\alpha_3}{\alpha_2} \frac{d_{50}}{C_D} \right]^{1/2} \tag{A.30}$$

The value of the drag coefficient, C_D , will depend on the relative proximity of other particles and the amount of shielding that they produce. Consider the analogous case of a single isolated particle falling through still water with a steady fall velocity, W_S. If the drag coefficient of the isolated particle is C_{DO} , it follows from a similar analysis that:

$$W_S = \left[2\, g\, (s-1) \frac{\alpha_3}{\alpha_2} \frac{d_{50}}{C_{DO}} \right]^{1/2} \tag{A.31}$$

Substituting in Equation (A.30) therefore gives:

$$U_S = V - W_S \left(\frac{C_{DO}}{C_D} \right)^{1/2} \tag{A.32}$$

The analysis relating to the volumetric sediment concentration, C_V , in Equations (A.9) to (A.13) of Section 5.2.1 also applies to sediment travelling in suspension. Since the particles occupy the full cross-sectional area of flow, $A_S = A$ so that Equation (A.13) becomes:

$$C_V = \frac{1}{j} \left(\frac{U_S}{V} \right) \tag{A.33}$$

where j is the ratio between the effective volume occupied by a particle and its actual volume. For upward flow in a vertical pipe, it follows from Equation (A.32) that:

$$C_V = \frac{1}{j} \left[1 - \left(\frac{W_S}{V} \right) \left(\frac{C_{DO}}{C_D} \right)^{1/2} \right] \tag{A.34}$$

As explained above, the quantity (C_{DO}/C_D) depends directly on the relative proximity of the particles, ie on the factor j, which also appears in Equation (A.34). It therefore follows that the limiting transport capacity of flow in a straight section of vertical pipe is likely to depend principally on the ratio between the average flow velocity, V, of the water and the fall velocity, W_S, of an individual sediment particle, ie:

$$C_V = \text{function} \left(\frac{W_S}{V} \right) \tag{A.35}$$

with other factors such as the size and shape of the particles or the diameter of the pipe having only a secondary influence on the relationship.

However, as mentioned previously, the flow velocity required to prevent the formation of a sediment deposit in the 90° bend at the bottom of the vertical pipe will normally always be greater than that corresponding to Equation (A.35).

5.3 Evaluation of experimental results

5.3.1 Effects of pipe slope and sediment size

The experimental data from Table A.1 (sand with d_{50} = 0.78mm) and Table A.2 (gravel with d_{50} = 4.3mm) are plotted in Figures 6 and 7 respectively to show the effect of pipe slope (θ) on the limiting flow velocity (V) needed to prevent deposition at different values of volumetric sediment concentration (C_V). [Note that for convenience in considering and applying the results, the same general symbol C_V will now be used for volumetric sediment concentration whether the pipe is horizontal, inclined or vertical].

Figures 8 and 9 show the same data from Table A.1 and A.2 plotted in a log-log format. For a given sediment type and inclination of pipe, it is apparent that the sediment concentration varied with approximately the third or fourth power of the limiting flow velocity. However, at low velocities the data points in the log-log plots are curved downwards, indicating the existence of a positive value of threshold velocity below which sediment transport will cease. It can also be seen from Figures 6 to 9 that the degree of scatter in the data was relatively small for a study involving sediment transport.

Both sediment types demonstrated a similar effect of pipe slope on limiting velocity. As would be expected, increasing the pipe slope upwards from the horizontal had the effect, initially, of increasing the velocity needed to transport a given concentration of sediment without deposition. However, it can also be seen from Figures 6 and 7 that the limiting velocity reached a maximum value at a pipe angle between about 22.5° and 45° and then started to decrease as the pipe slope was further increased. This is a significant result and one that needs to be reproduced by any theoretical model or design equation for inverted siphons.

Comparison of Figures 6 and 7 shows that, in general, the minimum velocity needed to transport a given concentration of the gravel without deposition was less than it was for the sand. This is perhaps contrary to what might be expected, but a similar behaviour has been observed with nearly horizontal pipes (see May, 1982) and is also demonstrated by Equation (A.1). The effect can be understood in terms of the relative volumes of the two types of sediment: in order to produce the same volumetric sediment concentration as one gravel particle (having a d_{50} size of 4.3 mm), it is necessary for the flow to be able to transport about 170 separate sand particles (with a d_{50} of 0.78 mm) at the same velocity without deposition. Another contributory factor is that, due to its larger size, a gravel particle will project further into the boundary layer at the pipe wall than a sand particle and so will experience a higher drag force. This inverse effect of particle size only occurs if the sediment is being transported in flume traction in contact with the invert of the pipe. At high enough flow velocities, the particles will begin to be lifted into suspension which is a more efficient mode of transport than flume traction. This will tend to increase the mobility of the smaller particles relative to the larger ones and reverse the trend that applies when they are all being transported in flume traction.

Figures 6 and 7 show that, for the sediment types tested, the limiting velocities needed for a vertical pipe are less than the equivalent values for nearly horizontal or inclined pipes. However, this result is dependent on the use of a long-radius bend at the bottom of the vertical pipe that enables a smooth transition to occur from flume traction upstream of the bend to suspended-load transport in the vertical pipe. If a sharp bend were used instead, significantly higher velocities would be needed to prevent the formation of a sediment deposit in the zone of flow separation at the bend.

When comparing the values of limiting velocity in inclined and vertical pipes, it also needs to be remembered that the criteria applied to the two cases are somewhat different. For an inclined pipe, the velocity needs to be sufficient to enable the sediment to continue moving in flume traction, without deposition, over an unlimited distance after the bend. For a vertical pipe, the velocity needs to be sufficient only to prevent deposition in the bend itself, where flow conditions are more non-uniform and turbulent than in a long straight length of inclined pipe. As mentioned in Section 4.2, avoiding deposition in the 90^o bend resulted in the flow velocity being above the minimum value required to lift the particles in suspension up the vertical pipe.

5.3.2 Threshold velocities

From the theoretical analysis described in Section 5.2.2, it was expected that the relationship between the mean flow velocity, V, and the sediment concentration, C_V , at the limit of deposition in upward sloping pipes could be of the form:

$$C_V = K\left[1 - \frac{V_{TI}}{V}\right]^4 V^3 \qquad \text{(A.36)}$$

where, according to Equation (A.26), the factor K is given by:

$$K = G\left(\frac{D^2}{A}\right)\left(\frac{d_{50}}{D}\right)^{0.6}\left[\,g\,(s-1)\,D\cos\theta\,\right]^{-3/2} \qquad \text{(A.37)}$$

G is a simple numerical coefficient that may vary with the angle of inclination of the pipe. V_{TI} is the effective threshold velocity of the sediment in an inclined pipe and, from Equation (A.21), is predicted to be related to the corresponding value, V_T , in a nearly horizontal pipe by the equation:

$$V_{TI} = V_T\left[\frac{\sin\theta + \mu\cos\theta}{\mu}\right]^{1/2} \qquad \text{(A.38)}$$

By writing Equation (A.36) in the form:

$$V = V_{TI} + \frac{1}{K^{1/4}}\left(C_V\,V\right)^{1/4} \qquad \text{(A.39)}$$

it was possible to use linear regression techniques to find the best-fit values of V_{TI} and K for each set of tests with a given pipe slope and sediment type; the results are given in Table A.3 and plotted in Figure 10. It is important to note that in the analysis the values of C_V were expressed as non-dimensional concentrations (eg a value of 50ppm was input to Equation (A.39) as $C_V = 50\times10^{-6}$).

The next step was then to determine from Equation (A.38) the values of the effective friction coefficient, μ, and the threshold velocity, V_T, that best fitted the data in Table A.3. The results for the two sediment types are given below, together with the values of V_T predicted independently from Equation (A.2). Data for the vertical pipe were not included in the regression.

Sediment type	Mean particle size d_{50} (mm)	Friction coefficient μ	Threshold velocity V_T (m/s)	
		From data	From data	Predicted from Eqn (A.2)
Sand	0.78	0.524	0.133	0.165
Gravel	4.3	0.410	0.180	0.174

It can be seen from the above results that the values of μ are in line with what would be expected for the coefficient of friction between sediment particles and the smooth walls of a pipe. Also, the estimated values of the threshold velocity are in fairly good agreement with those predicted by Equation (A.2), which was developed independently from a review of previous studies of sediment deposition in nearly horizontal pipes (see Section 5.1).

Figure 10 shows the variation of threshold velocity with pipe angle as predicted by Equation (A.38) using the above values of μ and V_T. Although there is some scatter in the best-fit values of V_{TI} from Table A.3, it can be seen that the form of the equation is able to account reasonably for the overall trends in the data. Figure 8 also includes the values of V_{TI} determined from the regression analysis for the case of the vertical pipe. It can be seen that extrapolation of Equation (A.38) gives a good estimate of V_{TI} in the case of the sand but a less accurate one in the case of the gravel. The level of agreement is of interest but may be coincidental since Equation (A.38) was derived theoretically for the case of flume traction and may not be directly applicable to suspended-load transport in a vertical pipe.

5.3.3 Correlation of data

In the next part of the analysis, the experimental data in Tables A.1 and A.2, together with the best-fit values of threshold velocity, V_{TI}, in Table A.3, were used to calculate the value for each test of the numerical coefficient G defined by Equation (A.26), ie:

$$G = C_V \left(\frac{A}{D^2}\right)\left(\frac{D}{d_{50}}\right)^{0.6} \left[\frac{V}{V - V_{TI}}\right]^4 \left[\frac{g\,(s-1)\,D\cos\theta}{V^2}\right]^{3/2} \qquad \text{(A.40)}$$

The results are included in Tables A.1 and A.2, together with the mean value of G and its standard deviation for each group of tests with a particular sediment type and pipe slope. It can be seen that the standard deviation within each group is low and generally about 10% of the mean value of G; it should be remembered that G is directly proportional to the value of sediment concentration. In the case of the tests with a vertical pipe, Tables A.1 and A.2 give values of the quantity $G/(\cos\theta)^{3/2}$ since it follows from Equation (A.40) that G will always have a value of zero at $\theta = 90^o$.

The variation of the mean value of G with the pipe gradient, $\sin\theta$, is shown for both sediments in Figure 11. Although there is some variability in the values, it can be seen that there is a well-defined trend for G to decrease with increasing pipe slope. The gradient of the data for the gravel is a little steeper than that of the sand, but for pipe slopes greater than 30^o there is no systematic difference between the behaviours of the two sediment types. The value of $G = 0.0297$ for sand at $\theta = 0^o$ is very close to the figure of $G = 0.0303$ in Equation (A.1), which was recommended for nearly horizontal pipes in CIRIA Report 141 (Ackers, Butler & May, 1996). In the case of the gravel, the corresponding value was found to be somewhat greater at $G = 0.0353$, although the difference is not too large compared with the variability normally found in studies of sediment transport.

The equation of the best-fit line for all the data points in Figure 11 was found to be:

$$G = 0.0311 - 0.0164 \sin\theta \qquad (A.41)$$

with a correlation coefficient of 0.645. The result obtained with the gravel at a slope of 30° appeared to be anomalous; omitting this data point from the regression then gave a revised best-fit equation of:

$$G = 0.0302 - 0.0168 \sin\theta \qquad (A.42)$$

with an improved correlation of 0.761. In order to maintain full consistency with Equation (A.1) for nearly horizontal pipes, it was decided to shift the datum very slightly and adopt the following as the recommended prediction equation for the coefficient G in Equation (A.26):

$$G = 0.0303 - 0.0169 \sin\theta \qquad (A.43)$$

The final form of the equation for predicting the limit of deposition in both horizontal and upward sloping pipes is therefore:

$$C_V = \left(0.0303 - 0.0169 \sin\theta\right)\left(\frac{D^2}{A}\right)\left(\frac{d_{50}}{D}\right)^{0.6}\left[1 - \frac{V_{TI}}{V}\right]^4\left[\frac{V^2}{g\,(s-1)\,D\cos\theta}\right]^{3/2} \qquad (A.44)$$

The method of applying this result is explained fully in Chapter 6 of this report.

As mentioned in Section 5.2.3, observations of the sediment behaviour in the vertical pipe indicated that the limit of deposition should be defined in terms of the flow conditions just needed to prevent a deposit forming in the 90° bend at the bottom of the vertical pipe. The mean values of the quantity $G/(\cos\theta)^{3/2}$ for the sand and the gravel were found to be 0.236 and 0.0771 respectively (see Tables A.1 and A.2). These values were compared with Equation (A.43) to determine the effective angle, θ_E , at which sediment deposition first occurred in the 90° bend at the bottom of the vertical pipe. From the results it was found that the critical angles were $\theta_E = 81°$ for the sand and $\theta_E = 71°$ for the gravel. This finding therefore makes it possible to apply Equation (A.44) to vertical pipes, as also explained in Chapter 6. However, the extension of the results to vertical pipes is only valid in cases where the relative radius of the bend is large enough to allow a smooth transition from flume traction at the start of the bend to suspended-load transport in the vertical pipe.

5.4 Recommendations for further study

Little experimental research appears to have been carried out on sediment movement in inverted siphons prior to this study. It was not possible to investigate all aspects of the problem in this single project, and further research on the following topics is recommended.

- Validation of the design equations developed in this study using experimental data for larger pipes.
- Investigation of the effect of wide sediment gradings on the limit of deposition in inclined pipes.
- Study of the effect of bend radius on deposition at the bottom of vertical pipes.
- Determination of the flow conditions governing the transition between bed-load transport and suspended-load transport of sediment in inclined pipes.
- Study of the behaviour of inverted sewer siphons taking account of the physical properties of real sewer sediments (eg additional effects due to cohesion and biological slimes). Controlled laboratory experiments using real sewer sediments would be difficult to carry out because of handling and scaling problems, but data on the actual performance of constructed siphons could usefully be compared with design predictions based on tests with non-cohesive sediments.

Table A.1 Data on limit of deposition for tests with sand

Mean sediment size: d_{50} = 0.78 mm
Specific gravity of sediment: s = 2.62
Internal diameter of pipe: D = 0.150 m

Angle of pipe θ	Mean flow velocity V (m/s)	Volumetric sediment concentration C_V (ppm)	Value of G from Eqn (A.40)
0^o	0.490	13.4	3.07×10^{-2}
	0.506	14.4	2.84×10^{-2}
	0.518	14.8	2.63×10^{-2}
	0.558	25.4	3.24×10^{-2}
	0.604	33.1	3.00×10^{-2}
	0.710	68.2	3.18×10^{-2}
	0.825	115	2.97×10^{-2}
	0.967	194	2.76×10^{-2}
	1.050	291	3.06×10^{-2}
			mean G = 2.97×10^{-2} sd of G = 0.20×10^{-2}
15^o	0.530	12.9	2.62×10^{-2}
	0.542	12.0	2.19×10^{-2}
	0.610	19.1	2.00×10^{-2}
	0.620	22.1	2.15×10^{-2}
	0.794	59.7	2.01×10^{-2}
	0.935	122	2.14×10^{-2}
	1.060	178	1.93×10^{-2}
	1.124	273	2.38×10^{-2}
			mean G = 2.18×10^{-2} sd of G = 0.23×10^{-2}
22.5^o	0.534	13.1	1.90×10^{-2}
	0.629	20.9	1.48×10^{-2}
	0.715	43.1	1.79×10^{-2}
	0.715	42.0	1.75×10^{-2}
	0.790	56.6	1.58×10^{-2}
	0.920	113	1.76×10^{-2}
	1.076	177	1.55×10^{-2}
	1.163	269	1.77×10^{-2}
			mean G = 1.70×10^{-2} sd of G = 0.14×10^{-2}

Table A.1 Data on limit of deposition for tests with sand (continued)

Angle of pipe θ	Mean flow velocity V (m/s)	Volumetric sediment concentration C_V (ppm)	Value of G from Eqn (A.40)
30°	0.574	12.6	1.91×10^{-2}
	0.621	20.3	2.09×10^{-2}
	0.760	45.5	1.85×10^{-2}
	0.785	57.0	2.02×10^{-2}
	0.809	59.1	1.84×10^{-2}
	0.920	114	2.07×10^{-2}
	0.927	116	2.05×10^{-2}
	1.045	177	1.94×10^{-2}
	1.069	182	1.82×10^{-2}
	1.163	276	2.00×10^{-2}
			mean G = 1.96×10^{-2} sd of G = 0.10×10^{-2}
45°	0.542	14.2	1.47×10^{-2}
	0.597	25.9	1.72×10^{-2}
	0.711	49.4	1.55×10^{-2}
	0.760	65.9	1.57×10^{-2}
	0.857	119	1.75×10^{-2}
	1.014	193	1.49×10^{-2}
	1.100	282	1.61×10^{-2}
			mean G = 1.59×10^{-2} sd of G = 0.11×10^{-2}
60°	0.511	16.2	2.55×10^{-2}
	0.560	27.9	2.61×10^{-2}
	0.668	52.7	1.99×10^{-2}
	0.710	70.2	1.97×10^{-2}
	0.817	133	1.97×10^{-2}
	0.896	214	2.12×10^{-2}
	0.951	322	2.49×10^{-2}
			mean G = 2.24×10^{-2} sd of G = 0.29×10^{-2}

	V (m/s)	C_V (ppm)	$G/(\cos\theta)^{3/2}$
90°	0.409	17.8	0.218
	0.456	36.1	0.226
	0.511	87.3	0.288
	0.534	87.7	0.229
	0.613	168	0.220
	0.676	281	0.233
	0.739	427	0.237
			mean = 0.236 sd = 0.024

Table A.2 Data on limit of deposition for tests with gravel

Mean sediment size	d_{50}	= 4.3 mm
Specific gravity of sediment:	s	= 2.63
Internal diameter of pipe:	D	= 0.150 m

Angle of pipe θ	**Mean flow velocity** V (m/s)	**Volumetric sediment concentration** C_V (ppm)	**Value of G from Eqn (A.40)**
0°	0.432	14.5	2.81×10^{-2}
	0.451	22.5	3.48×10^{-2}
	0.527	54.5	3.89×10^{-2}
	0.560	80.2	4.31×10^{-2}
	0.668	142	3.50×10^{-2}
	0.760	223	3.21×10^{-2}
	0.825	338	3.50×10^{-2}
			mean G = 3.53×10^{-2} sd of G = 0.48×10^{-2}
15°	0.495	12.7	2.68×10^{-2}
	0.503	17.2	3.28×10^{-2}
	0.589	42.1	3.22×10^{-2}
	0.640	70.1	3.46×10^{-2}
	0.743	127	3.01×10^{-2}
	0.743	132	3.13×10^{-2}
	0.817	202	3.09×10^{-2}
	0.904	313	3.07×10^{-2}
			mean G = 3.12×10^{-2} sd of G = 0.23×10^{-2}
30°	0.558	13.6	3.61×10^{-2}
	0.597	20.8	3.34×10^{-2}
	0.650	42.1	3.78×10^{-2}
	0.687	60.2	3.81×10^{-2}
	0.786	104	3.00×10^{-2}
	0.848	192	3.68×10^{-2}
	0.935	304	3.55×10^{-2}
			mean G = 3.54×10^{-2} sd of G = 0.28×10^{-2}
37.5°	0.550	13.4	1.38×10^{-2}
	0.585	21.4	1.53×10^{-2}
	0.656	47.5	1.81×10^{-2}
	0.695	65.6	1.85×10^{-2}
	0.802	126	1.76×10^{-2}
	0.896	199	1.67×10^{-2}
	0.990	292	1.59×10^{-2}
	1.000	308	1.60×10^{-2}
			mean G = 1.65×10^{-2} sd of G = 0.15×10^{-2}

Table A.2 Data on limit of deposition for tests with gravel (continued)

Angle of pipe θ	Mean flow velocity V (m/s)	Volumetric sediment concentration C_V (ppm)	Value of G from Eqn (A.40)
45°	0.558	11.1	1.81×10^{-2}
	0.621	28.5	2.21×10^{-2}
	0.664	49.2	2.49×10^{-2}
	0.699	65.3	2.43×10^{-2}
	0.786	113	2.18×10^{-2}
	0.880	200	2.15×10^{-2}
	0.959	303	2.14×10^{-2}
			mean G = 2.20×10^{-2} sd of G = 0.22×10^{-2}
60°	0.550	14.0	1.24×10^{-2}
	0.570	22.4	1.55×10^{-2}
	0.652	51.6	1.51×10^{-2}
	0.691	64.1	1.33×10^{-2}
	0.770	130	1.50×10^{-2}
	0.864	214	1.37×10^{-2}
	0.940	330	1.41×10^{-2}
			mean G = 1.42×10^{-2} sd of G = 0.11×10^{-2}

	V	C_V	$G/(\cos\theta)^{3/2}$
90°	0.430	17.5	7.89×10^{-2}
	0.460	30.1	8.71×10^{-2}
	0.530	55.9	6.94×10^{-2}
	0.566	83.9	7.26×10^{-2}
	0.636	164	7.78×10^{-2}
	0.707	263	7.49×10^{-2}
	0.760	388	7.92×10^{-2}
			mean = 7.71×10^{-2} sd = 0.56×10^{-2}

Table A.3 Best-fit values of threshold velocity, V_{TI}, and quantity K in Equation (A.36)

Sediment type	Angle of pipe θ	Threshold velocity V_{TI} (m/s)	K $(m/s)^{-3}$
Sand (d_{50} = 0.78 mm)	0^o	0.143	4.373×10^{-4}
	15^o	0.170	3.341×10^{-4}
	22.5^o	0.148	2.808×10^{-4}
	30^0	0.195	3.576×10^{-4}
	45^o	0.161	3.961×10^{-4}
	60^o	0.214	9.202×10^{-4}
	90^o	0.191	3.496×10^{-3}
Gravel (d_{50} = 4.3 mm)	0^o	0.160	1.432×10^{-3}
	15^o	0.223	1.333×10^{-3}
	30^o	0.304	1.775×10^{-3}
	37.5^0	0.240	9.550×10^{-4}
	45^o	0.292	1.509×10^{-3}
	60^o	0.279	1.632×10^{-3}
	90^o	0.210	3.115×10^{-3}

Part B – Guidelines for Hydraulic Design

6. APPLICATION OF HR EXPERIMENTAL RESULTS

6.1 General

This Chapter explains how to apply the results of the HR experimental study (see Part A) to determine suitable values of minimum velocity for ensuring self-cleansing conditions in inverted siphons.

