Highways Agency

Design of Pipes for Road Drainage

Literature Review

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Literature Review

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Executive Summary

This research and development project is intended to improve the understanding of the sediment loads from high-speed roads and to develop guidance for the future design and maintenance of road drainage systems.

The formal objective of the study is to bring together the hydraulic design of pipes into one Highways Agency Advice Note.

The study is to identify regional or geographical variations in the sediment load of pipe lines and to develop regional coefficients or a map based on anticipated sediment.

Deterioration of the pipe condition either by sediment deposition, pipe degradation, and corrugations forming in twin walled plastic pipes, or lack of maintenance needs to be factored into the hydraulic design.

The structural design of pipes, in particular plastic pipes and the bedding characteristics do not form part of this study, although monitoring the compaction of the side-fill to non rigid pipes is an issue still to be addressed.

The objectives are to be achieved via three stages covering a total of 18 months:

- Stage 1: Literature Review
- Stage 2: Data collection and development of formulae and draft of Advice Note
- Stage 3: Technical assistance with publication of the Advice Note

There is also a requirement to provide ad-hoc technical assistance during the course of this study, which effectively forms Stage 4 of the project.

This report is the output from Stage 1 of the project and identifies the guidance that is presently available to designers. In particular the review focuses on the roughness coefficients used and the effect of sediment within the pipe line. The study of sediment volumes arising from the carriageway is reviewed to establish the feasibility of applying coefficients for the adjacent land use and regional variations that may assist in the design. The conclusion is that land use may vary during the life of the carriageway and consequently this factor may not be appropriate.

Contents

1. Introduction

This Literature Review is the second output from the Highways Agency SSR Framework Task 163 and completes Stage 1 of the project. This report details the results of the review that may be used or developed to progress the Stage 2 Data collection and formulae development.

This review examines the current knowledge and the data available and identifies the gaps that may need to be filled in order to complete the Advice Note.

The aim of the review is to identify regional or geographical variations in the sediment load of pipelines and to develop regional coefficients or a map based on anticipated sediment. The review also identifies the assumptions used in hydraulic design in terms of minimum velocities and gradients. Data that takes account of the deterioration of the pipe condition, either by sediment deposition, pipe degradation, corrugations forming in twin walled plastic pipes, or lack of maintenance is reviewed and any amendments to these assumptions, when applied to highway drainage design, that are appropriate are identified for further analysis in Stage 2 of the project.

All flows in drainage pipes carry sediment to a greater or lesser degree, in one of the three following transport modes (or in a combination of more than one):

- Transport of fine sediment in suspension
- Transport of coarser sediment as bed load
- Erosion of deposited sediment bed.

Even in pipes without any sediment deposits, the discharge capacity of the pipe is reduced by up to 4% due to increased energy losses caused by the movement of sediment along the pipe invert (CIRIA 141[1]). With deposits, the effect of sediment is obviously more pronounced as it results from the combination of a reduction of cross-sectional area and an increased bed roughness. For small deposits, i.e. a depth of deposit corresponding to a few percent of the pipe diameter, the main factor in loss of discharge capacity is the increased bed roughness.

Where sediment is a design concern, pipes should be designed to achieve “self-cleansing” conditions which either prevent sediment from depositing or accept that a certain deposition is not detrimental and produce a balance between the processes of deposition and erosion during a specified period. The choice between the above two options is usually dictated by the need to minimise engineering costs, namely excavation and maintenance costs.

2. Project purpose

There is little guidance in the current Highways Agency documents on the minimum velocities and gradients necessary to maintain “self cleansing” velocities in highway drainage systems, in particular drainage pipelines. The recent Highways Agency research project to monitor the amount of sediment that enters the highway drainage system from the carriageway and surrounding area has produced a significant amount of data that needs to be brought in to the design process.

The objectives from the Inception Report are reproduced below:

- Identify data on the regional variations in the volume of sediments that can enter the drainage systems and consequently have a detrimental impact on the capacity of the pipes. Where these regional variations can be quantified, a suitable coefficient is to be developed so that future design will take sediment load variability into account. If practical, formulae will be developed to predict the anticipated sediment load.