Section 6.2 lists the design equations obtained from the analysis of the experimental data described in Chapter 5. Section 6.3 gives recommended data values for the design of inverted siphons, with particular emphasis on those used in sewerage systems. For other types of siphon, eg those carrying river or irrigation flows, field measurements should be made to determine suitable design values for the sediment characteristics and concentration. In Section 6.4 the procedure for calculating minimum velocities is illustrated by means of worked examples. “Indicative” values of minimum velocity for inverted sewer siphons are given in the general design recommendations in Chapter 7 of this report (see Tables B.1 and B.2 in Section 7.9); the derivation of these indicative values is explained in Section 6.5.

6.2 Design equations for minimum velocity

The limit of deposition in inverted siphons is defined in this report as the velocity below which there will be a transition from sediment travelling as individual particles in flume traction to movement as separated dunes along the invert (see Section 3.2). In the case of sewer sediments containing a wide range of particle sizes and densities, the limit of deposition will be determined by the larger, inorganic component of the material. The best-fit equation for determining the limit of sediment deposition in horizontal or upward sloping pipes of inverted siphons is Equation (A.44). For purposes of design, it is recommended to incorporate a safety factor of 10% into the design value of minimum velocity to account for scatter in the original experimental data and uncertainties in estimating quantities such as the sediment concentration and representative particle size. With this safety factor and assuming pipe-full flow, Equation (A.44) can be written in the form:

$$C_V = \left(0.0303 - 0.0169 \sin\theta\right)\left(\frac{4}{\pi}\right)\left(\frac{d}{D}\right)^{0.6}\left[1 - \frac{1.1\,V_{TI}}{V}\right]^4\left[\frac{V^2}{1.21\,g\,(s-1)\,D\cos\theta}\right]^{3/2} \qquad \text{(B.1)}$$

where:

- C_V is the volumetric sediment concentration (non-dimensional) and defined as the volumetric transport rate of sediment, Q_S (in m^3/s), divided by the volumetric flow rate of the water, Q (in m^3/s) ;
- θ is the angle of upward inclination of the pipe (in ^{o});
- D is the internal diameter of the pipe (in m);
- d is the “representative” particle size (in m) of the larger, inorganic component of the sediment ($\geq 0.6\times10^{-3}$m or 0.6mm, see Section 6.3.1);
- V is the mean velocity (in m/s) of the water in the pipe ($= 4Q/\pi D^2$);
- V_{TI} is the threshold value of V (in m/s) at which individual particles of the sediment will first begin to move along the invert of an inclined pipe;
- g is the acceleration due to gravity ($= 9.81 m/s^2$);
- s is the specific gravity of the sediment particles (non-dimensional);.

The value of C_V should be expressed as a non-dimensional number (eg $C_V = 50\times10^{-6}$, not 50ppm).

The threshold velocity, V_{TI}, in an inclined pipe is related to the corresponding value, V_T, in a horizontal (or nearly horizontal) pipe by Equation (A.38):

$$V_{TI} = V_T \left[\frac{\sin\theta + \mu \cos\theta}{\mu} \right]^{1/2} \qquad (B.2)$$

where μ (non-dimensional) is the effective coefficient of friction between the sediment particles and the walls of the pipe. The value of V_T in a horizontal (or nearly horizontal) pipe is given by Equation (A.27):

$$V_T = 0.125 \sqrt{g\,(s-1)\,d} \left(\frac{D}{d} \right)^{0.47} \qquad (B.3)$$

The units for the quantities should be the same as for Equation (B.1).

The above results assume that the siphon pipes are of circular cross-section. Experimental data and design equations are not available for predicting self-cleansing conditions in upward-sloping conduits of square or rectangular cross-section. In the absence of such information, an approximate estimate of the minimum velocity may be obtained from Equations (B.1) to (B.3) using an equivalent pipe diameter $D_E = \sqrt{(4A/\pi)}$, where A is the cross-sectional flow area of the square or rectangular conduit.

6.3 Data values for the design of inverted siphons

6.3.1 Sediment characteristics (d and s)

Foul sewage and surface water sewage typically contain a wide range of sediment types in terms of particle size, density and degree of cohesiveness. In the case of inverted sewer siphons, the most critical requirement will normally be to ensure that the larger, inorganic particles can be transported through the pipes without forming deposits.

When designing a new inverted sewer siphon or checking the performance of an existing one, data should be obtained where possible on the types of sediment that typically occur in the sewerage system. The "representative" particle size, d, used in the design equations should correspond to the grit fraction (coarse sands and gravel) which, in gravity systems, will normally only be transportable as bed load. Thus, d should be the mean size of the material that is larger than 0.6 mm. As an example, if a grading curve is such that 0.6mm corresponds to the d_{72} particle size, then the representative particle size, d, should be taken as equal to the d_{86} size in the grading (since d_{86} is the mean size between d_{72} and d_{100}). If the amount of material in this upper band of the grading is small or non-existent, it is recommended to assume a minimum value of d = 0.6mm (ie $d = 0.6\times10^{-3}$ m in Equations (B.1) and (B.3)).

If no site-specific data are available, it is recommended for UK conditions to assume the following values of representative particle size, d, for inverted sewer siphons:

Type of sewer	**Particle size d (m)**
Foul	0.6×10^{-3}
Combined	1.5×10^{-3} to 6.0×10^{-3} *
Surface water	1.5×10^{-3} to 6.0×10^{-3} *

Note: * Depending on catchment characteristics

An appropriate value of specific gravity for the coarser, inorganic fractions of sewer sediments is s = 2.6.

For siphons carrying river or irrigation flows, field measurements should be made to determine suitable design values for the sediment characteristics.

6.3.2 Friction coefficient (μ)

In order to calculate the value of the effective threshold velocity, V_{TI}, it is necessary to assume a value of the friction coefficient, μ, between the particles and the walls of the pipe. [Note: this will normally be less than tan γ, where γ is the angle of repose of the sediment, since the latter relates to particles sliding or rolling over other particles].

Analysis of the HR experimental data (see Section 5.3.2) showed that for the case of a plastic pipe the best-fit values of the friction coefficient were μ = 0. 52 for the 0.78 mm sand and μ = 0.41 for the 4.3 mm gravel. It can be seen from Equation (B.2) that reducing the value of μ has the effect of increasing the self-cleansing velocity needed in an inclined pipe. However, the walls of actual siphon pipes are likely to be somewhat rougher than those of the pipe used in the laboratory study.

Taking account of the above factors, it is therefore recommended to use a value of μ = 0.45 for design.

6.3.3 Sediment concentration (C_V)

For inverted sewer siphons, data on typical sediment concentrations in the sewerage system should be obtained where possible, eg from records of grit removal at a downstream treatment works. In the absence of such data, it is recommended in the case of UK sewers to assume the following design values, which are in accordance with the recommendations in CIRIA Report 141 (Ackers, Butler & May, 1996):

Sediment load in sewer	C_V
"Low" category	3.8×10^{-6}
"Medium" category	19×10^{-6}
"High" category	77×10^{-6}

Note : The above values of C_V are equivalent to concentrations by mass of 10 mg/l, 50 mg/l and 200 mg/l respectively, assuming a particle specific gravity of s = 2.6.

The "low"category will normally not be appropriate for siphons in combined sewers or separate surface water sewers unless an effective sediment removal system is located just upstream of the siphon.

For siphons carrying river or irrigation flows, field measurements or sediment transport calculations should be carried out to determine suitable design values of sediment concentration.

6.3.4 Vertical pipes

The analysis of the experimental data in Section 5.3.3 showed that Equations (B.1) to (B.3) for inclined pipes can also be used to determine the limiting velocity in the upward vertical leg of an inverted siphon. This is because any deposition will tend to occur first within the 90^o bend at the bottom of the vertical section of pipe. The effective slope angle, θ_E, corresponding to the point of deposition was found in the tests to be 81° for the sand and 71° for the gravel. For design purposes, it is recommended to assume a value of $\theta_E = 70°$ since this is likely to err on the conservative side.

Application of Equations (B.1) to (B.3) to vertical pipes is only appropriate if the 90° bends have a large enough radius of curvature to allow the mode of movement of the sediment to change smoothly from flume traction at the start of the bend to suspended-load transport in the vertical pipe. This requirement was achieved in the HR tests using a bend with a radius of curvature equal to 6.7× the diameter of the pipe (measured to the centreline of the bend). Information on the effect of using tighter bends is not available but, for design purposes, it is suggested that a minimum radius of curvature equal to 5× the pipe diameter could be expected to be satisfactory. In the case of a downward vertical leg, the 90° bend used at the bottom can be of tighter radius because there is no significant danger of sediment deposition at that location.

6.3.5 Head losses in pipework

The total head loss, H_T, between the inlet and outlet chambers of a siphon is made up of two components: the overall frictional loss, H_F, along the pipe; and the sum of point losses, H_P, at bends and other fittings and at the inlet and outlet of the pipe, ie:

$$H_T = H_F + H_P \tag{B.4}$$

where the values of head are all in m.

The presence of sediment in a flow will increase the frictional resistance of a pipe compared with a similar flow without sediment. Experiments by May (1982) indicated that the increase could be of the order of 5% to 10% provided the sediment was transported in flume traction without deposition. For assessing head losses in an inverted sewer siphon designed to be self-cleansing, it is therefore recommended to allow for a hydraulic gradient that is 10% greater than that produced by a similar flow without sediment.

The frictional loss for pipe-full flow without sediment can be calculated from the Colebrook-White equation:

$$\frac{V}{\sqrt{2\,g\,D\,i}} = -2\log_{10}\left[\frac{k_S}{3.71\,D} + \frac{2.51\,\nu}{D\sqrt{2\,g\,D\,i}}\right] \tag{B.5}$$

where:

- i is the head loss gradient along the pipe (m loss of head per m length of pipe);
- k_S is the roughness value (in m) for the pipe walls;
- ν is the kinematic viscosity of the liquid (eg = 1.30×10^{-6} m^2/s for water or sewage at 10°C).

Values of pipe wall roughness recommended in Sewers for Adoption (Water Authorities Association, 1989) are k_S = 0.6mm for foul sewers and k_S = 1.5mm for surface water and combined sewers. Similar figures should be suitable for inverted siphons of modern construction, but higher values may be appropriate when checking the performance of existing older siphons.

The overall frictional loss along the siphon is then calculated from the following equation using the value of i given by Equation (B.5):

$$H_F = 1.10 \sum \left(i\, L_P \right) \tag{B.6}$$

where L_P is the length (in m) of pipe of a given internal diameter, D. As described above, the factor 1.10 takes account of the additional resistance caused by the transport of the sediment.

Point energy losses in pipe systems are normally calculated using appropriate values of a non-dimensional loss coefficient, ξ, related to the velocity head, $V^2/2g$, of the flow in the pipe, ie:

$$H_P = \sum \xi \left(\frac{V^2}{2g} \right) \tag{B.7}$$

Typical values of ξ for various types of pipe fitting are given below; more detailed information on losses in pipe fittings can be found in publications such as Miller (1990) and Idelchik (1986). If a siphon has been designed to be self-cleansing, the presence of sediment in the flow should not produce any significant increase in the values of ξ for point losses.

Type of pipe fitting	**Head loss coefficient ξ**
Sharp-edged inlet	0.5
Radiused inlet	0.05
Sharp-edged outlet	1.0
Gate valve (fully open)	0.2
90^o bend*	0.2
45^o bend*	0.15
30^o bend*	0.1
15^o bend*	0.05

Note: * assuming centreline radius of curvature = 5× pipe diameter

6.4 Calculation procedure

6.4.1 Design example

An inverted sewer siphon is to be constructed under a wide river to carry flow from an existing combined sewerage system to a new treatment works. The overall length of the pipe barrel between the inlet and the outlet chambers on opposite banks of the river will be 80m. The siphon is required to operate under gravity with a maximum allowable difference in water level between the two ends of the pipe of 1.5m. The average dry weather flow (DWF) through the siphon will be 0.35 m^3/s, with twice-daily peaks equal to about 2×DWF.

Data on sediment concentrations in the existing sewerage network are not available but it is known from maintenance records that sediment deposition is a significant problem in the flatter parts of the system. It is therefore decided to categorise the sediment load as “high” and assume a design sediment concentration of $C_V = 77\times10^{-6}$ (see Section 6.3.3). Analysis of sediment samples of material deposited in pipes and chambers shows that there is a significant proportion of coarse sand and grit. Following the definition of representative particle size, d, given in Section 6.3.1, it is found that the material coarser than 0.6 mm has a mean size of about 2.8 mm. For design purposes, it is therefore decided to adopt a value of $d = 3.0\times10^{-3}$ m with a corresponding specific gravity of s = 2.6.

Two different layouts for the inverted siphon are to be investigated: one with the upward and downward sloping legs making an angle of 15^o to the horizontal; and the other with vertical upward and downward legs. It is therefore necessary to determine values of minimum self-cleansing velocity for the nearly horizontal length of the pipe under the river and for the upward leg with alternative angles of $\theta = 15^o$ and 90^o.

To cater for the full range of dry weather and storm flows, it will be necessary to use a multi-barrel siphon. The first stage in the hydraulic design of the system is to determine an appropriate diameter for the low-flow barrel of the siphon so that self-cleansing flows will occur frequently enough to prevent sediment deposits from accumulating. The form of the design equations (B.1) to (B.3) makes it difficult to determine the required pipe diameter directly. The procedure adopted in the following Sections therefore involves assuming a value of pipe diameter and determining the minimum velocity required to prevent deposition; the corresponding value of self-cleansing discharge can then be compared with the actual flow rates in the system. In the example described, the assumed diameter for the low-flow barrel of the siphon is 0.610m.

6.4.2 Horizontal pipe

(a) The first step is to calculate the threshold velocity, V_T, of the sediment in a nearly horizontal pipe. Using Equation (B.3) gives:

$$V_T = 0.125\sqrt{g\,(s-1)\,d}\left(\frac{D}{d}\right)^{0.47} = 0.125\sqrt{9.81\times(2.6-1)\times 3.0\times10^{-3}}\left(\frac{0.610}{3.0\times10^{-3}}\right)^{0.47} = 0.330\text{ m/s}$$

(b) The limiting velocity, V, needed to prevent sediment deposition is determined from Equation (B.1). For a horizontal pipe flowing full, the pipe angle $\theta = 0^o$ and V_T is equivalent to V_{TI}. It follows that:

$$C_V = 0.0303\left(\frac{4}{\pi}\right)\left(\frac{d}{D}\right)^{0.6}\left[1-\frac{1.1\,V_T}{V}\right]^4\left[\frac{V^2}{1.21\,g\,(s-1)\,D}\right]^{3/2}$$

Substituting into the equation the design data and the value of V_T from (a) gives:

$$77\times10^{-6} = 0.0303\times\left(\frac{4}{\pi}\right)\times\left(\frac{3.0\times10^{-3}}{0.610}\right)^{0.6}\times\left[1-\frac{1.1\times0.330}{V}\right]^4\times\left[\frac{V^2}{1.21\times9.81\times(2.6-1)\times0.610}\right]^{3/2}$$

which is equivalent to:

$$1.909 = \left[1-\frac{0.363}{V}\right]^4 V^3$$

This equation requires a trial-and-error technique to find the value of V. One method of doing this manually is to write the equation in the form:

$$V_2 = 1.909^{1/3}\times\left[1-\frac{0.363}{V_1}\right]^{-4/3}$$

where V_1 is an initial estimate of V, and V_2 is an improved estimate. Guess a first value of V_1 (say, 1.0 m/s) and use the equation to calculate V_2. Replace the old value of V_1 by a new estimate given by:

$$\text{new } V_1 = 0.5 \times (\text{ old } V_1 + V_2\ )$$

Then repeat the process until the difference between V_1 and V_2 is small enough to neglect. In this way it can be shown that the solution of the above equation is V = 1.71 m/s. As explained in Section 6.2, this estimate of the self-cleansing velocity includes a safety factor of 10%.

(c) The corresponding minimum self-cleansing discharge for the horizontal section of the 0.610m diameter siphon is:

$$Q = \frac{\pi}{4} D^2\ V = \frac{\pi}{4} \times 0.610^2 \times 1.71 = \ 0.500\, m^3/s\,.$$

6.4.2 Inclined pipe

(a) The threshold velocity, V_{TI}, of the sediment moving up the 0.610 m diameter pipe at a slope angle of $\theta = 15^o$ is determined from Equation (B.2) and the value of $V_T = 0.330$ m/s found in step (a) of Section 6.4.2. Based on the recommendation in Section 6.3.2, a friction coefficient of $\mu = 0.45$ will be assumed. Substituting in Equation (B.2) gives:

$$V_{TI} = V_T \left[\frac{\sin\theta + \mu\cos\theta}{\mu} \right]^{1/2} = 0.330 \times \left[\frac{\sin 15^o\ + 0.45 \times \cos 15^o}{0.45} \right]^{1/2} = 0.410\, m/s$$

(b) The limiting velocity, V, is found from Equation (B.1):

$$C_V = \left(0.0303\ \text{-}\ 0.0169 \sin\theta\right) \left(\frac{4}{\pi}\right) \left(\frac{d}{D}\right)^{0.6} \left[\ 1\ \text{-}\ \frac{1.1\, V_{TI}}{V} \right]^4 \left[\frac{V^2}{1.21\, g\, (s\,\text{-}1)\, D\cos\theta} \right]^{3/2}$$

Substituting in the value of V_{TI} and the other design data gives:

$$77 \times 10^{-6} = \ (0.0303\,\text{-}\,0.0169 \sin 15^o) \times \left(\frac{4}{\pi}\right) \times \left(\frac{3.0 \times 10^{-3}}{0.610}\right)^{0.6}$$

$$\times \left[\ 1\ \text{-}\ \frac{1.1 \times 0.410}{V} \right]^4 \times \left[\frac{V^2}{1.21 \times 9.81 \times (2.6\,\text{-}1) \times 0.610 \times \cos 15^o} \right]^{3/2}$$

which is equivalent to:

$$2.119 = \left[\ 1\ \text{-}\ \frac{0.451}{V} \right]^4 V^3$$

Using the trial-and-error technique described in step (b) of Section 6.4.2, it can be shown that the solution of this equation is V = 1.86 m/s. As explained in Section 6.2, this estimate of the self-cleansing velocity includes a safety factor of 10%.