- Develop guidance on the friction coefficients, k_s mm (or Manning’s “n” values), that are appropriate with sediment deposition and regional variations that may be appropriate. With this will be a coefficient (or safety factor) to ensure the probability that maintenance will be inadequate is taken into account during the design.

- Energy losses caused by multiple side connections such as gully connections affect the hydraulic performance of pipes; hence a secondary objective is to provide guidance on the design of pipes with multiple side connections.

- The guidance resulting from this project will form the basis of an Advice Note for inclusion in Volume 4 of the Design Manual for Roads and Bridges (DMRB[2]). In addition, should amendments to existing text be required, this text shall be drafted for approval by the Environmental Projects Board.

It is important to ensure that the potential for sediment deposition within the pipeline is fully taken into account during the design process and that adequate guidance on how this can be achieved is provided. This should ensure that pipes are designed to minimise the maintenance element and reduce the risk of carriageway flooding.

3. Current hydraulic design guidance

3.1 Documents

The principal document in the Highways Agency's **Design Manual for Roads and Bridges**[2] that specifies the criteria for the design of highway drainage systems is **HD33: Surface and sub-surface drainage systems for highways**. This document does not contain any guidance on minimum flow velocities and minimum gradients necessary to ensure that sediment remains in suspension and is not deposited in the pipeline. This is the reason for this project.

Further Highways Agency guidance is contained in the **HA105: Sumpless gullies**; this provides advice on minimum flow velocities and minimum gradients relating to the movement of increased sediment loads in pipelines due to the elimination of the sump from the gully pot. The advice in HA105 is reviewed in Chapter 4.

Elsewhere, the design of surface water sewers is often based on the guidance given in **Sewers for Adoption**[3], a design guide produced by the water industry primarily for the design of sewers for new developments. The required hydraulic design criteria for surface water sewers (2.13.4 and 5) is that the minimum velocity should be 1m/s at pipe full flow and the pipe roughness coefficient (k_s) should be 0.6mm.

BS EN 752[4]: Drain and sewer systems outside buildings Part 4 Hydraulic design and environmental considerations, is the British Standard for the design of foul and surface water sewers. Clause 8, self cleansing velocities, states that for pipes smaller than 300mm, self cleansing of sewer pipes can be achieved by ensuring either a velocity of at least 0.7m/s or that the pipe gradient is no flatter than 1:DN, where DN is the pipe diameter. This criterion relates to both foul and surface water, but the Standard indicates that steeper gradients may be required for drains, i.e. surface water conveying pipes.

The Standard states that for larger diameter drains and sewers (300mm diameter and larger) higher velocities may be necessary particularly if relatively coarse sediment is expected. While coarse sediment is not defined, it is a reasonable assumption, supported by findings of monitoring programmes, that highway drainage pipelines will be prone to the ingress of coarse sediment.

Clause NA.3.1 of the National Annex (NA) pertains to combined and surface water sewers. This states that the minimum diameter for surface water drains should be 75mm but notes that sewerage undertakers will not normally adopt sewers less than 150mm diameter.

Drains and sewers of 75mm and 100mm diameter should be laid no flatter than 1:100 (NA.3.2); however this is not relevant to highway drainage. (It was recommended that the minimum pipe diameter should be 150mm, see buried pipelines reports). The National Annex (NA.3.3) states that surface water drains and sewers of DN 150 to DN 900 should be designed to achieve a self cleansing velocity of 1.0m/s in pipe full conditions. Where higher minimum velocities are required for larger sewers, the Standard refers to CIRIA R141[1]: Design of sewers to control sediment problems.

The guidance in Annex NA is based on average sediment conditions and assumes that a sediment deposit up to 2% of the pipe diameter can be accepted for the types of pipe used in road drainage.

With regard to pipe surface roughness the Standard (NB.2.2) assumes the presence of grit within the pipe and states a roughness coefficient (k_s) of 0.6mm and refers to the Tables for the Hydraulic Design of Pipes and Sewers[5] for further guidance on roughness coefficients (k_s) for different pipe materials.

The Standard raises the issue of head losses at manholes and bends, which is an issue that is generally ignored in longhand design but which is relevant for consideration in highway drainage where multiple pipeline entries occur due to gully connection. Local head losses are approximately proportional to the square of the flow velocity, V, and they are usually defined by a head loss coefficient, which is the ratio of the head loss and the velocity parameter $V^2/2g$ (kinematic head).

The head loss coefficient data contained in Tables NB.2 and NB.3 of the National Annex relate to losses when the pipe is flowing under surcharged conditions. Flows at pipe full condition will be less than the tabulated values, although the conditions where junctions occur in manholes or at catchpits, are not tabulated and the head loss coefficient could be expected to be greater.

The tables from BS EN 752 are reproduced below:

Table NB.2 – Headloss coefficient, k_L for manholes

Plan shape of manhole	Headloss coefficient, k_L		
	Type of manhole		
	Straight through	30° bend	60° bend
Rectangular	0.10	0.40	0.85
Circular	0.15	0.50	0.95

Table NB.3 gives values of k_L for 90° circular bends, flowing full for various ratios of bend radius, R, to nominal bore, D. These values only apply where the length of straight downstream pipe exceeds 30 pipe diameters.

Table NB.3 – Headloss coefficient, k_L at bends

Bend radius/pipe diameter R/D	Headloss coefficient, k_L
0.5	1.00
1.0	0.25
1.5	0.18
2.0	0.16
5.0	0.18
10.0	0.24

Tables for the hydraulic design of pipes and sewers[5] by HR Wallingford and Barr: This publication, now in its 8th edition, has been widely used by engineers for the design of pipe systems. It is a tabulation of the Colebrook-White equation covering diameters from 20mm to 4000mm. The tables are arranged in terms of incremental pipe roughness values (k_s) and give flow rates and velocities for varying pipe gradients. A list of recommended roughness values for different pipe materials is provided, where pipes are classified according to pipe material and condition. “Good” and “normal” condition are associated with new and clean pipes, from which it can be inferred that “poor” condition may be associated with old and/or pipes with sediment. The aim of the project is to produce a similar series of tables of pipe diameter and gradient with varying roughness coefficients (k_s value), and potential sediment ingress.

This document also provides some design information on how to make allowance for head losses at manholes. At straight through manholes, the suggested head loss coefficients K are (these exclude any junctions at the manhole):

	Values of K	
	Part-full	Full-bore
Open-channel manhole	<0.1	0.05 – 0.25
Open-channel manhole with bend	0.3	1.5
Open-channel manhole with pipe bend beyond manhole	0.3	0.3

Stormwater Drainage Manual, Planning, Design and Management, Hong Kong Drainage Services Department[6]**, 1995:** Guidance is given in this manual on the local loss coefficients to apply for the design of sewer networks to account for inlets, outlets, bends, joints, manholes, etc. The recommended values were based on advice by Consultants AB_2H, and an amended version is tabulated below.

Type of loss	K	Type of loss	K	Type of loss	K
Entry losses		**Intermediate losses**		**Exit losses**	
Sharp-edged entrance	0.50	Elbows[a] 22.5° 46 90	 0.20 0.40 1.00	Sudden enlargement	1.00
Slightly rounded entrance	0.25	Close radius bends[b] 22.5° 25° 90°	 0.15 0.30 0.50	Bellmouthed outlet	1.00
Bellmouth entrance	0.05	Long radius bends[c] 22.5° 45° 90°	0.10 0.20 0.40		
		Sweeps[d] 22.5° 45° 90°	 0.05 0.10 0.20		
		Mitre elbows 22.5° 30° 45° 60° 90°	 0.15 0.20 0.30 0.25-0.65 0.30-1.25		
		Tees Flow in line Line to branch or branch to line: - sharp-edge - radiused	 0.35 1.20 0.80		
		Angled branches Flow in line Line to branch or branch to line: - 30° angle - 45° angle - 90° angle	 0.35 0.40 0.60 0.80		

Notes:
a – Radius/Diameter≈0.5
b - Radius/Diameter≈1
c - Radius/Diameter= 2 to 7
d - Radius/Diameter=8 to 50

Urban Drainage Design Manual – Hydraulic Engineering Circular No. 22, 2nd Edition, US Department of Transportation[7], 2001: Among other types of local head loss, HEC 22 gives recommendations for head loss coefficients at inlets or manholes ("access holes", as they are known in the USA):