(c) The corresponding minimum self-cleansing discharge for the upward leg of the 0.610m diameter siphon is:

$$Q = \frac{\pi}{4} D^2 \ V = \frac{\pi}{4} \times 0.610^2 \times 1.86 = \ 0.544 \, m^3 / s$$

Comparing these results with the values in Section 6.4.3 for the horizontal section of the siphon barrel, it can be seen that the effect of the 15° upward slope of the pipe is to increase the value of minimum self-cleansing velocity by 9.4%.

6.4.3 Vertical pipe

(a) As explained in Section 6.3.4, the critical point for sediment deposition in the vertical leg of a siphon is located within the 90° bend at the base of the vertical pipe. The limiting velocity, V, can be determined from Equations (B.1) and (B.2) using a value of equivalent pipe slope of $\theta_E = 70°$. Therefore, using the value of $V_T = 0.330$ m/s found in step (a) and the recommended value of friction coefficient of μ = 0.45, it is found from Equation (B.2) that:

$$V_{TI} = V_T \left[\frac{\sin\theta + \mu\cos\theta}{\mu} \right]^{1/2} = 0.330 \times \left[\frac{\sin 70° + 0.45 \times \cos 70°}{0.45} \right]^{1/2} = 0.514 \text{ m/s}$$

(b) The limiting velocity, V, is found from Equation (B.1):

$$C_V = (0.0303 \ - \ 0.0169 \sin\theta) \left(\frac{4}{\pi} \right) \left(\frac{d}{D} \right)^{0.6} \left[1 \ - \ \frac{1.1\, V_{TI}}{V} \right]^4 \left[\frac{V^2}{1.21\, g\, (s - 1)\, D \cos\theta} \right]^{3/2}$$

Substituting in the value of V_{TI} and the other design data gives:

$$77 \times 10^{-6} = (0.0303 - 0.0169 \sin 70°) \times \left(\frac{4}{\pi} \right) \times \left(\frac{3.0 \times 10^{-3}}{0.610} \right)^{0.6}$$
$$\times \left[1 \ - \ \frac{1.1 \times 0.514}{V} \right]^4 \times \left[\frac{V^2}{1.21 \times 9.81 \times (2.6 - 1) \times 0.610 \times \cos 70°} \right]^{3/2}$$

which is equivalent to:

$$0.803 = \left[1 \ - \ \frac{0.565}{V} \right]^4 V^3$$

Using the trial-and-error technique described in step (b) of Section 6.4.3, it can be shown that the solution of this equation is V = 1.64 m/s. As explained in Section 6.2, this estimate of the self-cleansing velocity includes a safety factor of 10%.

(c) The minimum velocity for the vertical pipe is slightly less than the value of V = 1.71 m/s that was found in Section 6.4.2 to apply to the horizontal leg of the siphon. Therefore, ensuring that deposition will not occur in the horizontal pipe will also prevent any deposition in the vertical pipe or in the 90° bend at its base. However, this conclusion is dependent on the 90^0 bend having a radius of curvature (on its centreline) of not less than 5× the pipe diameter, ie not less than about 3 m (see Section 6.3.4).

6.4.4 Head losses in siphon

For structural and maintenance reasons it has been decided to construct the siphon with the upward and downward legs at an angle of $\theta = 15^0$ to the horizontal. Therefore, as shown in Section 6.4.3, the minimum flow rate needed to maintain self-cleansing flow conditions in the 0.610 m diameter pipe is 0.544 m^3/s. The average dry weather flow (DWF) through the siphon will be about 0.35 m^3/s. Although some deposition may occur for short periods each day, the twice-daily peaks of about 2×DWF (ie 0.7 m^3/s) should ensure satisfactory flushing and remove any temporary small deposits of sediment. The use of a 0.610 m diameter pipe is likely to be the optimum choice for the low-flow barrel of the siphon. A smaller pipe would result in higher flow velocities and produce unnecessary extra head losses. The next larger size of standard pipe would require a self-cleansing discharge that was greater than 2×DWF so there would be a risk of a build-up of deposits during periods of dry weather.

The head loss between the inlet and outlet chambers of the siphon corresponding to the self-cleansing discharge of 0.544 m^3/s is made up of two components: frictional losses along the pipe; and point losses at the bends and at the inlet and outlet chambers.

For combined sewers of modern construction, it is suggested in Section 6.3.5 to assume a roughness value of k_S = 1.5 mm (ie 1.5×10^{-3} m). It can be found from Equation (B.5) that the head loss gradient in a 0.610 m diameter pipe of this roughness will be i = 0.00721 when the flow velocity V= 1.86 m/s (corresponding to the self-cleansing discharge of Q = 0.544 m^3/s). The overall head loss along the 80m length of the inverted siphon is given by Equation (B.6) which includes an allowance of an additional 10% loss due to the presence of the sediment:

$$H_F = 1.10\, i\, L_P = 1.10 \times 0.00721 \times 80 = 0.634\ \mathrm{m}$$

Point head losses in the siphon are calculated from Equation (B.7). Suitable values of head loss coefficient for the present case (see Section 6.3.5) are: ξ_I = 0.5 for a sharp-edged entry in the inlet chamber; ξ_O = 1.0 for the exit loss in the outlet chamber; and ξ_B = 0.05 for each of the two 15° bends. The overall loss factor for the siphon barrel is therefore $\Sigma\ \xi$ = 1.6 so substituting in Equation (B.7) gives:

$$H_P = 1.6 \times \left(\frac{1.86^2}{2 \times 9.81} \right) = 0.282\ \mathrm{m}$$

The total head loss across the 0.610 m diameter barrel at the self-cleansing discharge of Q = 0.544 m^3/s is found from Equation (B.4) to be:

$$H_T = H_F + H_P = 0.634 + 0.282 = 0.916\ \mathrm{m}$$

The maximum discharge that can be carried by the 0.610 m diameter pipe is determined by the maximum allowable head loss of 1.5m between the inlet and outlet chambers of the siphon. An estimate of the maximum discharge, Q_M, can be obtained using the fact that the total head loss is approximately proportional to the square of the discharge. Therefore, it follows that:

$$Q_M = 0.544 \times \left(\frac{1.5}{0.916} \right)^{1/2} = 0.696\ \mathrm{m}^3/\mathrm{s}$$

This maximum flow rate is equal to twice the dry weather flow rate of 0.35 m^3/s. Suitable sizes for the other barrels will need to be determined in a similar way so that the siphon can cope satisfactorily with higher flow rates occurring during wet weather and heavy storms. However, in these cases, it would be appropriate to design for a "medium" category of sediment concentration (see Section 6.3.3) since most of the larger inorganic sediments should continue to be carried by the low-flow barrel.

6.5 Indicative values of minimum velocity

When carrying out initial designs of sewer siphons or developing general guidelines, it is convenient to have a set of indicative values of minimum self-cleansing velocity that can be applied generally, although in some cases the recommendations may err somewhat on the conservative side.

The HR experimental results and Equations (B.1) to (B.3) show that the worst-case condition for deposition in inverted siphons occurs at an upward slope of between about 22.5^{o} and 45^{o}. Therefore, values of minimum velocity calculated for a value of $\theta = 37.5^{0}$ and with a safety factor of 10% (see Section 6.3.6) will be conservative for all angles of inclination. In the case of a vertical pipe, it can be shown from Equations (B.1) to (B.3) that the minimum velocity will be a little less than that needed to prevent deposition in a horizontal section of the pipe.

The particle size, d, has an effect on the minimum velocity required to transport a given concentration of sediment without deposition. As explained in Section 5.3.1, finer particles are harder to transport in flume traction than larger particles. However, once the flow velocity is high enough to transport the sediment in suspension, the trend will be reversed and the finer particles will become more mobile than the larger ones. Taking account of these factors and the design values for particle size recommended in Section 6.3.1, it is considered appropriate to determine the indicative values of self-cleansing velocity using an assumed particle size of $d = 1.5\times10^{-3}$ m.

Adopting the following design values:

$\mu = 0.45$
$s = 2.6$
$d = 1.5\times10^{-3}$ m
$\theta = 0^{o}$ or 37.5^{o}

it can be shown that Equations (B.1) to (B.3) can be simplified to the following alternative forms:

<u>Nearly horizontal or vertical pipes</u>

$$C_V = 9.42\times10^{-6}\left[1 - \frac{1.1\,V_T}{V}\right]^4 \frac{V^3}{D^{2.1}} \qquad \text{(B.8)}$$

where V and V_T are in m/s and D is in m, and V_T is calculated from Equation (B.3).

<u>Upward inclined pipes</u> ($0^{0} < \theta \leq 60^{o}$)

$$C_V = 8.81\times10^{-6}\left[1 - \frac{1.1\,V_{TI}}{V}\right]^4 \frac{V^3}{D^{2.1}} \qquad \text{(B.9)}$$

where the same units apply, and V_{TI} is calculated from Equations (B.2) and (B.3). These results incorporate a factor of safety of 10% in terms of velocity (see Section 6.2).

The indicative values of minimum self-cleansing velocity given in Tables B.1 and B.2 in Section 7.9 are derived from these equations for three values of volumetric sediment concentration as recommended in Section 6.3.3: low sediment load ($C_V = 3.8\times10^{-6}$); medium sediment load ($C_V = 19\times10^{-6}$); and high sediment load ($C_V = 77\times10^{-6}$). Values are not given for pipe diameters greater than 2.0 m because above this size the velocities may be high enough to transport the sediment in suspension and not in flume traction, as is implicitly assumed for application of Equations (B.1) to (B.3) and Equations (B.8) and (B.9).

7. DESIGN RECOMMENDATIONS

7.1 General

The guidelines on the hydraulic design of inverted siphons presented in this Chapter are based on the results of the HR experimental work (see Part A and Chapter 6) and on information from a review of available literature, supported by details of some typical schemes that have been constructed. A summary of key data from the literature review is given in Appendix 2. The Appendix has a similar structure to this Chapter in order to allow for ease of reference. Thus, as an example, Section 7.4 provides recommendations on the design of inlet chambers and is matched by Section A2.4 in Appendix 2 which gives the corresponding background information from the references.

7.2 Alternatives to inverted siphons

Inverted siphons are expensive to construct and can potentially be troublesome in operation; this is particularly the case for siphons carrying sewage. Alternatives should therefore be thoroughly considered before a decision is made to construct a new inverted siphon. Possible means of avoiding the use of an inverted siphon are:

- Diverting the flow to another sewer or drainage system.
- Taking the flow around the obstacle rather than underneath it.
- Taking the flow to another location where levels make it possible to construct the pipe or sewer at a continuous grade.
- Reconstructing the sewer or drain downstream of the obstacle at a lower level so as to eliminate the need for an upward-sloping leg.

If there is no practicable alternative to an inverted siphon, it will normally be better to design it to operate as a gravity system than as a pumped system (unless the siphon is part of a rising main). Siphons using pumps tend to require more maintenance, have continuing energy costs and are vulnerable to pump or power failures.

7.3 Number of pipe barrels

When designing an inverted siphon a key question is how many separate pipe barrels need to be provided. The factors determining the choice are:

- Nature of the flow – eg, river or irrigation water, storm water, foul sewage, combined sewage.
- Amount of settleable solids carried by the flow.
- Type of siphon – gravity or pumped.
- Maximum and minimum design rates of flow for the siphon.
- Frequency of occurrence of different flow rates.
- Minimum allowable size of pipe.
- Minimum allowable flow velocities in the pipes.
- Maximum allowable head loss across the siphon.

For siphons serving foul sewers, it is recommended that there should normally be two barrels, each sized to carry the maximum design rate of flow but with a side weir installed in the inlet chamber so that only one barrel is normally in operation (see Section 7.4). The size of a barrel should be such that the required self-cleansing velocity (see Section 7.9) is reached or exceeded at least once per day. Two barrels provide an emergency back-up and allow one barrel to be closed for routine maintenance or cleaning. More than two barrels will not usually be necessary because of the limited range of flows that typically occur in separate foul water sewers. If only one barrel is provided, consideration should be given to the provision of a flushing system (see Section 7.8).

For siphons serving storm water sewers or combined sewers, the greater range of flow rates may require the use of more than the minimum two barrels. The low-flow barrel in a combined system should be sized so that the required self-cleansing velocity (see Section 7.9) is reached or exceeded at least once per day. The maximum flow rate that can be carried by the low-flow barrel will be determined by the maximum allowable head loss across the siphon (see Section 7.10). The criteria for sizing other barrels are harder to define, but it might be reasonable to require the self-cleansing velocity in the second barrel to be achieved by wet-weather flows occurring, on average, once every two months. However, for each barrel, its maximum flow capacity will be dependent on the maximum allowable head loss across the siphon.

It is recommended that no pipe or conduit in an inverted siphon should be smaller than 200mm in diameter because of the danger of blockage and the difficulty of cleaning.

7.4 Inlet chamber

An inlet chamber is normally required for an inverted siphon receiving flow from a gravity drainage system. The chamber is located at the start of the downward-sloping leg (see Figure 1) and has the following purposes:

- To convey water smoothly into the siphon from the upstream sewer or open channel with minimum loss of head.
- To divide flows in the required ratios between different barrels of the siphon (where more than one pipe is used).
- To provide access for inspection and maintenance.
- To allow installation of penstocks or gate valves for closing individual barrels of the siphon for emptying and cleaning.

Efficient division of flow between the barrels of a siphon can be achieved by constructing side weirs on one or both sides of the low-flow channel within the inlet chamber. The lengths of the side weirs can be chosen so as to minimise the increase in water level in the chamber needed to bring successive barrels into operation. The gradient of the low-flow channel (and a suitable length of the incoming pipe or sewer) needs to be flat enough to ensure that flow conditions are sub-critical along the length of the side weir; if this is not the case, there is a danger that a hydraulic jump may occur in the low-flow channel and produce an unsatisfactory division of flow. The level of a side weir should be set as high as possible, consistent with the required flow split, so as to help ensure that any sediment moving along the invert of the channel always enters the low-flow barrel of the siphon. If the layout of the site results in the low-flow channel in the inlet chamber being curved in plan, it is recommended to locate a side weir on the outside of the bend because this will reduce the danger of bed-load material passing over the weir. Methods for predicting the flow capacity of side weirs are described by Chow (1959, Chapter 12); results of recent studies on discharge coefficients of side weirs are given by Borghei, Jalili & Ghodsian (1999).

The entrance to a siphon pipe should be designed so that it does not become submerged at high rates of flow. If surcharging does occur, a slow-moving zone of water will form above the entrance and cause an undesirable accumulation of scum and floating debris in the inlet chamber. Methods of preventing surcharging include:

- Setting appropriate overflow levels for side weirs supplying other barrels of the siphon.
- Using a well-rounded transition to the upper part of the entrance to each pipe.
- Forming a smooth downward step in the invert of the channel just before the entrance to the pipe.

The purpose of the step is to accelerate the flow smoothly into the pipe. The size and shape of the step should be determined so that, at the design rate of flow, the profile of the water surface is a reasonable match to the vertical profile of the inlet transition.

7.5 Sloping legs of siphon

For the downward leg of an inverted siphon, there is no hydraulic reason for choosing a particular gradient (sloping or vertical) since other parts of the siphon will be more critical in terms of sediment deposition. The selected gradient should be decided taking account of the layout of the site, structural and constructional factors, and maintenance requirements. Bends should be of a long enough radius to prevent flow separation and limit local head losses to acceptable values (see Section 7.10).

For the upward leg of a siphon, the highest self-cleansing velocities are required if the angle is between about 22.5^o and 45^o, ie upward gradients between 1 : 2.4 and 1 : 1 (vertical : horizontal). Vertical pipes are more efficient than sloping pipes at transporting sediment upwards, but it is necessary to use long-radius bends at top and bottom in order to prevent local deposition. The results of the HR experiments indicated that a bend radius of 5× the internal pipe diameter (measured to the centreline) would be sufficient to prevent deposition. If the angle of the upward leg needs to be more than 60^o, it is recommended to construct it as a vertical pipe.

Vertical or steeply-inclined pipes may not be compatible with certain methods of cleaning and maintenance. In order to remove heavy solids with jetting equipment, the upward gradient should not be steeper than about 1 : 5.

7.6 Horizontal leg of siphon

"Horizontal" legs of siphons should be given a small gradient (eg 1 : 200) in order to create a well-defined low point in the system. An access chamber and draw-off arrangements should be located at this point so that a closed barrel can be emptied for maintenance (see Section 7.8). The horizontal leg of a siphon should be of constant diameter in order to facilitate cleaning operations.

7.7 Outlet chamber

An outlet chamber has the following purposes:

- To convey water smoothly from the siphon into the downstream sewer or channel with minimum loss of head.
- To provide access for inspection and maintenance.
- To allow installation of penstocks or gate valves for closing individual barrels of the siphon for emptying and cleaning.

With multi-barrel siphons it is important to prevent back-flow within the outlet chamber because this could cause sediment from an operating barrel to be deposited in the non-operating barrels. Alternative methods of preventing this are to:

- Use side walls to constrain the flow from each barrel so as to limit the size and position of any recirculation zones in the chamber.
- Set the invert levels of the other barrels of the siphon above the water level that occurs when the low-flow barrel is operating at its maximum capacity.
- Arrange for the other barrels to discharge over weirs that separate them from the low-flow channel.

7.8 Desilting and cleaning arrangements

Penstocks or gate valves should be installed at the inlet and outlet chambers to allow individual barrels of siphons to be closed for cleaning or repair.

An access chamber should be located at the low point of the system so that each barrel can be emptied individually. In the case of sewer siphons, it is not desirable to construct the access chamber as an open manhole because it will remain full of standing sewage and accumulate floating debris. Also, the constant and possibly large hydrostatic pressure may cause seepage of sewage through the walls and base of the manhole. The preferred option is to carry the siphon pipes straight through the access chamber and to provide them with removable hatch-box covers or tee-pieces with valves. This arrangement allows the access chamber to remain dry during normal operation of the siphon, and also makes it possible to use a portable pump to remove water from a closed barrel and discharge it into another barrel that is still in use.

Installation of a permanent flushing system should be considered for single-barrel siphons and for situations where naturally-occurring flow rates do not produce self-cleansing conditions sufficiently frequently. Possible means of producing flushing flows include:

- Suitable operation of an upstream pumping station. Combined operation of duty and stand-by pumps can produce a short-term increase in flow rate through a siphon. If the pumping station is located too far upstream, storage in the pipes or channel may attenuate the surge flow produced by the pumps and result in an insufficient increase in flow rate through the siphon.
- Temporary holding back of water in the upstream sewer or channel by means of a penstock. Rapid opening of the penstock, combined with the increased upstream head, will increase the flow velocity through the siphon for a limited time.
- Diversion of flow from the upstream sewer or channel into a holding tank. A cistern-type siphon can be used to discharge a flushing flow when the water level in the tank reaches a required level. If only a limited amount of backing-up can be permitted in the upstream sewer or channel, the tank may need to be relatively shallow and have a large surface area in order to provide the necessary volume of storage. This may result in sediment depositing in the tank, leading to maintenance problems and increasing the risk of the cistern device becoming blocked. An overflow weir should be provided so that flow can bypass the holding tank.

In order to be effective, the volume of a flushing flow should be at least equal to the volume of water contained in the pipes of the inverted siphon.

If it should be necessary to remove blockages or deposited sediment from an inverted siphon, possible methods include:

- Rodding – this is suitable only for smaller diameter pipes and requiring access through chambers or hatch-boxes at approximately 100m intervals.
- Jetting – in order to be able to remove heavy solids, the gradients of the upward-sloping and/or downward-sloping legs of the siphon may need to be limited to a maximum of about 1 : 5 (see Section 7.5).
- Use of sewer balls – a flexible ball (eg formed from interleaved steel bands) is inserted at the upstream end of the system; the flow causes an increase in pressure on the upstream side of the ball and this moves it along the pipe, with the deposited sediment being pushed in front of it; water jetting between the ball and the walls of the pipe also helps to erode the deposits. For the method to be successful, the sewerage system must be able to produce a sufficient head difference to keep the ball moving without causing flooding problems further upstream. The pipes in the siphon should be of a constant diameter throughout and should not have any sharp bends. Also, the inlet and outlet chambers and access chambers need to be designed so that the sewer ball can be easily inserted and removed.