		Values of K
Inlet	straight run	0.50
	angled 90°	1.50
	angled 60°	1.25
	angled 45°	1.10
	angled 22.5°	0.70
Manhole	straight run	0.15
	angled 90°	1.00
	angled 60°	1.85
	angled 45°	0.75
	angled 22.5°	0.45

As can be seen from the information summarised above, the various design documents available for the estimation of local head losses do not specifically mention the effect of sediment in pipes. However, it can be inferred that the possible effect of sediment is incorporated in the recommended design values for the coefficient K. This issue is discussed further in Section 4.2.

3.2 Software analysis

There is no known software available to predict the rate of generation of sediment on highway surfaces related to road layout, location etc. One of the outputs of this project could be a system, possibly using GIS or a paper/ PDF map, that would aid this essential input function.

DMRB contains no advice on selection of suitable drainage design software. There are, however, several commercial software tools available that can be used to design or analyse the performance of highway drains. Sediment generation and movement are not explicitly modelled in most of the software packages.

The packages may not be specifically or solely intended for highway drainage, but are applicable to foul and surface water drainage in general.

The main packages used are:

- HydroWorks/ InfoWorks (Wallingford Software Ltd.)
- WinDes and WinDAP (Micro Drainage Ltd.)
- Mike Urban/ Mouse (DHI Water & Environment)
- Pipeflow (Thomas Telford Ltd.)

These are discussed in turn below, based on experience and information contained in material provided by the publishers of the software. A number of other drainage-related programs are also discussed. In addition, some consultants have their own in-house software, which may be as simple as a spreadsheet for small systems: these have not been reviewed, but it may be possible to do so as part of this project.

3.2.1 HydroWorks/ InfoWorks CS (Collection Systems)

This is the de facto UK industry standard for the analysis of existing sewerage systems, its main users being water utilities and their consultants. HydroWorks has been superseded in recent years by InfoWorks, which uses essentially the same simulation engine but has an improved map-based graphical interface and a database instead of file-based format. This keeps an audit trail of all changes and ensures that project data is kept together, minimising the risk that incorrect versions of models are used. It features sophisticated and flexible presentation of results that allows the cause of problems to be pinpointed and the effect of proposed changes to be highlighted.

The simulation engine uses a numerical method to solve the St Venant equations at each computational node and each timestep to simulate time-varying flow in a network, which can be looped and contain branches, pumps, weirs etc.; it can deal with both closed pipes and open channels.

Its main strength is in simulating the behaviour of existing systems and evaluating proposed improvements/ alterations, including sustainable drainage systems (SuDS) measures. It can also be used for completely new systems, but does not have specific features to help a designer to, for example, lay out a drainage network or optimise sizes and gradients or produce drawings for construction. It can, however, import and export data from and to CAD and GIS packages.

InfoWorks CS features explicit modelling of sediment and pollutants, needed mainly to simulate the impact of overflows into rivers, subject to purchase of an add-on licence. As pollutants can be attached to sediments on the contributing surfaces, in gully pots and in sewers, an accumulation of sediment often causes a concentrated “first flush” of pollution in a spill from an overflow, so it is important to be able to model both the deposition and erosion of sediment. Two sediment fractions can be modelled, and parameters can be varied to suit circumstances.

A strength of the software is the option to make sediment deposition and erosion affect the hydraulic simulation, at the expense of extended run times. This is of particular value in situations where sediment is readily deposited and rarely scoured away due to very slack gradients, and builds up to a significant depth over time so that it restricts the capacity of the pipework.

3.2.2 WinDes and WinDAP

Microdrainage software comes in two variants. WinDes is aimed at designers of new systems, while WinDAP focuses on simulation of existing systems in much the same way as InfoWorks. WinDes is commonly used to design new road and estate drainage, and has a full set of tools to facilitate layout and optimise sizes and gradients within definable parameters such as minimum pipe full velocity, gradient or overall depth.

Minimum velocities are used in WinDes to prevent settlement, and reports in WinDAP can be used to identify areas where, for example, velocities do not exceed a definable self-cleansing value in a one-year time series of storms. There is no specific modelling of sediment. This is not necessarily a great disadvantage, provided that the minimum velocities are correctly identified outside the software.

3.2.3 Mike Urban/ Mouse

This software has similar functionality to InfoWorks. It uses different algorithms and a different interface, and is more commonly used in continental Europe than in the UK. It includes, also subject to purchase of the appropriate licence, explicit modelling of pollutants and sediment.

3.2.4 Pipeflow

Pipeflow is a simple piece of software that can be used to identify depth and velocity of flow in a variety of shapes of pipe and open channel, both full-bore and part filled, in much the same way as the Wallingford Tables[5]. It can also be used to calculate the minimum gradient to achieve a given velocity at a known flow, making it suitable as an aid to the design of small or simple drainage systems. Sediment is not explicitly represented.

3.2.5 Other hydraulic analysis packages

There are several other software packages that can be used in the design of open channel and piped drainage systems, but are not as commonly used in the UK.

StormNET (Boss International/ US FEMA) can perform hydrological, hydraulic and quality simulations, but it is not clear whether sediment movement is included.

HY-22 (US FHWA) can perform basic design calculations but has no sediment transport modelling facilities.

HydroCAD is a useful tool for storage pond design in association with a piped network, but is otherwise not relevant.

Note: HEC-RAS (US Army Corps of Engineers) and ISIS/ InfoWorks RS (River Systems) (Wallingford Software) are river channel analysis tools and cannot deal with piped drainage.

3.2.6 Other linked software

It is also worth mentioning software that is used for drainage design, but does not include the full analytical capability.

Autodesk PDS-Drainage is one such. Its strength is in preparing drawings as an integrated part of the site layout, including highways, buildings and other services. It has dynamic links to Micro Drainage, but has only basic computational functionality.

CivilCAD (Sivan Design Ltd.) is an aid to drainage layout, but does not have hydraulic analysis capabilities.

Bentley PowerCivil (Bentley Systems Inc.), used in association with MicroStation CAD software, is another aid to drainage design, but it is not clear what hydraulic design capability it has.

3.2.7 Summary

There is no known software available specifically to predict the generation of sediment on highway surfaces.

Explicit modelling of movement of sediment in drainage systems is a specialist application, and does not feature in the majority of software commonly used for highway drainage design. It requires additional software costs to the standard hydraulic analysis licence, and particular skills and experience in the users. However, the design software generally available uses velocities as a surrogate, so the issue can usually be simplified to defining suitable target minimum values to ensure that self-cleansing conditions are regularly achieved. This should be adequate for most cases. Where an existing problem has to be analysed and solutions found, it may be necessary to move to explicit modelling, in which case specialist advice should be sought.

This project could seek to simplify the process of identifying cases where such specialist advice is needed, as well as design rules that are readily applicable in the remaining cases. Alternatively, and if it were considered useful, it would be feasible to develop an ad-hoc tool to model deposition/ erosion in a single pipe, which could be made available to users of DMRB.

4. Pipe Roughness

The roughness of a pipe is the texture of the internal surface texture. This is the smoothness of the pipe barrel and the protrusion of surface features in to the pipe flow. This can be expressed in terms of Manning's n number or the coefficient of surface roughness k_S expressed in millimetres.

Surface roughness k_s values are based on a comparison of surface roughness to sand grain sizes, as a uniform surface coat and are used with the Colebrook-White equation to determine head losses due to friction at the walls of pipes. In engineering applications the value of k_s is often chosen by judgement of the pipe condition (e.g. good or poor) and takes account not only of surface roughness but also of variations in roughness within the same material and the effect of pipe joints.

4.1 Effects of ageing on pipe roughness

It has been established that, although ageing normally increases the hydraulic roughness of pipes, this effect is significantly more important in water mains (where formation of deposits can occur with time) than in drains, particularly foul drains. As pointed out in Escarameia and May[8](1995), the case of surface water drains is less clear because these pipes flow part-full most of the time, can have large sediment deposits and carry water with varying levels of pollution.