Maintenance requirements should be considered carefully when determining the overall layout of a siphon. The point to which any deposited sediment will be moved by cleaning operations should be provided with good access and designed so that the collection and removal of the material can be carried out as easily as possible.

7.9 Minimum velocities in pipes

The most important performance requirement for an inverted siphon is that it should be able to operate continuously over long periods without the pipes becoming blocked by sediment. If sediment deposits form in the pipes, they can increase the overall head loss through the system considerably. In the case of a single barrel siphon, this can lead to backing up of the flow in the upstream lengths of sewer or channel, resulting in surcharging and possible surface flooding. In the case of a multi-barrel siphon, if sediment is deposited in the primary pipe (which operates continuously carrying the smaller flows and most of the sediment), it will cause more water than intended to be diverted to the second pipe. As a result, the flow rate through the primary pipe will decrease, leading to increased deposition that may continue until the pipe is completely blocked by sediment.

The minimum flow velocity required to transport sediment through an inverted siphon without deposition depends on:

- the diameter of the pipe;
- the angle of inclination of the pipe;
- the mean size of the coarser, inorganic sediments ($d_{50} \geq 0.6$ mm) that are most likely to deposit;
- the volumetric concentration of this size fraction entering the siphon.

Values of minimum velocity for a particular siphon can be calculated from the design equations and the information given in Chapter 6 (which were obtained from the experimental study described in Part A of this Report).

For the particular case of inverted siphons in gravity sewers, it is convenient to define a set of "indicative" values of minimum velocity and discharge which can be used as a guide for initial design or for checking the performance of an existing siphon. Recommended indicative values are given in the following Tables B.1 and B.2, and their derivation is explained in Section 6.5. The values are likely to err on the conservative side and they should not necessarily be considered as "standard" values. For detailed design, account should be taken of any relevant information about the characteristics of the sewerage system; more specific values of minimum velocity may then be calculated using the design equations in Chapter 6.

The following points should be noted when using Tables B.1 and B.2:

- The minimum velocities and discharges are given for three categories of sediment loading (low, medium and high) that are considered to be typical of conditions in UK gravity sewers (see Section 6.3.3 for details). The "low"category will normally not be appropriate for the primary barrels of siphons in combined sewers or separate surface water sewers unless an effective sediment removal system is located just upstream of the siphon.

- In the case of a multi-barrel siphon, it may be appropriate to assume that the sediment loading for the second and subsequent barrels will be one category lower than for the primary barrel, provided that the flow-splitting arrangement in the inlet chamber is effective in keeping bed-load material within the low-flow channel (see Section 7.3).

- The factor of safety provided by the indicative values varies depending on the angle of inclination of the pipe but is a minimum of 10% relative to the results of the experimental study and the assumed data values (see Section 6.5 for details).

- The values of minimum velocity and discharge for upward vertical pipes are only valid if suitable long-radius bends are used at either end of the section of vertical pipe. The results of the HR experiments indicated that a bend radius of 5× the internal pipe diameter (measured to the centreline) would be sufficient to prevent deposition. If bends of too tight a radius are used, higher velocities will be necessary to prevent sediment depositing at the bottom of the vertical pipe. A long-radius bend is also needed at the top of the vertical pipe to enable the suspended sediment to be discharged smoothly into the sewer downstream of the siphon.

- Values of minimum velocity and discharge for pipe diameters other than those given in Tables B.1 and B.2 can be determined by interpolation or from Equations (B.8) and (B.9). It is not recommended to use these equations for pipe diameters greater than 2.0m because the flow conditions are likely to be different in type from those to which the equations apply (see Section 6.5).

- Guidance on the frequency with which self-cleansing conditions should be achieved is given in Section 7.3.

The minimum velocities given in Tables B.1 and B.2 apply to siphon pipes of circular cross-section. Suitable design equations are not currently available for predicting self-cleansing conditions in upward-sloping conduits of square or rectangular cross-section. In the absence of such information, an approximate estimate of the minimum velocity may be obtained by treating the conduit as equivalent to a circular pipe of equal cross-sectional area, ie by using in Tables B.1 and B.2 an equivalent pipe diameter $D_E = \sqrt{(4A/\pi)}$, where A is the flow area of the square or rectangular conduit.

7.10 Head losses

When designing an inverted siphon to operate under gravity, a key factor is the maximum water level that can be allowed in the inlet chamber at the maximum design rate of flow. The difference between this level and the corresponding level in the outlet chamber (which will usually be determined by the characteristics of the pipe or channel system downstream) defines the maximum disposable head under which the siphon must operate. Part of the disposable head may be lost as a result of changes in water level across weirs that are used to divide the flow between individual barrels of the siphon (see Section 7.4); similar types of loss may also occur if weirs are used in the outlet chamber to prevent back-flow (see Section 7.7). The remaining disposable head is available to overcome :

- the flow resistance of the siphon pipes
- the head losses that occur at the inlet and outlet of the siphon
- the point head losses that occur at pipe fittings such as bends.

Information on how to calculate these head losses is given in Section 6.3.5.

The relationship between the minimum flow rate required for self-cleansing (see Section 7.9) and the maximum flow rate permitted by the available head determines the number and size of siphon barrels needed to cater for a particular range of discharges. An illustration of the design procedure is given by the worked example in Section 6.4.

Table B.1 Indicative values for self-cleansing conditions in horizontal pipes, downward-sloping pipes and upward vertical pipes of inverted sewer siphons

Sediment loading	Internal pipe diameter (m)	Minimum self-cleansing flow conditions	
		Velocity (m/s)	Discharge (m^3/s)
Low	0.15	0.42	0.0074
	0.225	0.53	0.021
	0.30	0.63	0.045
	0.375	0.72	0.080
	0.50	0.86	0.168
	0.75	1.09	0.483
	1.00	1.30	1.02
	1.25	1.49	1.82
	1.50	1.66	2.94
	1.75	1.83	4.39
	2.00	1.98	6.22
Medium	0.15	0.57	0.010
	0.225	0.72	0.029
	0.30	0.87	0.061
	0.375	0.99	0.110
	0.50	1.19	0.233
	0.75	1.53	0.676
	1.00	1.83	1.44
	1.25	2.11	2.59
	1.50	2.37	4.19
	1.75	2.62	6.29
	2.00	2.85	8.95
High	0.15	0.77	0.014
	0.225	0.99	0.040
	0.30	1.19	0.084
	0.375	1.38	0.152
	0.50	1.66	0.325
	0.75	2.15	0.951
	1.00	2.59	2.04
	1.25	3.00	3.68
	1.50	3.38	5.97
	1.75	3.74	8.99
	2.00	4.08	12.8

Table B.2 Indicative values for self-cleansing conditions in upward-sloping pipes of inverted sewer siphons (For pipe slopes $\leq$ 60^{o})

Sediment loading	Internal pipe diameter (m)	Minimum self-cleansing flow conditions	
		Velocity (m/s)	Discharge (m^3/s)
Low	0.15	0.52	0.0925
	0.225	0.66	0.026
	0.30	0.78	0.055
	0.375	0.88	0.098
	0.50	1.04	0.205
	0.75	1.32	0.584
	1.00	1.56	1.23
	1.25	1.78	2.19
	1.50	1.98	3.51
	1.75	2.17	5.23
	2.00	2.35	7.40
Medium	0.15	0.68	0.013
	0.225	0.86	0.034
	0.30	1.02	0.072
	0.375	1.17	0.129
	0.50	1.39	0.273
	0.75	1.78	0.785
	1.00	2.12	1.66
	1.25	2.43	2.98
	1.50	2.72	4.81
	1.75	2.99	7.20
	2.00	3.25	10.2
High	0.15	0.89	0.016
	0.225	1.14	0.045
	0.30	1.36	0.096
	0.375	1.56	0.173
	0.50	1.87	0.368
	0.75	2.42	1.07
	1.00	2.90	2.28
	1.25	3.34	4.10
	1.50	3.76	6.64
	1.75	4.15	9.97
	2.00	4.52	14.2

Note : The margin of safety provided by the indicative values varies with the angle of inclination of the siphon pipe (see Section 7.9)

7.11 Air movement induced by flow

Water flowing in part-filled pipes creates a corresponding movement of air in the space above the water surface. The presence of an inverted siphon will interrupt the flow of air which will therefore seek to escape from the system, either at the inlet chamber or, if that chamber has air-tight covers, at manholes farther upstream. Similarly, the flow of water in the downstream pipes will create a new air demand and tend to suck in air from the atmosphere at the outlet chamber.

There will usually be no difficulty in providing vented covers or pipes to admit air at the downstream end of a siphon. However, in the case of foul or combined gravity sewers, allowing air to escape at the upstream end of a siphon may be unacceptable because of the unpleasant smell and possible health risks. Options for dealing with the problem are to:

- Design the inlet chamber so that the water can entrain air as it enters the siphon barrel and carry it through the system to the downstream side.
- Use an activated charcoal filter (or equivalent) to "scrub" the air vented from the inlet chamber.
- Use a separate pipe to carry the air from the upstream side of the siphon to the downstream side.

The first option is not feasible unless the water velocity in the downward-sloping leg of a siphon is sufficient to prevent air bubbles and air pockets rising upwards and travelling against the flow back into the inlet chamber. Analysis of information given by Falvey (1980) suggests that the water velocity (V, in m/s) needed to prevent back-flow of air in a downward-sloping is related to the internal diameter of the pipe (D, in m) and its angle of inclination (θ) by the equation:

$$V = 8.1\ D^{0.5} \left[\sin\left(-\theta\right)\right]^{0.71} \qquad \text{(B.10)}$$

The negative sign for θ is necessary because in this Report the angle θ is defined as being positive upwards. Comparison of minimum velocities given by Equation (B.10) with the self-cleansing velocities in Tables B.1 and B.2 suggests that siphons designed for the "heavy" sediment loading may be able to transport air through the system provided that the angle of the downward-sloping leg of the siphon is not greater than about 10^o (or about 1 vertical : 5 horizontal). However, use of a physical model may be necessary to help define a suitable geometry for the inlet to the siphon pipe.

The third option of using a separate pipe to carry the air is a more common solution. The diameter of the pipe (termed an "air line" or "air jumper") needs to be sized in relation to the maximum rate of air flow produced by the water in the upstream sewer. An empirical method of sizing is to make the cross-sectional area of the air line equal to twice the cross-sectional area above the water surface in the upstream sewer under design conditions. The air line may be laid parallel to the siphon barrel or it may be attached to a suitable structure spanning the crossing (eg an overbridge in the case of a siphon built beneath a road in an underpass). In all cases the air line needs to be provided with a drain valve to allow removal of any moisture that condenses in the pipe.

7.12 Sulphide generation

If the time of travel of sewage through an inverted siphon is more than about ten minutes during periods of low flow, the lack of contact with air may lead to the generation of hydrogen sulphide gas (H_2S). Bubbles of the gas may escape into air pockets within the siphon where it could potentially cause serious corrosion of the pipe walls. Additionally, H_2S may produce odour and safety problems in the sewer downstream of the siphon.

7.13 Structural and materials aspects

Inverted siphons that are constructed on or under river beds should have sufficient weight to prevent flotation. If the pipes are located on or just below the bed, they should also be protected from erosion and possible movement by the flow.

Types of pipe that are typically used for inverted siphons include cast or ductile iron, steel and concrete. Further details of the properties of different materials and types of pipe are given in Section A2.13 of Appendix A2. Siphon barrels can also be rectangular in cross-section, formed for example using precast-concrete culvert units. If siphon barrels need to be laid at a considerable depth, an alternative option is to construct them by tunnelling; if bolted segments are used to form the walls of the tunnel, an additional lining may be needed to produce a smooth internal surface.

7.14 Air-cushion siphons

In this type of siphon, air is introduced to the horizontal barrel of the siphon under sufficient pressure to overcome the hydrostatic pressure applied by the hydraulic grade line between the inlet and outlet chambers. The air is retained by swan-neck bends at either end of the horizontal pipe and causes the siphon barrel to flow part-full of water; variation of the air pressure enables the cross-sectional area of the water to be adjusted to produce self-cleansing velocities at all rates of flow. The hydraulic design of air-cushion siphons is outside the scope of this Report but some further details are given in Section A2.14 of Appendix A2.

7.15 Examples of siphons

Details of examples of inverted siphons that are in operational use are given in Section A2.15 of Appendix A2.