Although the applicability of conclusions for water carrying pipes to highway drainage pipes has not been fully established, ageing will normally lead to a reduction of carrying capacity through: 1) the formation of wall deposits, 2) deformation of the pipe cross-section or 3) deterioration of the internal pipe surface (or a combination of all). Of these factors, the first is considered to have the largest impact on pipe hydraulic capacity. Colebrook and White[9] (1937) showed that the loss of carrying capacity of pipes due to formation of wall deposits can, without appreciable error, be entirely attributed to an increase in surface roughness rather than to a reduction in cross-sectional area. For irregularities smaller than 2mm and pipes of 500mm diameter, these authors showed that assuming the irregularities increased the hydraulic roughness, a loss of capacity of 20-30% would result; if the irregularities were treated as a uniform reduction in cross-section, the loss in capacity would have been less than 3%. Several authors have investigated the effects of ageing in water conveying pipes (see summary in Escarameia and May[8], 1995) but where surface water pipes are concerned the detrimental effects of ageing are generally overtaken by the potential for sediment deposition. However, if the design of pipes minimises the risk of sediment deposition, as is the aim of the Highways Agency with the present project, maybe some allowance for pipe ageing should be made. It is suggested that when using values from the HR Tables for the hydraulic design of pipes and sewers[5] values of pipe roughness associated with "poor" condition should be selected to account for ageing.

A deformation of the pipe cross-section, unless extreme, will still be associated with the original cross-sectional area and therefore the overall design capacity (which assumes pipe full conditions) will in principle not be appreciably affected. For a detailed design incorporating part-full conditions, changes in cross-sectional shape can be relevant but this is unlikely to deserve consideration for the design of highway drainage.

Deterioration of the internal walls of pipes with time can occur due to erosion of the pipe or lining material. However, with improvements in material quality it is considered that the likelihood of this factor being relevant for the design of surface drainage pipes, where flow velocities are small, is low.

4.2 Effects of pipe corrugation on roughness

Corrugated pipes can be found principally in road culverts where they come in large diameters or, when perforated and in small diameters, in road-edge filter drains and land drainage applications. Typical materials are plastic and metal but it should be noted that the hydraulic roughness of corrugated pipes is very specific to the geometry of the actual corrugations (and any projecting boltheads and overlapping joints). Specific advice should be sought from the relevant manufacturers.

4.3 Effects of pipe misalignment and joint eccentricity

Modern jointing methods allow accurate alignment of pipes but misalignments and joint eccentricity can and do still occur mainly due to inadequate control during the installation phase, ground settlement or simply pipe ageing. The main detrimental effect of pipe section misalignment on hydraulic performance is the generation of steps at joints, as well as an overall change in flow direction and potential for intrusion of roots, sediment, etc, into the pipe. Head losses can also be generated by differences in the internal diameter of pipe sections which occur during the manufacturing stage but these can reasonably be “absorbed” by the general friction coefficient attributed to the pipes.

Henderson[10] (1984) produced a guide for estimating the effect of joint displacement in pipes flowing full. A table was produced providing the roughness values, k_d, to add to the value of k_s, of the pipe; this was based on measurements on sewer pipes with diameters between 150 and 450mm and length between 600 and 900mm. The table is reproduced below (note that joint steps of 20mm and above are considered as structural damage):

Mean joint step (mm)	k_d due to misalignment (mm)
2-5	0.06
5-9	0.15
10-13	0.30
14-18	0.60
19-27	1.5
28-40	3.0
41-55	6.0
>55	15.0

5. Sediment transport data

The major objective of this project is to bring the research into sediment loads in pipelines to the public realm.

5.1 Sediment Loads

There is limited information available on the amounts of sediment present in highway environments. The following documents provide much of the current data:

Sediment loads from high speed roads[11]

Historically, a number of small scale studies had been undertaken investigating the amount of sediment generated from carriageways in Britain. The Highways Agency did not consider these studies to be comprehensive or representative, particularly as these studies have predominantly involved collection of sediments from highways in urban locations, whereas the majority of the Highways Agency network is located in rural environments.

Between June 2005 and May 2006 Atkins undertook a field study for the Highways Agency collecting sediment data from 15 sites in the motorway and trunk road network in England. All but two of the sites were located in rural locations with varying land uses, number of lanes, cross sectional profiles and geographical locations.

Samples were collected at two month intervals from gully pots fitted with mesh filter baskets with an aperture size of 150µm. This size was selected as particles of this size and smaller are likely to travel in suspension and hence not affect the hydraulic performance of pipe networks. A 1m strip of the upstream contributing area was also wet vacuumed to collect the siltation that could potentially reach the gully during subsequent rainfall events, and had reached the kerb from the carriageway or surrounding land. These samples were subjected to a variety of physical tests to determine wet and dry weights and volumes, wet and dry bulk densities, silt-clay fraction, particle size distribution and organic content. Concentrations were calculated based on daily averaged rainfall data provided by the Met Office from ten nearby weather stations.

D_{50} results (50% of the accumulated weight percentage is smaller than this particle size) were determined for each site based on the particle size distribution curves derived from the study and averages for each category calculated and compared to data collected from other studies primarily undertaken in urban locations. Data indicated that sediments in urban areas were comparable to other studies and that rural areas were more varied than those in urban locations. Such variations may be due to surrounding geological conditions.

Land Use	Rural General	Urban	Rural Grassland	Rural Arable	Rural Forested	Rural Dense Woodland
Sediment Load kg/ha/year	1350	1790	1200	1100	710	No result
D_{50} Results (mm)	0.89 (coarse sand)	0.48 (medium sand)	2.15 (Gravel)	1.59 (V. Coarse Sand)	0.60 (Coarse Sand)	0.55 (Coarse Sand)

Table 5.1: Representative D_{50} particle at the monitored locations

A final report was issued to the Highways Agency "Monitoring sediment loads from high speed roads"[11], the main conclusions of which identified that the most significant factor affecting sediment arising from highways are operational activities, particularly construction works and proximity to commercial activities such as quarrying or farming. Such activities can be transitory, seasonal or permanent in their existence. In terms of what could be considered as normal circumstances a number of factors were investigated and surrounding land use was considered to have the greatest effect on sediment loads generated. Particularly, highways in urban locations are likely to accumulate approximately twice the sediment of those highways in rural locations. Non-operational factors considered to affect the sediment loads of high speed roads were considered in order of significance as follows:

- Surrounding land use (most significant)
- Geographical location (geological nature)
- Cross sectional profile
- Number of lanes (least significant)

It was concluded that the geographical location has an impact on the sediment load, second to the surrounding land use. It is considered that the geological conditions of an area can affect the amount of sediments that are present at a particular site. While the sample sites were selected across England with the intent of providing a reasonable variation in geographical spread, the sites selected only represented a limited variety of geological conditions. This could be simplified by considering the WRAP class (Winter Rainfall Acceptance Parameter as defined within the Flood Studies Report) of each site and could provide a map based approach to sediment bed load prediction for design purposes. In considering this approach it is recognised that the sites selected are limited as no sites were selected in areas of WRAP classes 3 or 5. The WRAP values for any area can be obtained from the revised soil map or from the 1:1000000 version covering the whole of the UK included in the Wallingford Procedure. The index broadly describes infiltration potential and was derived by a consideration of soil permeability, topographic slope, and the likelihood of impermeable layers. Five classes of soil are recognised as shown in the table below taken from the Wallingford Procedure (Table 4.3).

Soil Class	WRAP	Runoff	SOIL
1	Very high	Very low	0.15
2	High	Low	0.30
3	Moderate	Moderate	0.40
4	Low	High	0.45
5	Very low	Very high	0.50

Different classes of soil

The report recommended developing tools to determine sediment loads for highways drainage design. A simple sediment load prediction tool was developed for use in estimating non-operational sediment loads for any site in England, based on the information gathered during the study. The prediction tool uses factors, determined empirically from the data collected in the sediment loads study, and weights their effect on the base load in accordance with the considered significance of the factor (e.g. surrounding land use has the greatest weighting). A factored bed load is produced from this, which is the weight of sediment per m^2 generated from a carriageway. This weight can then be multiplied by an individual contributing area to determine a weight of sediment arriving at a particular gully on a drainage system.