8. ACKNOWLEDGEMENTS

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Figures

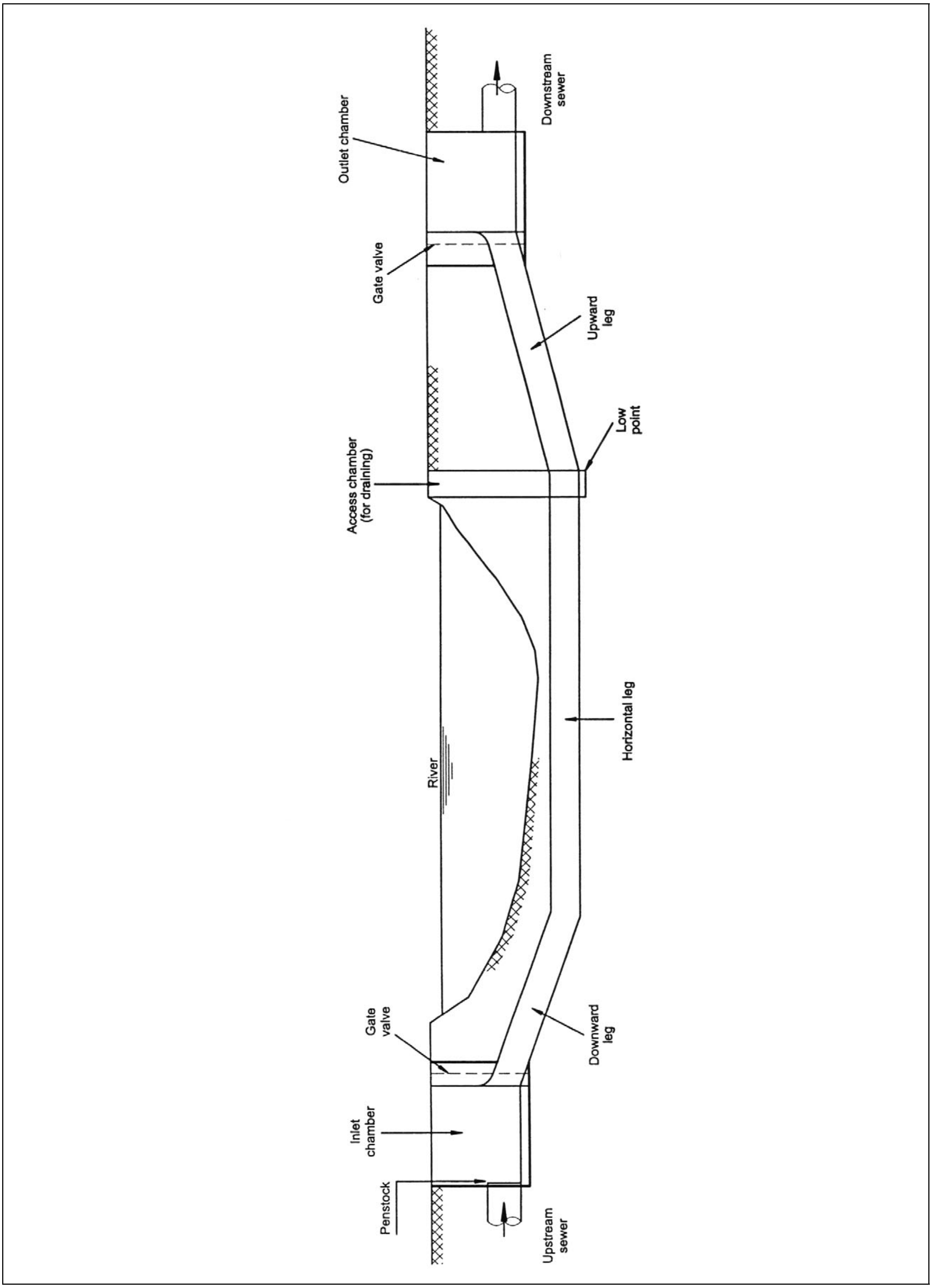


Figure 1 Layout of typical inverted siphon

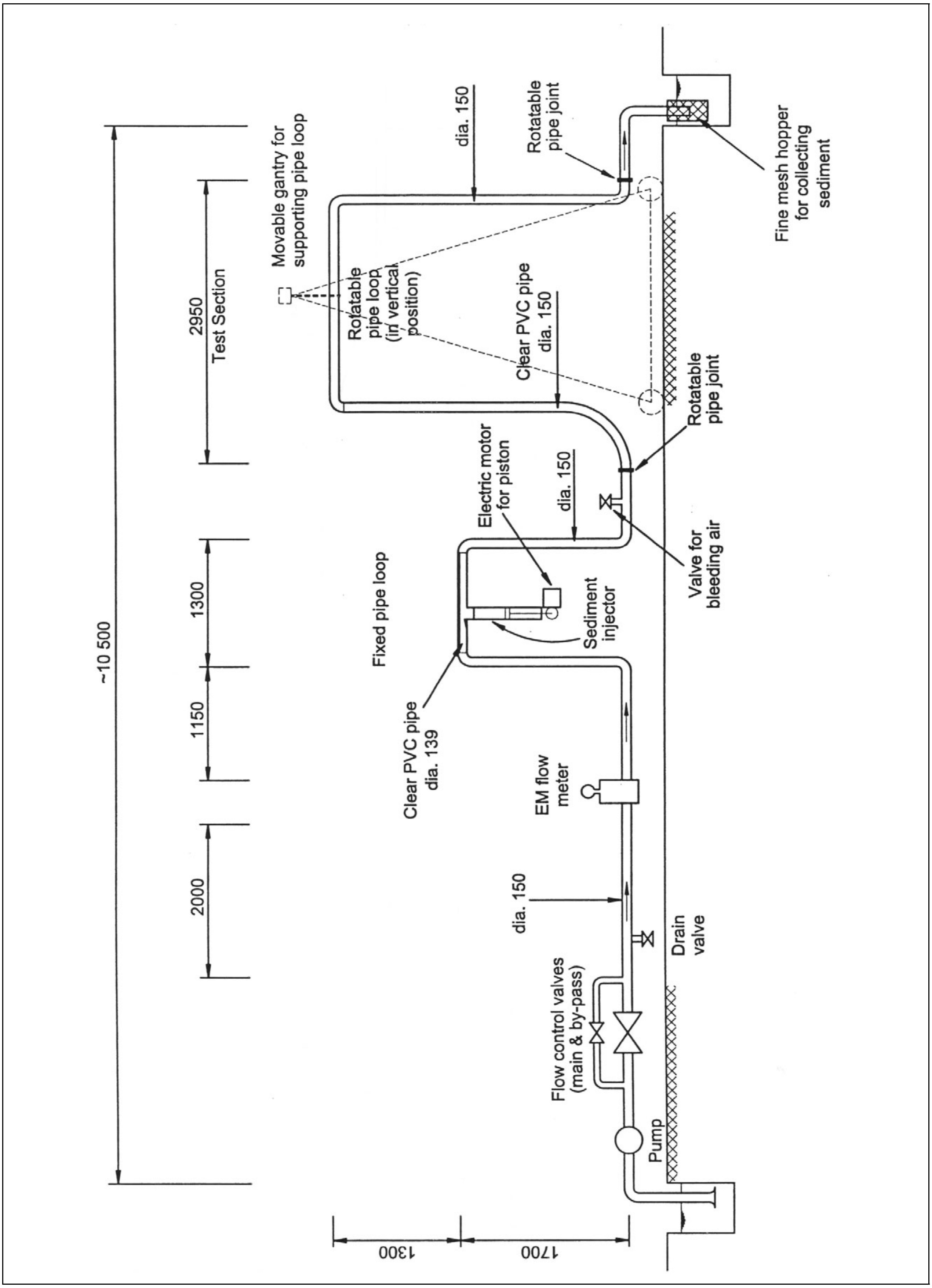


Figure 2 Diagrammatic layout of HR test rig

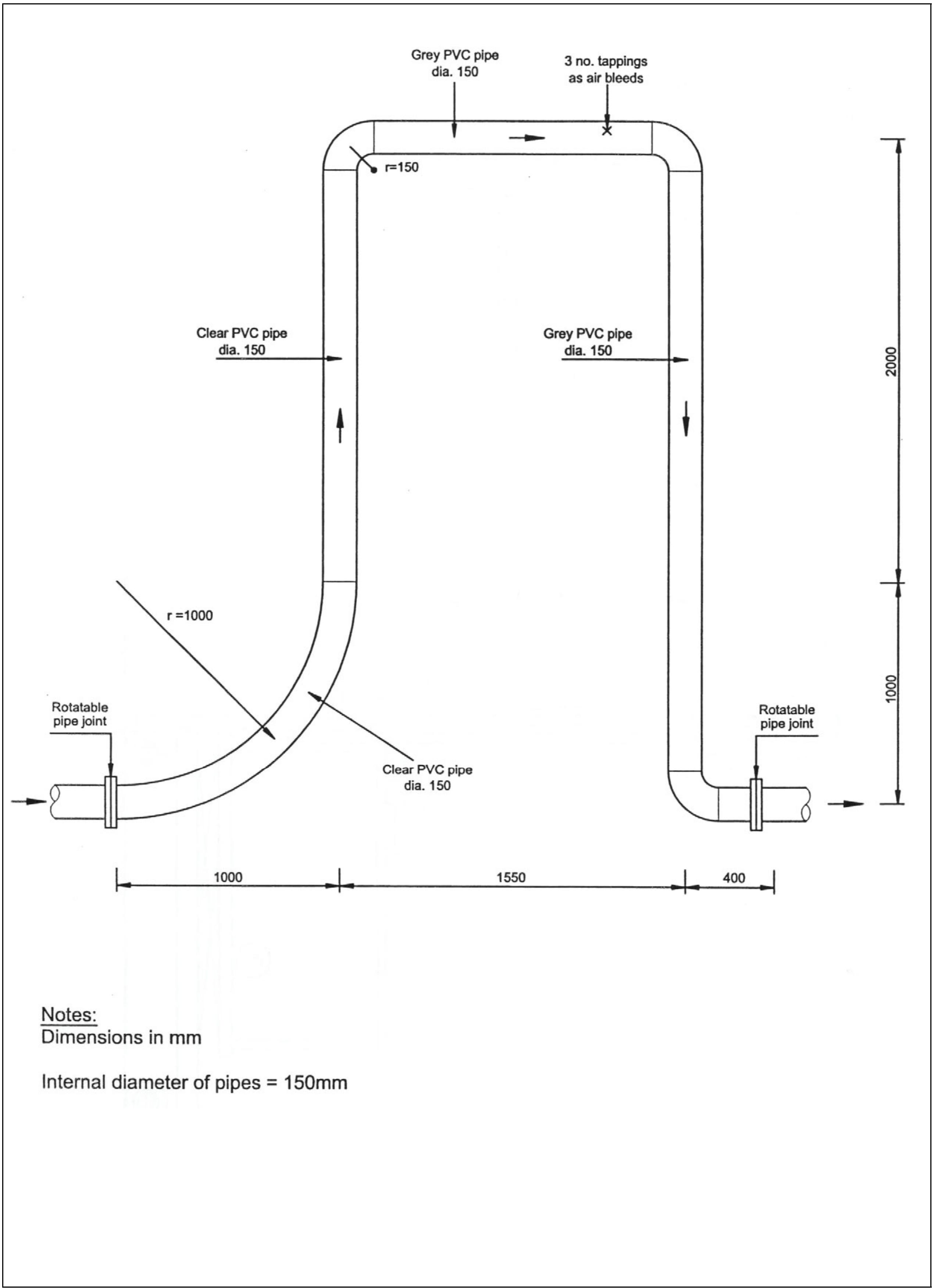


Figure 3a   Geometry of rotatable pipe loop

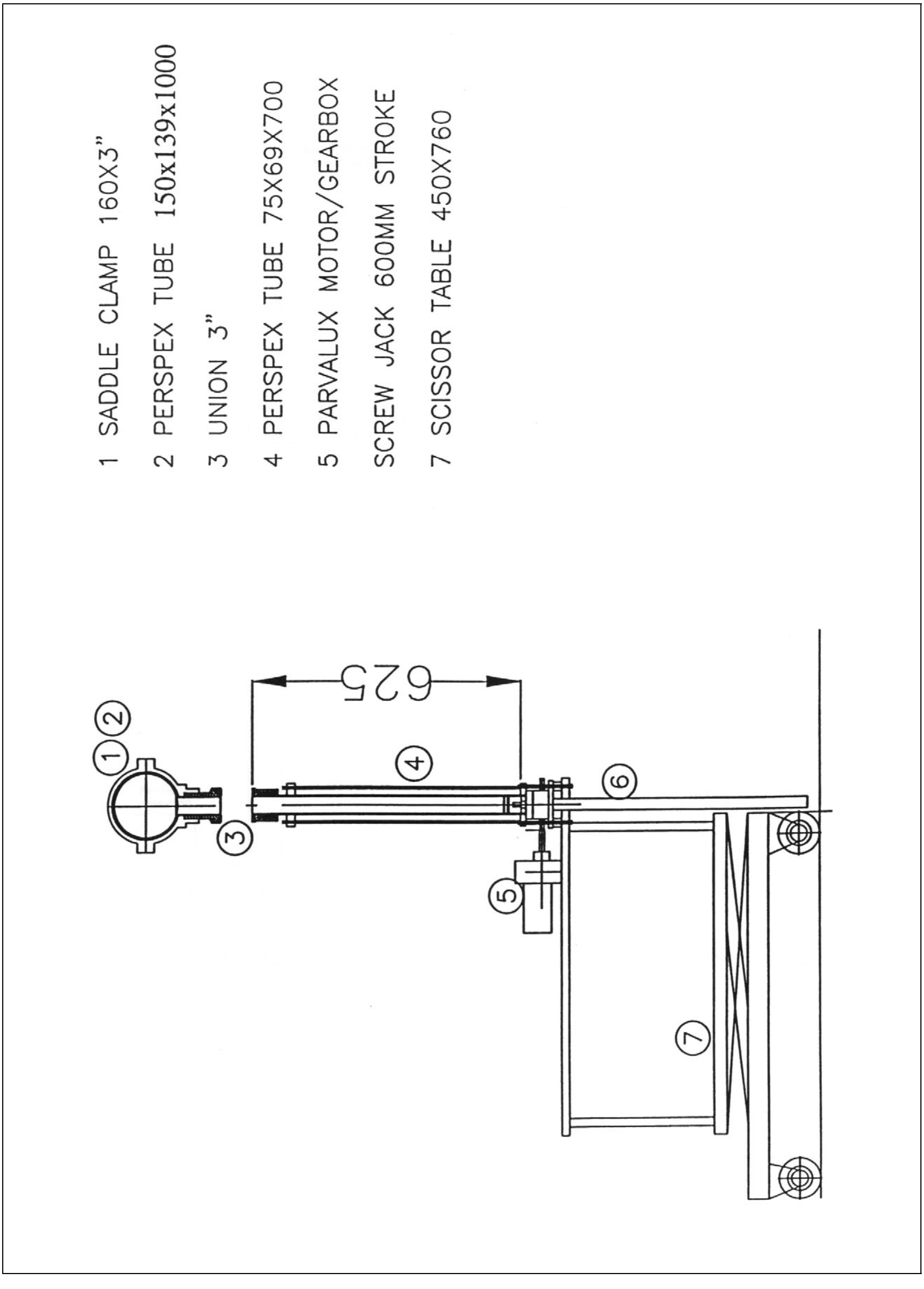
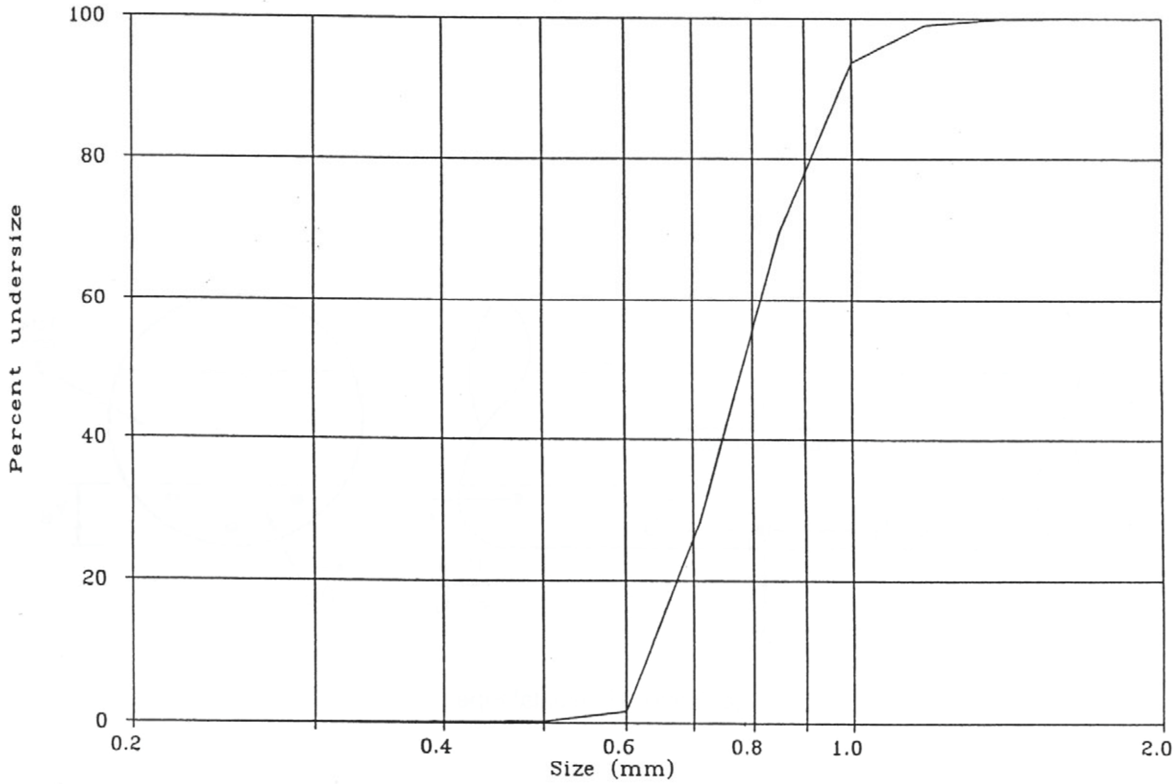
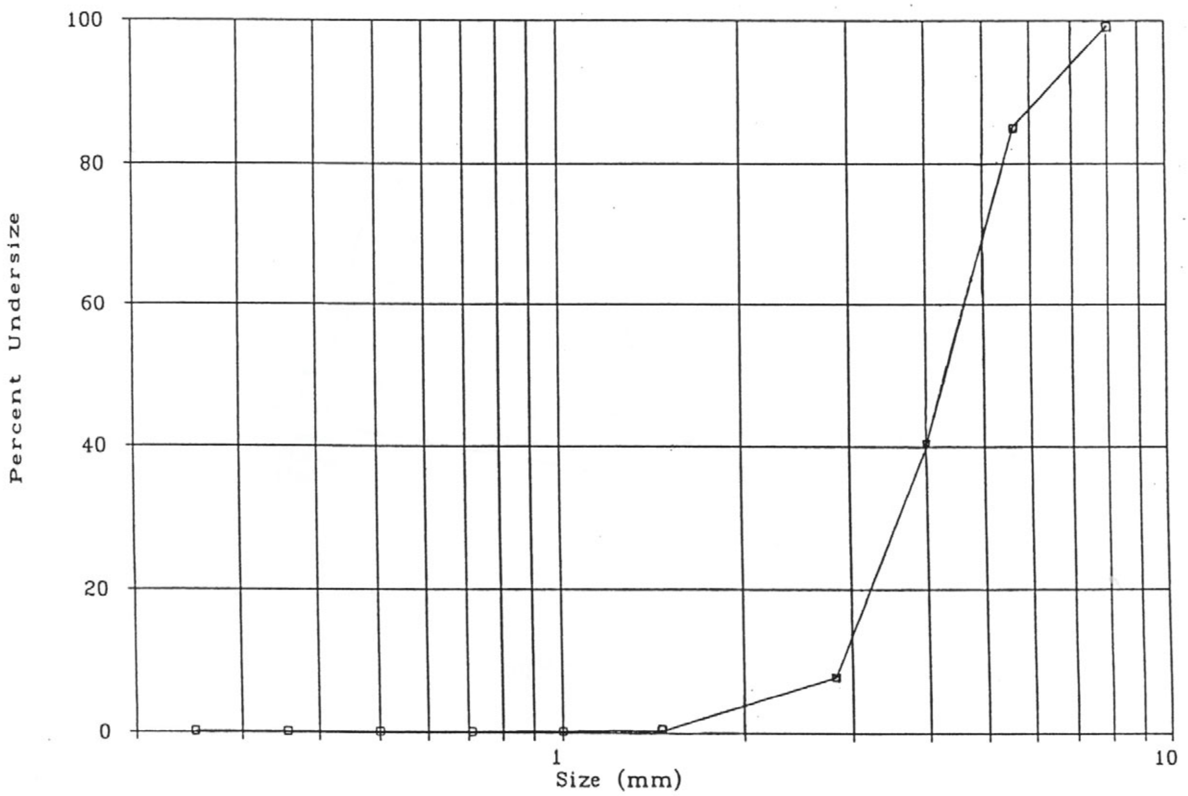


Figure 3b Layout of sediment injection system



a. coarse sand



b. fine gravel

Figure 4 Grading curves of sediments used in tests

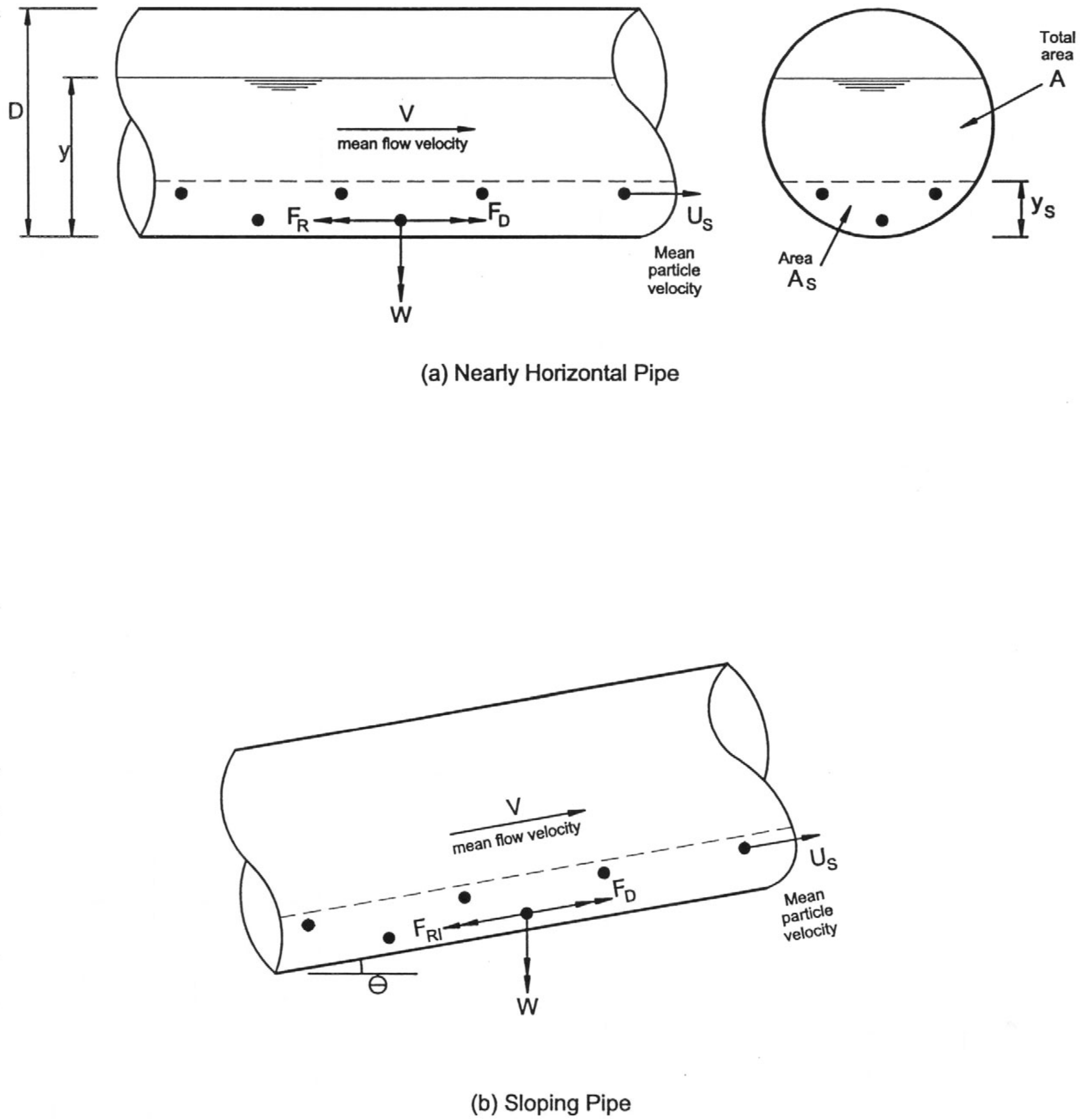


Figure 5 Definition diagram for sediment movement in flume traction

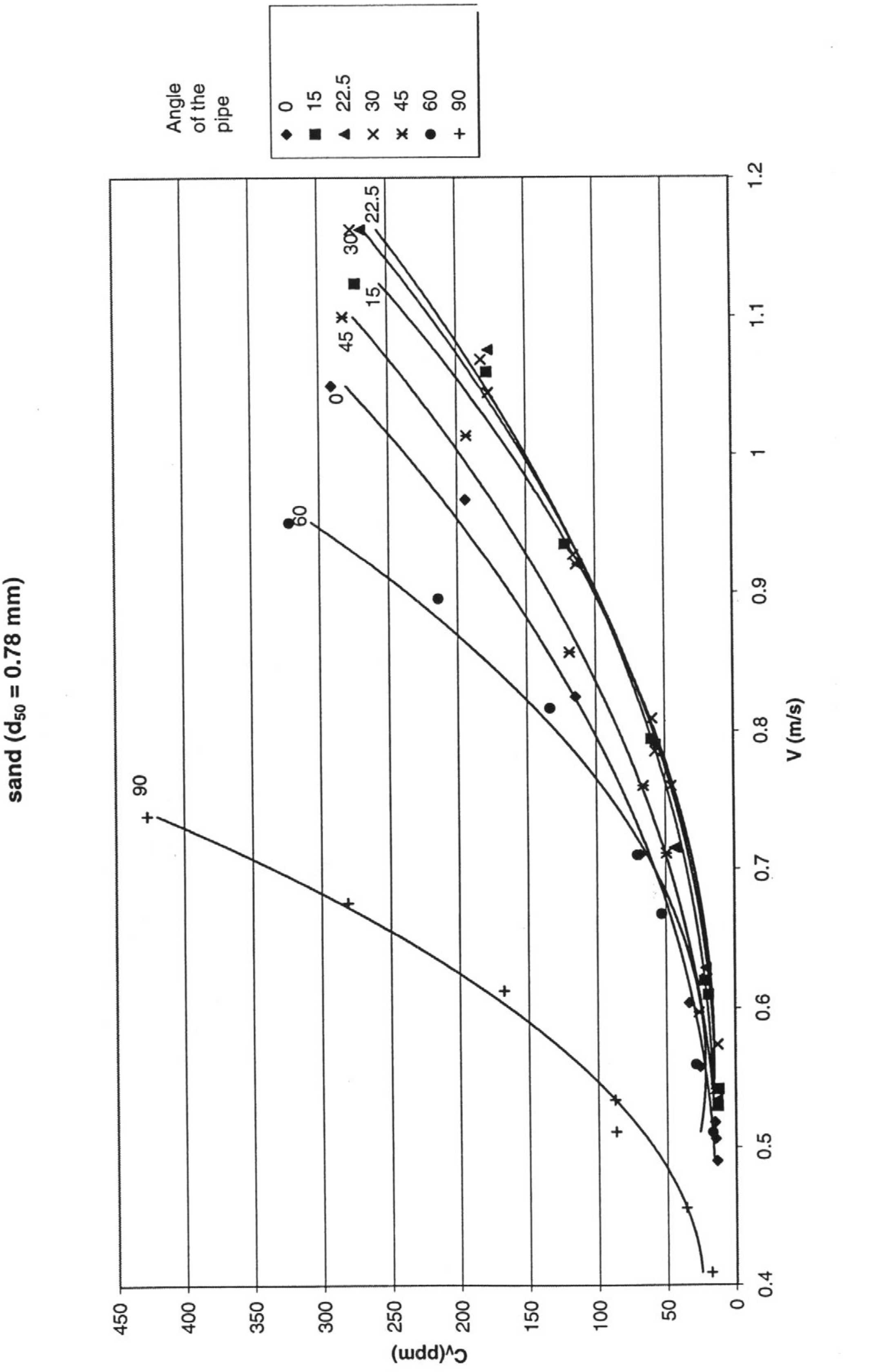


Figure 6 Variation of limiting flow velocity with sediment concentration and pipe slope for tests with sand

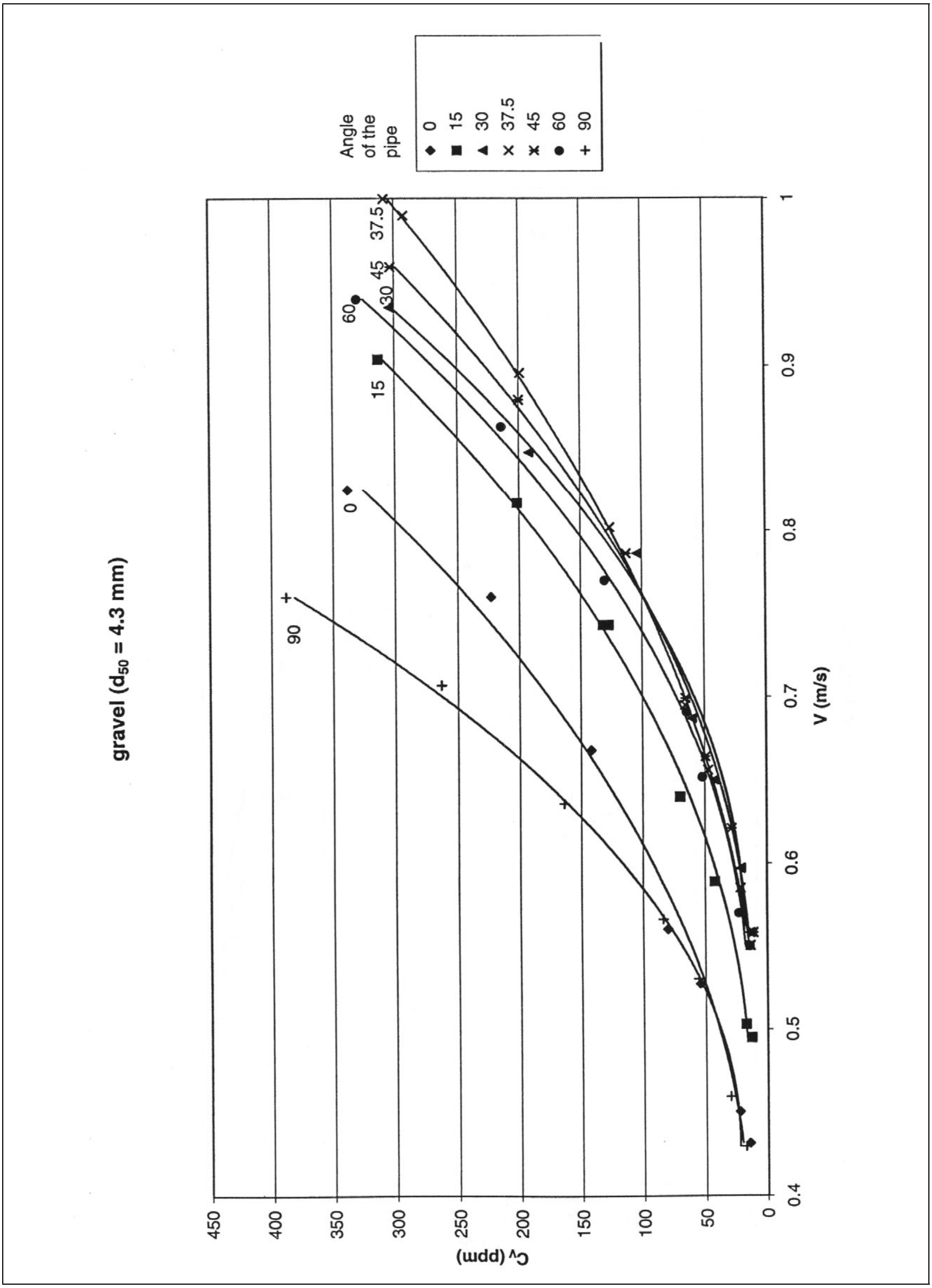


Figure 7 **Variation of limiting flow velocity with sediment concentration and pipe slope for tests with gravel**

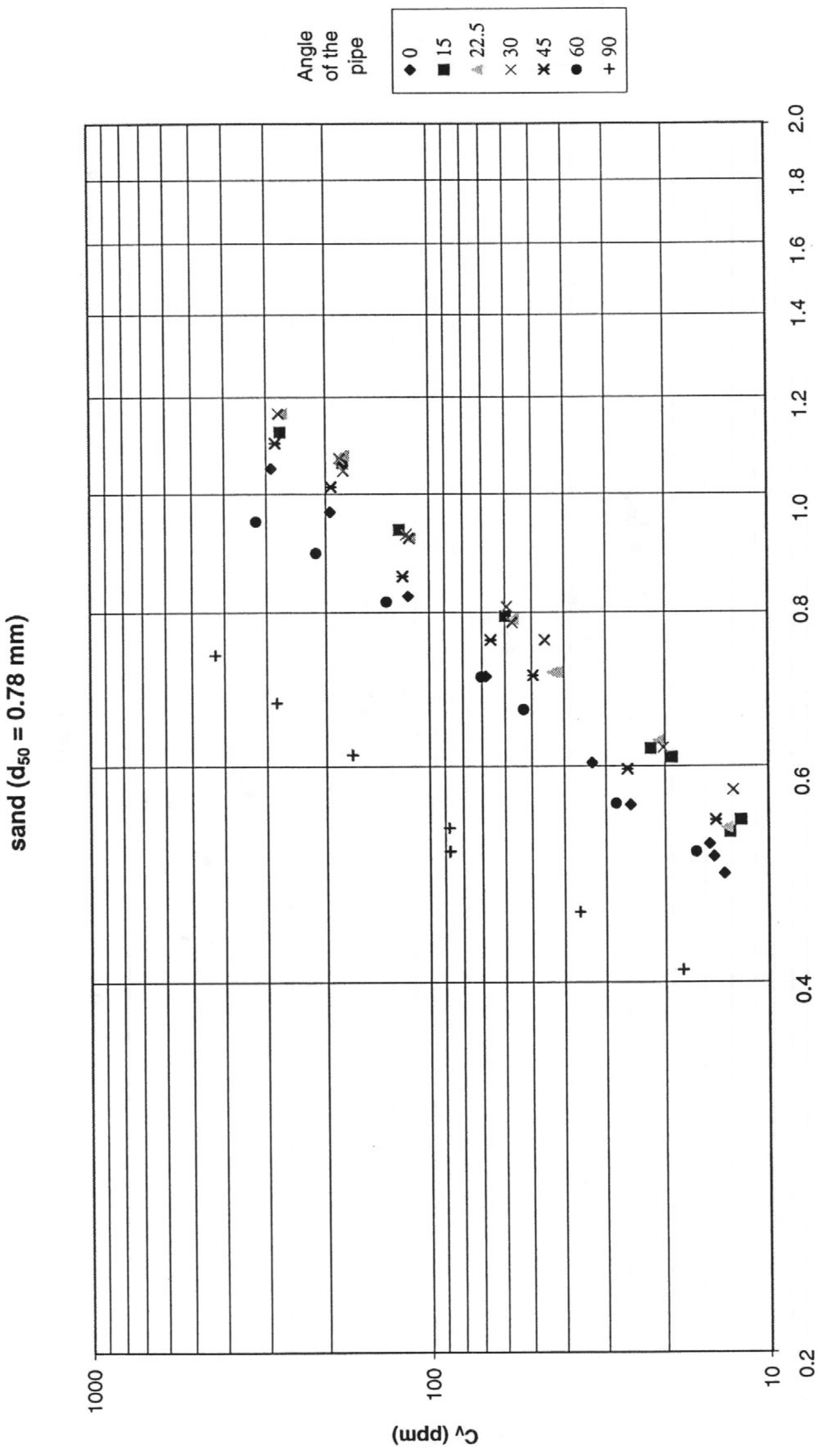


Figure 8 Variation of limiting flow velocity with sediment concentration and pipe slope for tests with sand (log-log format)

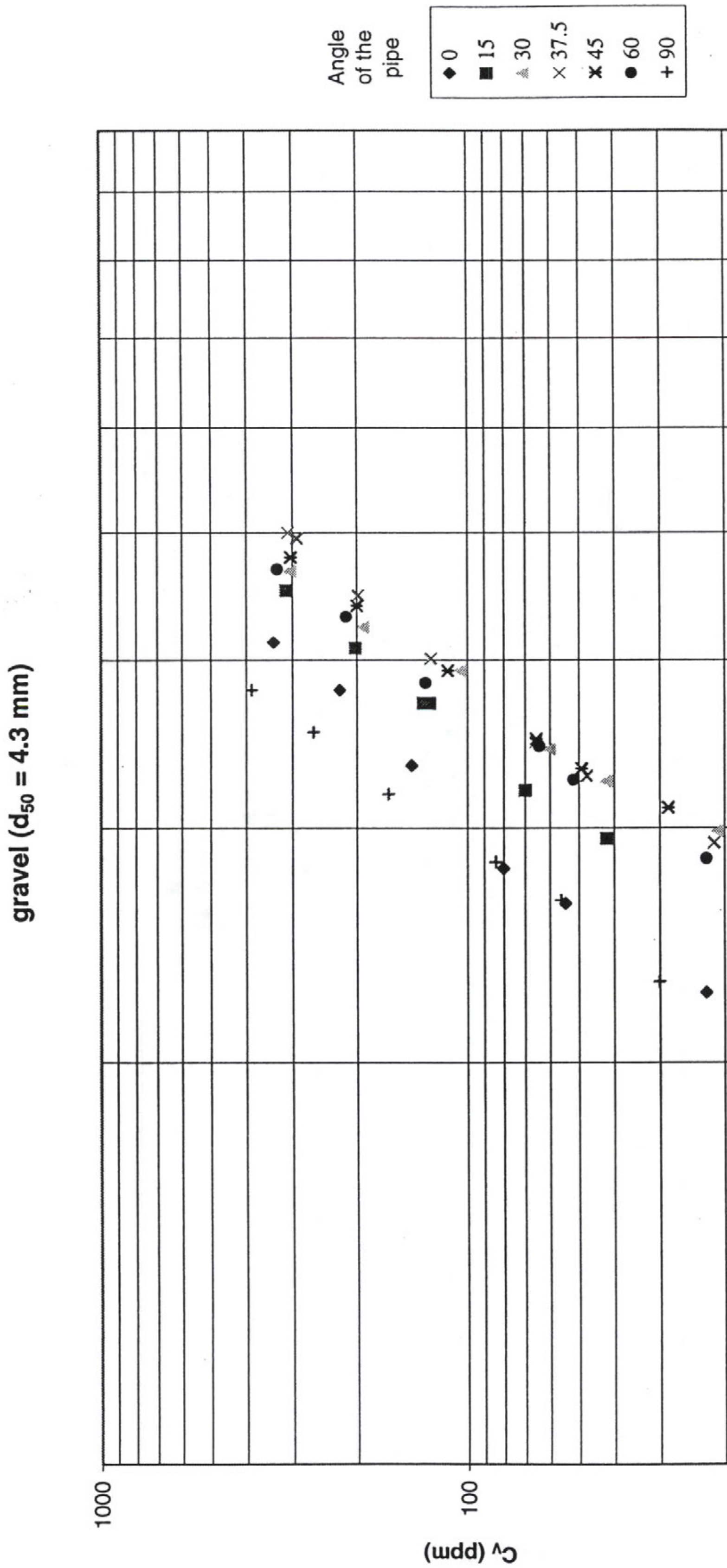


Figure 9 Variation of limiting flow velocity with sediment concentration and pipe slope for tests with gravel (log-log format)

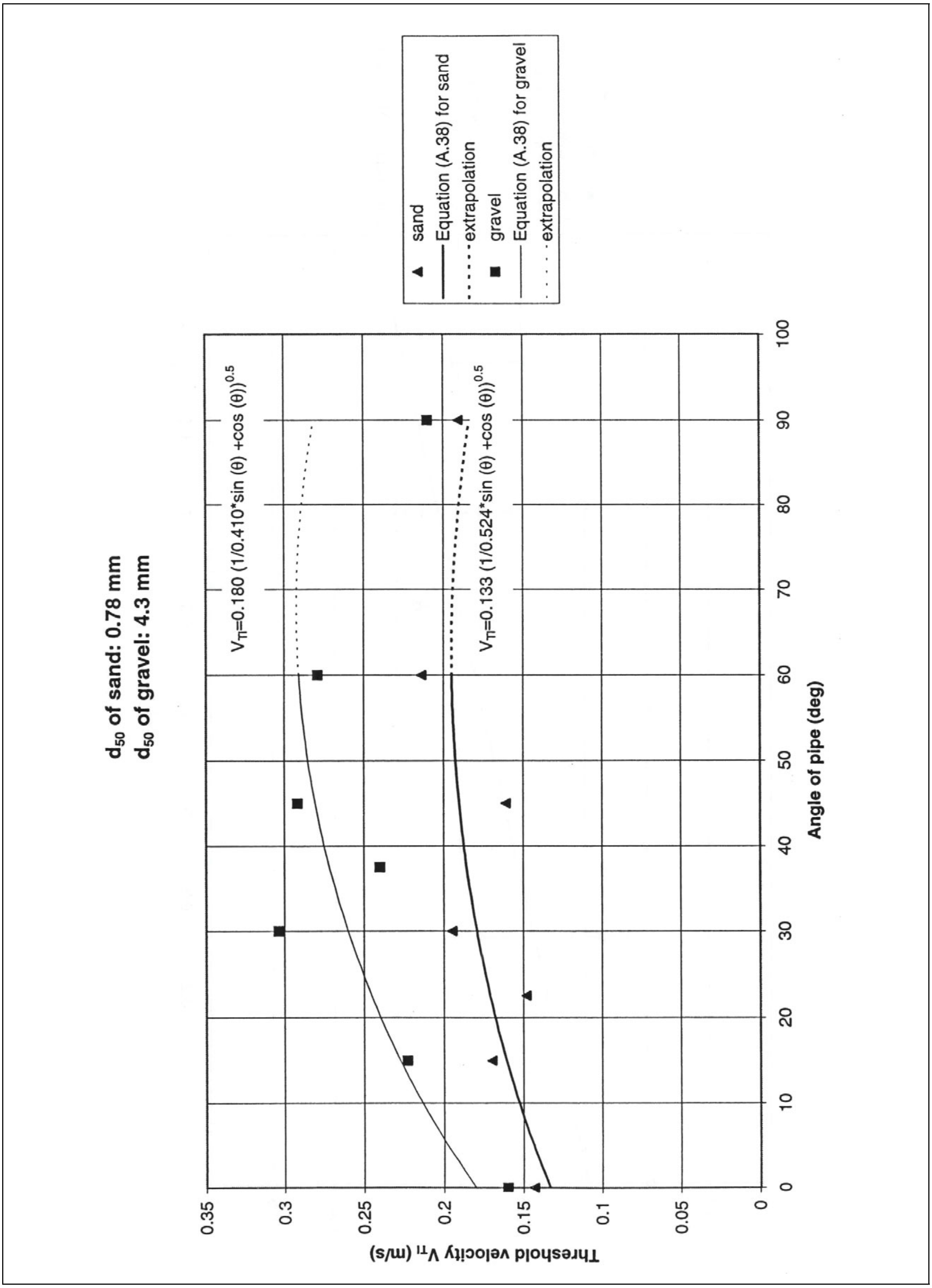


Figure 10 Variation of threshold velocity with pipe slope

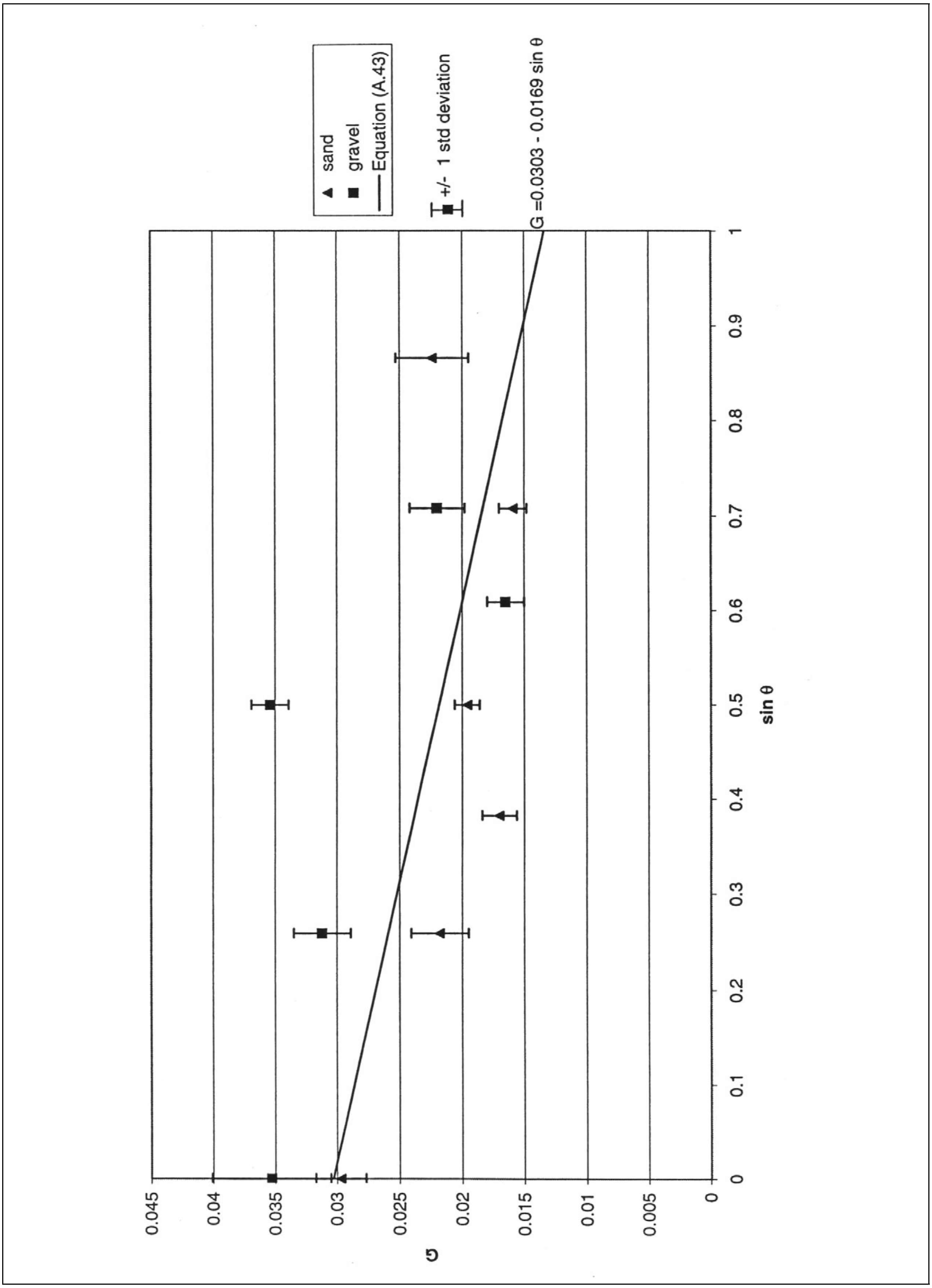


Figure 11 Variation of coefficient G with pipe gradient

Plates

Plate 1 Sediment moving in flume traction well above limit of deposition

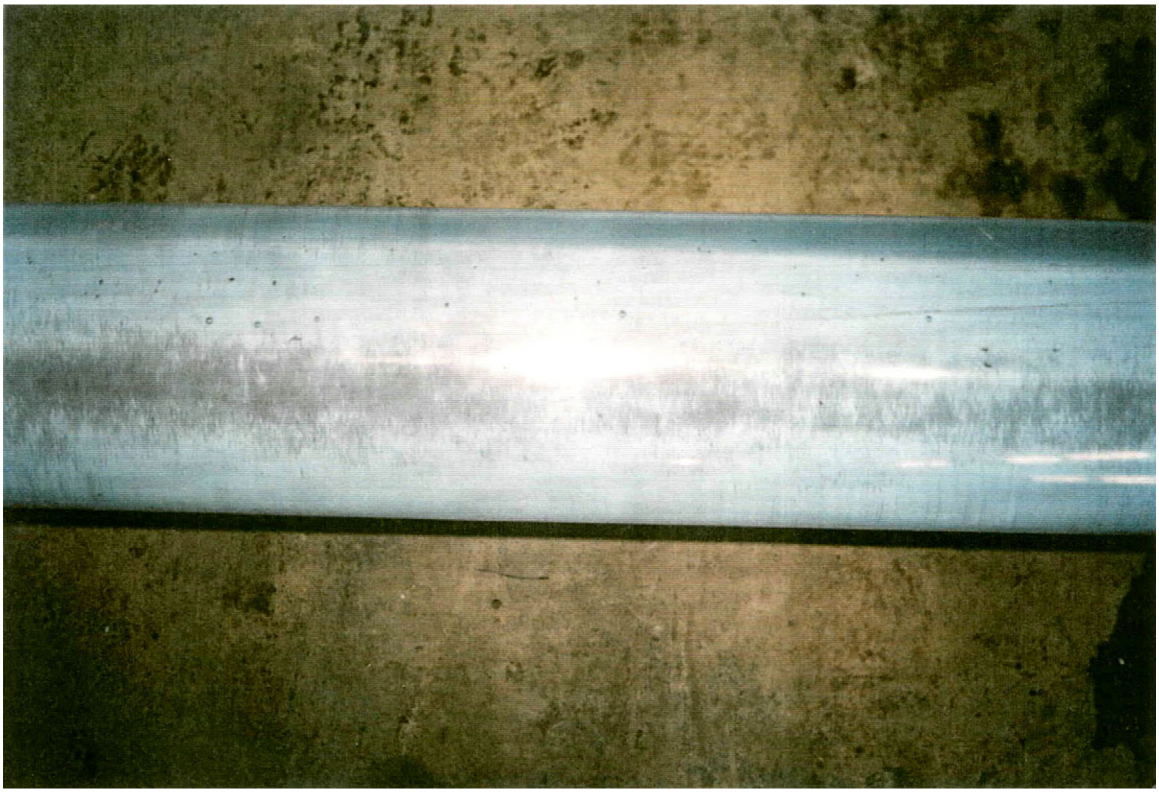


Plate 2 Sediment moving in flume traction just above limit of deposition

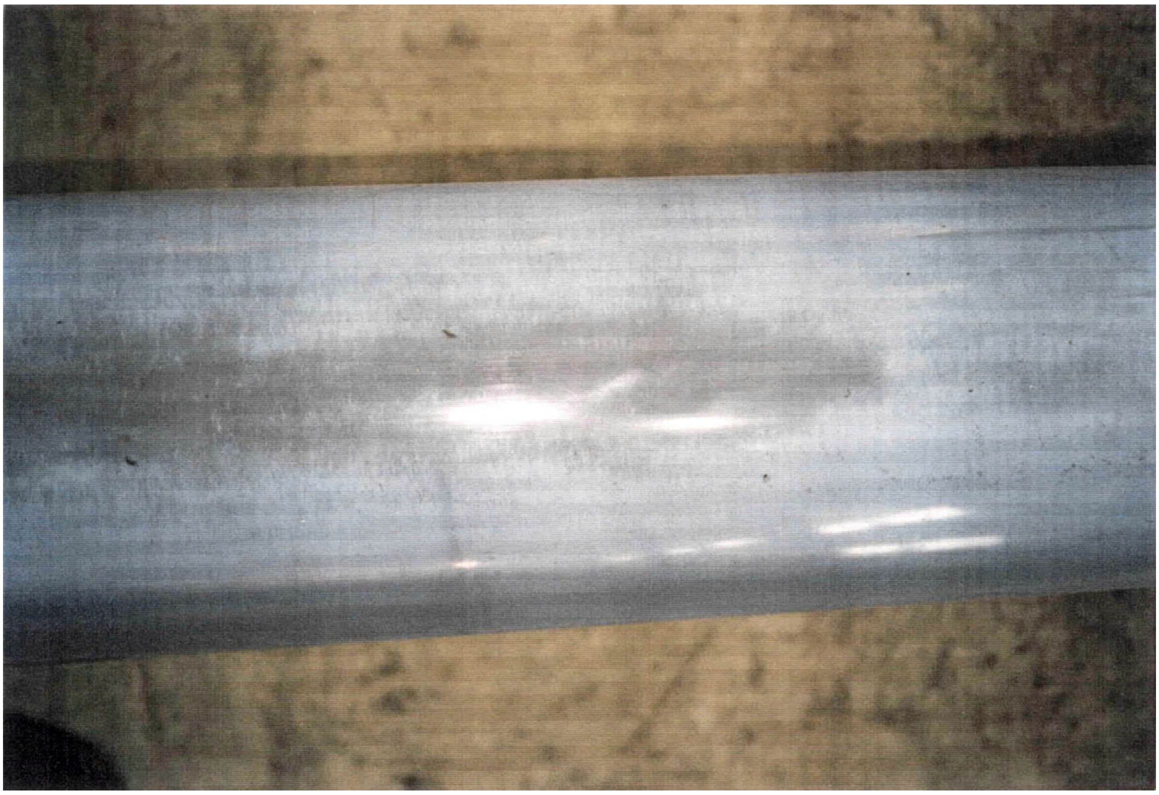


Plate 3 Limit of deposition – sediment on point of forming deposited bed

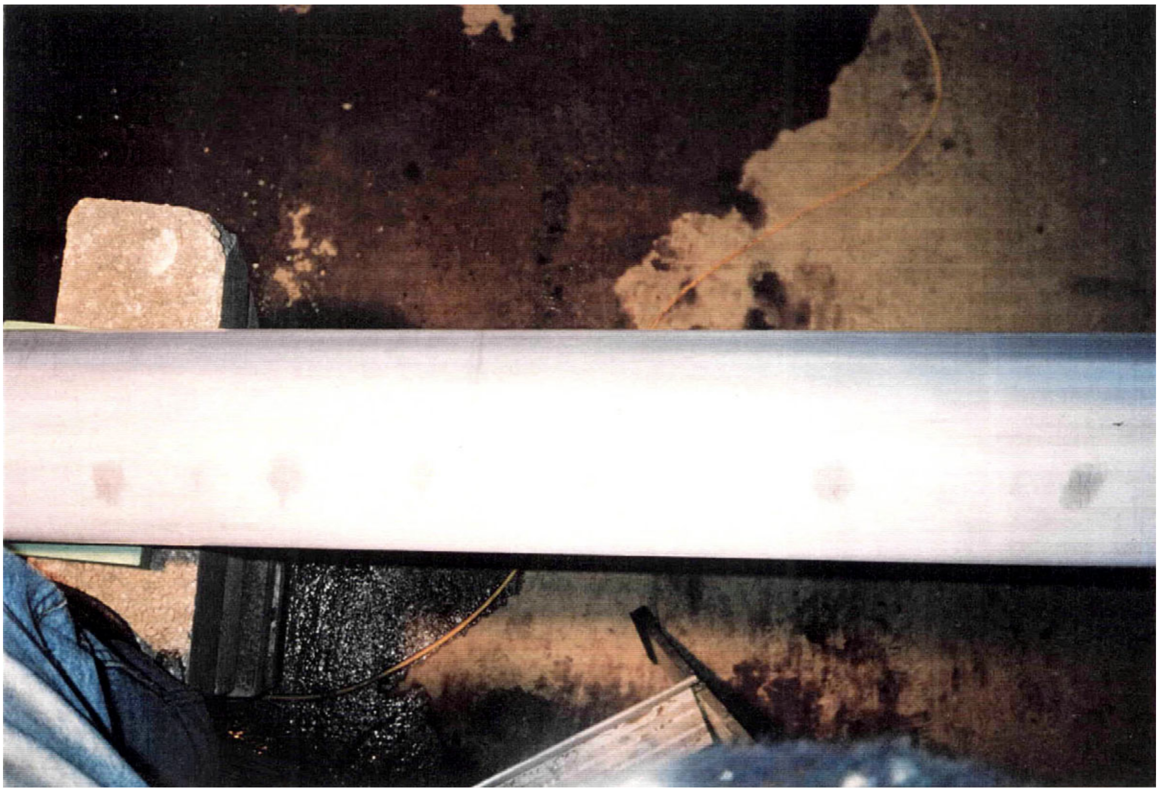


Plate 4 Small isolated dunes moving up pipe

Plate 5 Large isolated dune – viewed from above

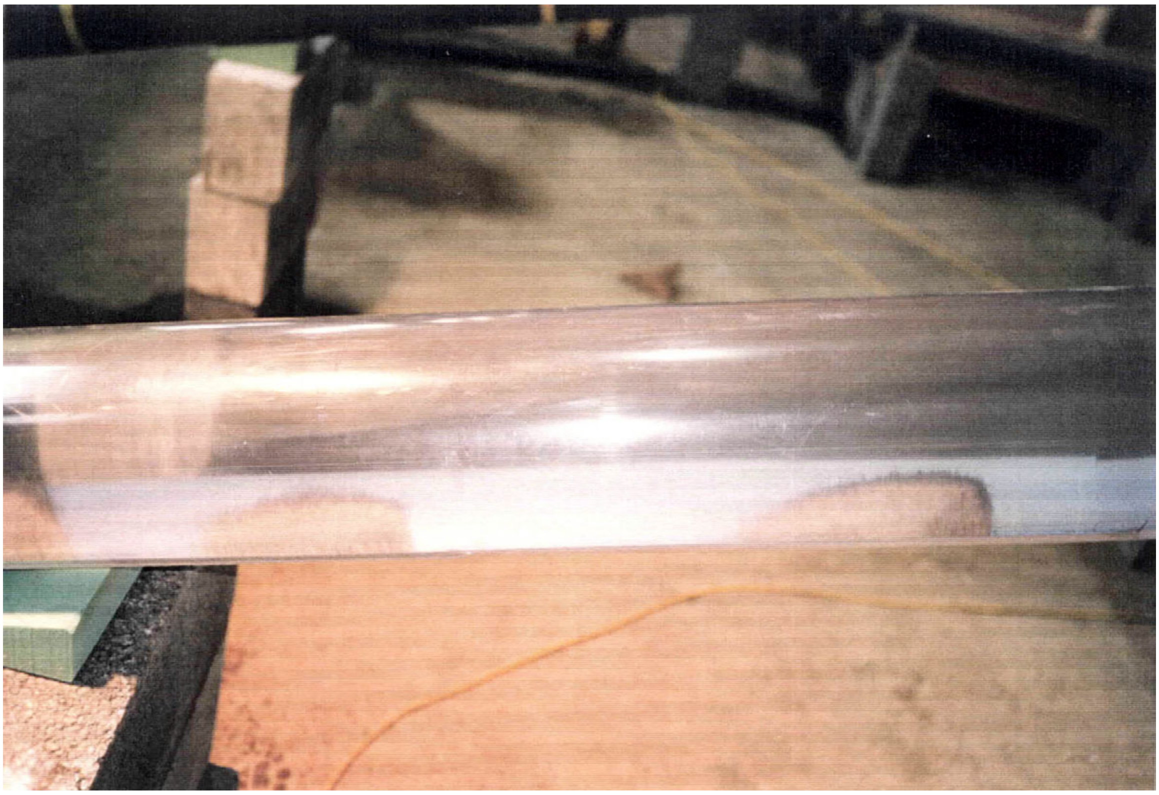


Plate 6 Large isolated dunes – viewed from the side

Appendices

Appendix 1

Questionnaire on inverted sewer siphons

NOTES FOR COMPLETION OF QUESTIONNAIRE ON INVERTED SYPHONS FOR SEWERS

1. The questionnaire is intended to obtain information on the general design requirements used by engineers for new inverted syphons for sewers.

2. The questionnaire is divided into two parts. Part A covers general features of syphons. The information in Part B may vary according to the type of sewer (eg. stormwater or foul and gravity main or pumped rising main); if so please photocopy page 2 of the questionnaire for each type of sewer considered. If a single design procedure is adopted for any kind of sewer, then the box containing "All" or "Either" should be ticked off.

3. In Part B, item B.6, if de-silting chambers are included, please give their location (eg. before entrance to system, at upstream leg or at downstream leg).

4. In Part B, item B.7, please describe briefly the structures used to divide flow between barrels (eg. weirs or gates).

QUESTIONNAIRE ON INVERTED SYPHONS

Name of Contact Person, Address and Telephone :

Name and address of Water Company for which syphons are designed (if different from above) :

A) General requirements for new inverted syphons

A.1 Preferred cross-sectional shape (eg. circular, horseshoe, etc)

CIRCULAR

A.2 Preferred type of syphon material (eg. concrete, cast iron etc)

VARIES

A.3 Approximate number of inverted syphons in Water Company region

30 - 50

A.4 Additional comments (continue on attached sheets, if necessary)

SPLIT FLOW CHAMBERS AT EACH END

A.5 Brief description of any problems in operation

PROBLEMS ARE RARE. BLOCKAGE IS THE ONLY SERIOUS PROBLEM AND IF THIS HAPPENS, BOTH ACCESS AND DIVERSION METHODS USUALLY APPEAR WANTING. WE USE VARIOUS TECHNIQUES TO OVERCOME THESE.

✓ Please tick the appropriate box
▹ Please fill in

B) General design requirements

If the design requirements vary with the type of sewer, please use photocopies of this page, indicating the type of sewer for which the information given below is relevant

B.1 Type of sewer

Stormwater	Foul	Combined	All
✓	✓	✓	✓

Gravity	Pumped	Either
✓	✓	✓

B.2 Minimum number of syphon barrels required. If more than one, please give criteria for determining number

▹ MINIMUM 2 FOR FLOW VARIATION AND OPERATIONAL DIVERSION. OTHER CRITERIA: FLOW + TRENCH/TUNNEL ECONOMICS

B.3 Slopes of syphon legs

Upstream leg		Downstream leg	
min	max	min	max
▹ 10° – 20°	▹ 90°	▹ 10° – 20°	▹ 90°

B.4 Minimum slope of "horizontal" leg

Flat	Other
✓ ✓	▹

B.5 Minimum flow velocity

▹ 0.7 – 1.1 m/sec DEPENDING ON DIAMETER OF PIPES

B.6 Provision of de-silting chambers

No	Yes. Please give location
✓ UNUSUAL	✓ ▹

B.7 Method of dividing flow between levels

▹ SPLIT FLOW CHAMBERS

Appendix 2

Review of literature on inverted siphons

Appendix 2 Review of literature on inverted siphons

A2.1 General

This Appendix summarises previously published data on design practice for inverted siphons and also gives details of some typical schemes that have been constructed. The information was evaluated and used (in conjunction with the HR experiments described in Part A of this report) to produce the general design guidelines presented in Chapter 7.

For ease of reference, this Appendix has a similar structure to that of Chapter 7. Thus, for example, Section 7.4 of the main report provides recommendations on the design of inlet chambers; background information supporting the recommendations can therefore be found in the similarly titled Section A2.4 of this Appendix.

A2.2 Alternatives to inverted siphons

(a) Erdos (1991) points out that inverted sewer siphons are intrinsically troublesome and alternatives should be sought where possible. Other solutions that may be appropriate include:

- Diverting the flow to another sewer or sewer system.
- Diverting the sewer to another location where it can be constructed under the obstacle at a continuous grade (eg, away from a section of road that is in cutting to an area where the road is on embankment).
- Reconstructing the sewer so that downstream of the obstacle it continues at a lower elevation that eliminates the need for an upward-sloping leg.

If options such as these are not possible, it is recommended to construct a gravity-flow inverted siphon rather than to use pumping (unless there is a large cost differential in favour of pumping). Systems using pumps tend to require more maintenance, have continuing energy costs, and are vulnerable to pump or power failures.

A2.3 Number of pipe barrels

(a) UK recommendations on the design of inverted sewer siphons were given in a Civil Engineering Code of Practice on Drainage (1950). This states that a single pipe barrel will not usually be able to ensure self-cleansing conditions because of the natural variations in flow rate that occur in gravity drainage systems. However, twin barrels should normally be sufficient for separate foul water systems or partially separate systems. For combined systems or larger schemes three or more barrels may be needed. The following design flow capacities for pipes in a multi-barrel design are suggested:

1st pipe	To take all flows up to 1½ DWF
2nd pipe	To take additional flows between 1½ DWF and 3 DWF
Other pipes	To take additional flows above 3 DWF

where DWF is the average dry-weather flow rate in the drainage system. For checking minimum velocities in a separate foul water siphon it is recommended that the minimum flow rate be assumed to be of the order of 1/3 DWF.

(b) ASCE (1970) states that some engineers consider that more than one siphon barrel is unnecessary for use with separate foul water sewers. Provided suitable minimum velocities occur daily, temporary deposits that form during low flows will be flushed through the system. However, in critical locations, a second barrel may be constructed purely as an emergency back-up. Single barrel siphons with diameters between 0.15m and 2.3m have been constructed in the USA.

For combined sewers it will normally be necessary to use more than one siphon barrel because of the wider range of flow rates. It is recommended that the primary barrel should be sized so that its flow capacity is equal to the maximum dry-weather flow rate.

(c) Imhoff, Müller & Thistlethwayte (1971) recommend that inverted siphons should normally have more than one barrel, with the first being designed to carry the peak dry weather flow rate [equivalent to about 2 DWF]. Single-barrel siphons may be adequate for small sewage flows but regular flushing is necessary.

(d) Escritt (1984) suggests that a single pipe barrel may be satisfactory for an inverted sewer siphon that does not have to carry discharges greater than about 6 DWF, provided the flow velocity at this peak discharge is not less than about 2m/s. However, if sufficient head is available, it would be better to aim for a maximum velocity of 4m/s to minimise the risk of any deposition. Twin barrels should be used if the maximum discharge exceeds about 6 DWF; the capacity of the pipe carrying the lower flow rates should typically be about 3 DWF.

(e) Erdos (1991) recommends that inverted sewer siphons should be assumed to have a design life of fifty years. Therefore, when determining design flow rates, account should be taken of foreseeable increases in flow rate during that period due to estimated population growth and planned changes in catchment usage.

It is recommended that one redundant barrel should always be provided in order to allow another barrel of the siphon to be closed for maintenance and repairs. Therefore a minimum of two barrels is needed; they should be of the same size and each capable of carrying the design flow rate. It there are three or more barrels, they should if possible be of the same size, but failing this the redundant barrel should be equal in size to the largest of the other barrels. No pipe or conduit in an inverted siphon should be smaller in diameter than 200mm because of the danger of blockage and the difficulty of cleaning.

A2.4 Inlet chamber

(a) Civil Engineering Code of Practice on Drainage (1950) recommends that inlet chambers (and also outlet chambers, see Section A.2.7) need to be large enough to allow room for personnel to enter and carry out maintenance work. Penstocks or diversion gates should be installed to enable individual siphon pipes to be closed to incoming flow.

(b) ASCE (1970) recommends the use of lateral side weirs to divide flow between multi-barrel siphons, but also shows an example of a US design with transverse weirs across the entrances to the twin barrels.

(c) Valentin & Kleinschroth (1975) used an inlet chamber similar to that of a vortex drop shaft in order to induce a spiral motion in the flow entering a model of an inverted siphon. The tests were made with a U-shaped siphon with vertical upstream and downstream legs. The spiral motion was claimed to assist the removal of air or gas from the flow. Instead of collecting along the soffit of the horizontal pipe, injected air bubbles were drawn to a central core in which the air was able to travel against the water flow towards the inlet chamber. A larger inlet chamber was needed than with a conventional design, and the energy associated with the spiral flow increased head losses at lower discharges.

(d) Escritt (1984) warns that the entrance to a siphon pipe should be designed so that it does not become surcharged at high flow rates. If surcharging does occur, undesirable scum and floating debris will accumulate in the inlet chamber. Sluice valves or penstocks should be provided at either end of a siphon pipe so that it can be closed off for cleaning.

(e) British Standards Institution Code of Practice BS 8005 : Part 1 (1987) suggests that, for short siphons, the invert of the inlet chamber may be designed to be the lowest point in the system, with the pipes beneath the obstacle sloping at an upward gradient from the chamber. For longer siphons, it is recommended to use a separate wash-out chamber for draining the system and removing deposited material.

A2.5 Sloping legs of siphon

(a) Civil Engineering Code of Practice on Drainage (1950) recommends that gradients of 1 : 5 (vertical : horizontal) are suitable for both the downward and upward sloping legs of siphons. However, it is noted that space limitations often make it necessary to use steeper gradients.

(b) ASCE (1970) recommends that siphon pipes should not have sharp vertical or horizontal bends that would make cleaning by modern methods difficult. The upward-sloping leg should not be so steep as to prevent the removal of heavy solids by jetting equipment. Some US agencies limit the upward slope to 15% (ie, 1 vertical : 6.7 horizontal), but steeper slopes are generally used.

(c) Imhoff, Müller & Thistlethwayte (1971) state that the upward leg of an inverted siphon should not be vertical and that its slope should be limited to 1 : 2 (vertical : horizontal).

(d) Shrestha & DeVries (1991) state that the upward and downward sloping legs of inverted siphons should not be steeper than 1 : 2 (vertical : horizontal); this recommendation relates to siphons for drainage and irrigation canals and is based on information given by Aisenbrey et al (1978)).

(e) Erdos (1991) recommends that the upward sloping leg should not have a gradient steeper than 15% (ie, 1 : 6.7) so as to allow sediment to be transported up into the outlet chamber.

A2.6 Horizontal leg of siphon

(a) There should be no change in diameter along a siphon barrel because this could hamper cleaning operations.

(b) Shrestha & DeVries (1991) recommend that the horizontal leg of an inverted siphon for drainage and irrigation canals should have a gradient of 1/200 (ie, S = 0.005).

A2.7 Outlet chamber

(a) Civil Engineering Code of Practice on Drainage (1950) points out the need for careful shaping of outlet chambers for multi-barrel siphons. This is because, during periods of low flow, it may be possible for water to recirculate within the chamber and cause sediment to be deposited into the ends of siphon barrels that are not operating. Means of closing off the ends of siphon pipes also need to be provided to enable cleaning or repairs to be carried out.

(b) ASCE (1960) comments that lateral confinement of the flow discharging from a siphon barrel (by means of side walls) helps prevent back-eddying of flow into barrels that are not operating.

(c) Escritt (1984) warns of the danger of backflow at the downstream ends of multi-barrel siphons. To avoid this, it is recommended that pipes carrying storm flows should be arranged to discharge at a level above that occurring when the dry weather-flow pipe is running at its maximum capacity (eg, 3 DWF). Alternatively, the pipes carrying storm flows can be arranged to discharge over weirs that separate them from the low-flow channel. Sluice valves or penstocks should be provided at either end of a siphon pipe so that it can be closed off for cleaning.

A2.8 Desilting and cleaning arrangements

(a) Civil Engineering Code of Practice on Drainage (1950) recommends that arrangements should be incorporated in the design to enable individual barrels of siphons to be closed for cleaning. A draw-off valve should be located at the lowest point of a pipe. In the case of multi-barrel systems, it may be possible to use a portable pump to remove water through the valve of the barrel being cleaned and discharge it via the corresponding valve in one of the other barrels that is still operational.

Access chambers may be constructed at either end of the horizontal leg of the siphon to permit rodding. The preferred alternative is to take the siphon pipes straight through the chambers but to fit them with hatch-box covers that can be removed when access is needed; this enables the chambers to remain dry during normal operation. If the chambers are constructed as open manholes, they will be permanently filled with water or sewage to a level determined by the outlet chamber and therefore need to be able to withstand the resulting hydrostatic pressures ; also, floating material will readily collect in the chambers and produce objectionable conditions.

Means of flushing may be necessary where velocities in an inverted siphon are inadequate to maintain self-cleansing conditions. One possibility for a small siphon is to use a penstock to increase temporarily the water level in the upstream length of sewer; on opening the penstock rapidly, the increased head difference acting on the siphon will generate higher than usual flow velocities through the pipes for a limited time. If a siphon is located just downstream of a pumping station, flushing flows may be produced by suitable operation of the pumps.

(b) Escritt (1984) recommends that, if possible, hatch boxes should be installed at intervals of 100m along siphons to allow rodding to be carried out. A wash-out valve should be installed at the lowest point of a siphon so that it can be emptied into a manhole from which it can be pumped or discharged under gravity.

A permanent flushing system may be possible in the case of systems where the available head difference across the siphon is insufficient to produce self-cleansing velocities. Flow from the upstream sewer is diverted to a holding tank until it is full enough to trigger discharge by a cistern-type siphon (of a design suitable for use with raw sewage). The volume of the holding tank should be at least equal to the volume of water contained in the pipes of the inverted siphon. If the amount of permissible backing-up of flow in the upstream sewer is limited, the holding tank will need to be relatively shallow and have a large surface area; this may cause sediment to settle in the tank and increase the possibility of the cistern-type siphon becoming blocked. An overflow weir should be installed so that flow can bypass the holding tank. This type of flushing system is applicable only in a minority of cases.

A2.9 Minimum velocities in pipes

(a) According to Civil Engineering Code of Practice on Drainage (1950), deposition problems can occur if velocities of 1.2m/s are not regularly achieved in the smallest pipes of inverted siphons. It is stated that sand tends to deposit at velocities below about 0.45m/s, but higher velocities of the order of 0.7m/s to 0.75m/s are needed to erode deposits once they have formed. If the deposited material becomes bound by organic matter, the velocity required for erosion may increase to between 1.2m/s and 1.4m/s. The Code gives the following values of velocity needed to move different sizes of stone (having a specific gravity of $s = 2.55$) :

Stone size (mm)	Minimum flow velocity (m/s)	
	Horizontal pipe	Vertical pipe
50	0.9	-
25	0.6	1.0
12.5	0.45	0.75
6.25	0.3	0.45
2.5 (sand)	0.7*	0.25

Note : Velocity needed to lift sand into suspension

(b) ASCE (1960) suggest that minimum velocities to avoid sedimentation are 0.6m/s to 0.9m/s for domestic sewage and 1.5m/s for combined sewage.

(c) ASCE (1970) recommends that the primary barrel of a siphon should be sized so that a velocity of 0.6m/s to 0.9m/s is reached at least once each day.

(d) Imhoff, Müller & Thistlethwayte (1971) state that inverted sewer siphons should be designed for a minimum velocity of 1.2m/s.

(e) Erdos (1991) recommends that a siphon pipe should be sized so that, if possible, a velocity of at least 1.2 m/s is achieved every day, with an absolute minimum of 0.6m/s. If this is not possible, the siphon may need to be operated on an intermittent basis, with flow being temporarily stored in the upstream sewer. Alternatively, water from the public supply or elsewhere may be used to produce a daily flushing flow.

A2.10 Head losses

(a) Civil Engineering Code of Practice on Drainage (1950) recommends that allowance be made for some limited sediment deposition when calculating head losses in siphon pipes. A roughness value in the Manning resistance equation of about n = 0.015 was suggested.

(b) ASCE (1970) recommends using a conservative value of Hazen-Williams C = 100 for calculating head losses in siphon pipes. This is stated to be equivalent to Manning roughness values of n = 0.014 in small pipes and n = 0.018 in large pipes [which appears the opposite of what would normally be expected].

Splitting flow between siphon barrels by means of weirs introduces extra energy losses; these reduce the head available to the barrels that come into operation at higher discharges. The energy loss at a weir can be assumed to be equal to the head over the weir when it is passing its design rate of flow.

(c) Escritt (1984) points out the importance of determining accurately the head difference available between the two parts of a gravity drainage system connected by an inverted siphon. Values of the head difference at both high and low rates of flow need to be known.

(d) Shrestha & DeVries (1991) reviewed data on head losses in pipe bends given by Aisenbrey et al (1978) and recommended the following formulae:

$\zeta_B = 0.003\phi$ for $\phi \leq 10^o$

$\zeta_B = 0.004\phi - 0.01$ for $10^o \leq \phi \leq 25^o$

where ζ_B is the non-dimensional loss coefficient for the bend (see Equation (B.7)) and ϕ is the angle turned by the bend (in degrees).

The authors recommended that allowance should be made for the extra head loss produced in pipes by the presence of sediment, particularly in the case of long siphons where the frictional resistance is the dominant factor. A formula due to Graf & Acaroglu (1967) for the homogeneous movement of suspended sediment in conduits was recommended.

(e) Erdos (1991) recommends that a Manning roughness value of n = 0.014 be used to calculate frictional losses irrespective of the type of pipe used.

A2.11 Air movement induced by flow

(a) ASCE (1970) highlights the problems caused by the movement of air along a sewer that is flowing part full. An inverted siphon intercepts the flow of air and can cause a build-up of air pressure in the inlet chamber equivalent to several inches of water head. If the chamber is sealed, air may force its way out through manholes and connections farther upstream. Alternatively, if the air is vented from the chamber, it may produce serious odour problems. In a similar way, flow in the downstream sewer will tend to draw air from atmosphere into the outlet chamber. A solution to the problem is to use a separate pipe to carry the intercepted air from the inlet chamber to the outlet chamber; this pipe is termed an air line or air jumper. The air line may be suspended above the hydraulic grade line of the sewer or may be run alongside the siphon. A method of draining condensate from the air line needs to be provided. The diameter of the air line is typically half that of the siphon barrel.

(b) Valentin & Kleinschroth (1975) quote research by de Lara and Veronese that the water velocity in a nearly horizontal pipe needs to exceed 0.4m/s in order to be able to transport air along with the flow. In a downward sloping pipe, air may be able to travel against the direction of the water flow and vent back into the inlet chamber of the siphon.

(c) Escritt (1984) states that manholes or chambers of an inverted siphon should be provided with ventilating columns or other means of ventilation to cater for the air flow in the sewer intercepted by the siphon.

(d) Erdos (1991) states that venting of untreated air from an inverted sewer siphon to the atmosphere is not normally acceptable in the USA because the noxious odours can cause a public nuisance or hazard. In the case of siphons beneath highways, the usual solution is to provide a high level air line to transport intercepted air from the inlet chamber to the outlet chamber, from where it will be drawn away by the flow in the downstream sewer. The air line may conveniently be mounted in or supported by a structure such as an overbridge. Erdos (1991) recommends use of the following empirical method for sizing the air line: viz, the cross-sectional area of the air line should be twice the cross-sectional area above the water surface in the upstream sewer under design conditions. If an air line is not used, any air or gases discharged to atmosphere must be cleaned or scrubbed by an activated charcoal filter or equivalent method of treatment.

A2.12 Sulphide generation

(a) ASCE (1970) highlights problems that can be caused by the generation of hydrogen sulphide (H_2S) in long inverted siphons. If sewage is out of contact with air for a long enough time, significant amounts of dissolved sulphide can be formed within the flow. This is able to escape into air pockets that form within the siphon and can lead to serious corrosion of pipe walls; additionally it may cause odour and safety problems in the downstream sewer. For warm sewage or sewage with a high BOD, sulphide generation may be significant if the time of travel through the sewer during periods of low flow is of the order of 10 minutes or longer.

A2.13 Structural and materials aspects

(a) Civil Engineering Code of Practice on Drainage (1950) mentions the need to ensure that inverted siphons constructed on or under river beds should have sufficient weight to prevent flotation. If the pipes are on or just below the bed, they should be protected from erosion and possible movement by the flow.

(b) Erdos (1991) gives details of types of pipe that are suitable or unsuitable for use in inverted siphons.

Suitable

- **reinforced concrete pipe or box units** – commonly used due to high strength, economy and abrasion resistance. Corrosion by sewage gases is possible if a siphon is emptied and air allowed to enter; corrosion can be prevented by use of PVC sheet as a liner.
- **steel** – high strength and suitable for air lines, but needs lining or coating to prevent possible severe corrosion by sewage gases.
- **cast or ductile iron** – high strength and specially suitable for smaller siphons. Internal protection is needed against corrosion by sewage gases.
- **vitrified clay pipe** – relatively high strength and resistant to corrosion, but not able to sustain high internal pressures.
- **aluminium** – needs some corrosion protection and too expensive for most situations.
- **PVC, ABS and polyethylene** – resist corrosion but need to have suitable pressure rating.

Unsuitable

- **non-reinforced concrete** – develops cracks and is neither watertight or airtight.
- **asbestos cement** – liable to corrosion and health hazard.
- **corrugated pipes (steel, aluminium or plastic)** – not usually pressure rated and inadequate watertightness and structural strength.
- **fibreglass, polyester, GRP, other fibre-reinforced materials** – poor service record due to problems such as de-lamination and joint distortion.

A2.14 Air-cushion siphons

(a) The concept of the air-cushion siphon was devised by R Stahn of German consultants GITEC and described in an article in World Water (1983) based on a paper by Kuntze (1982). In essence, air is introduced to the horizontal leg of the siphon under sufficient pressure to overcome the hydrostatic pressure of the water and cause the pipe to flow part full. Adding or removing air alters the cross-sectional area of flow in the pipe and thereby makes it possible to maintain self-cleansing velocities over a wide range of flow rates without the need for multiple barrels. Swan necks are used at either end of the horizontal leg to trap air in the crown of the pipe. Injection of air at a steady rate is necessary to make up for air that is entrained into the flow by turbulence and carried out of the siphon. This helps to remove sewage gases that would otherwise tend to build-up in the crown of the pipe. Automatic equipment is used to adjust the air pressure in the siphon in response to changes in the water level in the upstream sewer. Details of two air-cushion siphons that have been constructed are given in A2.15(e).

A2.15 Examples of siphons

(a) ASCE (1960) gives an example of a three-barrel inverted sewer siphon constructed under an expressway in Chicago, USA. The siphon connects upstream and downstream sections of a sewer that measures 3.66m wide by 2.90m high. Side weirs are used in the inlet chamber to split flow between the three barrels which have diameters of 1.37m, 2.13m and 2.74m. The downward and upward sloping legs of the siphons are at angles of about 50^o to the horizontal (ie, 1 vertical : 0.84 horizontal). Each horizontal leg is 86.5m long and slopes downwards in the direction of flow; the pipe gradients are 1/570 for the two largest barrels and 1/189 for the 1.37m barrel. The lowest point in the system is 4.2m below the invert level of the upstream sewer. In the outlet chamber, the barrels discharge over weirs that are 0.73m above the invert level of the downstream sewer. The difference in invert level between the upstream and downstream sewers is only 46mm.

(b) Wilkinson (1966) gives details of the Mailsi inverted siphon in Pakistan which was constructed to carry flow from an irrigation canal under the Indus river. The siphon has four barrels (each measuring 4.1m square and about 565m long) which are contained within the structure of a concrete gated-weir that controls flows in the Indus and prevents local scouring of the bed. The sloping sections of the siphon are inclined at an angle of about 20^o to the horizontal (ie, 1 vertical : 2.75 horizontal), and the horizontal section is about 450m long. The maximum design discharge for the irrigation canal was 147 m^3/s, corresponding to a maximum flow velocity through the barrels of the siphon of 2.2m/s. The system was designed to limit the total head loss to 0.67m under these conditions. The head loss due to the frictional resistance of the barrels was estimated to be 0.50m (based on a Manning roughness value of $n = 0.014$), with point losses at locations such as the entrance and exit and at bends accounting for the remaining 0.17m.

(c) Steel (1968) describes an inverted sewer siphon constructed under the River Avon as part of the Bristol regional foul water drainage scheme. The siphon consists of two concrete pipe barrels with diameters of 0.91m and 1.52m. The smaller pipe is designed to carry flow rates up to 2 DWF with peak daily velocities of 1.2m/s to 1.5m/s. Flow from an upstream sewer turns through a horizontal angle of about 98^o within an inlet chamber containing a side weir that supplies flows in excess of 2 DWF to the larger of the two pipes. The downward legs of the siphon are inclined at 30^o to the horizontal (ie 1 vertical : 1.73 horizontal) and descend through a vertical distance of 34.0m. The near-horizontal sections of pipe are completely surrounded in concrete; each is about 125m long and has a fall of 0.3m in the direction of flow. The pipes then rise vertically through a height of 33.6m and discharge to the downstream sewer through horizontal openings that can be closed by stop logs. The vertical pipes are contained within a shaft that provides access to the bottom of the siphon; at this point each pipe is fitted with a tee and a cast iron cover that can be removed for cleaning purposes. The difference in invert level between the sewers just upstream and downstream of the siphon is 0.71m. The approximate cost in 1966 of constructing the siphon was £90,000

(d) The following information on inverted sewer siphons in France was obtained during this study by courtesy of French consultants OTUI.

A siphon was constructed at Place de la Porte de Versailles in Paris to divert a sewer beneath a railway station. The single barrel has an internal diameter of 1.5m with vertical upstream and downstream legs. The transition between the sewer and the vertical shaft has an inner radius of 1.5m. The horizontal leg is about 52m long and slopes downwards in the direction of flow at a gradient of 3%; the lowest point is about 13.7m below the invert level of the upstream sewer. The inlet and outlet chambers are located directly above the vertical shafts so as to provide access.

At Bezons a 4.0m diameter sewer crosses under a river by means of an inverted siphon. At its upstream end the siphon curves sharply downwards to a slope of about 45^o followed immediately by a reverse bend with an outer radius of 7.25m. Within this transition the pipe diameter reduces to 3.25m. The pipe then slopes down under the river at a gradient of about 1 vertical : 8 horizontal.

A single barrel siphon with a diameter of about 3.0m was constructed under the river Seine at Chatou. A long-radius bend connects the upstream sewer to the downward sloping leg which makes an angle of 44.6^o to the horizontal. The horizontal section under the river is 110m long and slopes downwards in the direction of flow at a gradient of 1/109. The upward sloping leg makes an angle of 30.6^o to the horizontal and joins the downstream sewer by a transition with a vertical radius of curvature of about 32m. At the upstream end of the siphon there is a gate chamber and a second chamber for introducing a cleaning ball. A discharge channel upstream of the gate chamber allows flow to be diverted from the sewer into the river if the siphon needs to be closed.

(e) World Water (1983) describes the Norderelbe air-cushion siphon (see A2.14) that has been built under the river Elbe in Germany. It has twin barrels each of 2.4m in diameter and about 610m long. Flow from the inlet chamber enters each barrel by means of an inlet shaft measuring 1.3m by 2.5m in plan and three smaller inlet pipes; each shaft and inlet pipe has a swan neck to retain air in the crown of the pipe. Rapid variations in the water level in the horizontal leg can be achieved by pumping or removing air directly from the crown of the pipe; losses due to air being entrained by the flow are made up by steady injection of air through the inlet pipes. The operating pressure for the air is 0.5 bar (equivalent to about 5.1m head of water). The system has been shown to be capable of maintaining self-cleansing velocities for water flow rates varying between $1m^3/s$ and $10m^3/s$. Another air-cushion siphon has been constructed at Basel, Switzerland using 2.0m diameter pipes and an operating pressure of 1.8 bar (equivalent to about 18.3m water head).