The study did not consider the transport of sediments within a drainage system, although one catchpit was included in the study as a collection point. This site did not demonstrate any significant departure from the main study conclusions. Both kerbed and channel surface conveyance systems were included in the study.

Other sediment load data has been collated in the following studies:

Guidelines for The Environmental Management of Highways (IHT) [12], 2001

This document provides data on total solids arriving at receiving water courses from European studies based on traffic flows:

Vehicles per day (ADT)	Total solids kg/ha/year
<5000	2218 – 3640
5000 - 15000	479 – 7289
15000 - 50000	848 – 873
>50000	1930 – 10410

The traffic volumes included refer to the different types of highway including residential roads (<5000 vehicles/day), urban roads (5000 – 15000 vehicles/day), rural motorways (15000 – 50000 vehicles/day) and all types of motorway (>50000 vehicles/day). The results of the Atkins study would fall within the 15000 – 50000 range for rural highways with expected values within the range of 848 to 873. The measured loads from the Atkins study gave an average figure of 1090 kg/ha/year, which is greater than the predicted. A value of 1790 for urban highways derived from the Atkins study is less than that predicted in the IHT study. However, it is apparent that the order of magnitude for each environment is comparable.

CIRIA 142 Control of Pollution from Highway Drainage Discharges[13], 1994

CIRIA 142 investigates the control of pollution from highways and the various interception methods. It includes data on quantities of pollutants again based on traffic flows (table 5.1):

Vehicles per day (ADT)	Total solids kg/ha/year
<5000	2500
5000 - 15000	5000
15000 - 30000	7000
>30000	10000

These values are stated as being overestimates from previous studies and could explain why the Atkins results are considerably less in the case of both rural sites (15000 – 30000) and urban sites (>30000).

CIRIA 134 Sediment Management in Urban Drainage Catchments[14], 1995

This document provides various data relating to sediment loads and characteristics including:

Classification and physical characteristics of sewer sediments (adapted from Crabtree, 1989)[15].

Type	Description & Location	Particle size (%*)			Sat. bulk density kg/m³
		50-2 mm	2-0.063 mm	<0.063 mm	
A	Coarse, loose, granular material found in pipe inverts	33	61	6	1720
B	As A but concreted by the addition of fat, bitumen, cement etc into a solid mass	N/A	N/A	N/A	N/A
C	Mobile, fine grained deposits found in slack flow zones, either in isolation or above Type A material	0	55	45	1170
D	Organic pipe wall slimes around the mean flow level	6	62	32	1210
E	Fine-grained mineral and organic deposits found in SSO storage tanks	9	69	22	1460

* Dryweight of ashed residue

The report also provides data on "Ultimate" sediment load on trial sites, which exist under the rainfall / cleaning regime that is adopted in a particular area and may be generally stated as within a range of 250-300 g/m kerb (Sartor and Gaboury, 1984)[16]. The "Ultimate" state refers to the long term equilibrium brought about by two opposing effects of sediment supply and sediment removal (rainfall and sweeping).

Useful data on sediment build up in gully pots is provided and recommendations on cleaning frequencies including for rural areas (table 6.3). The report provides guidance on efficiencies of gully pots, varying with flow rates (figures 4.9 and 4.10). The report recommends the development of guidance in the design of gully pots as none existed then.

5.2 Transport in pipes

Research and design guides providing information on the transport of sediment in pipes include:

CIRIA Report 141 Design of sewers to control sediment problems[1]

This publication is the most comprehensive document available giving design guidance on how to determine and minimise the effects of sediment in pipes. As well as presenting theoretical background and sediment transport formulae, figures are given showing minimum flow velocities for the transport of sediment in suspension and as deposited bed. Figures for transport properties of cohesive sediment are also given. With the aim of providing a simplified design aid for circular foul and surface water pipes in UK conditions, design tables for pipe diameters up to 5000mm are also included in this document.

Sediment transport studies have shown that pipes flowing part-full (but above 1/3 full) have a higher transport capacity than full-bore. The design of self-cleansing pipes therefore usually assumes full-bore conditions since these provide a conservative approach. The guidance recommends that the "sewer" is designed to discharge freely, i.e. designed on a part full basis with no backing up because a "sewer" flowing full and backing up has a tendency to deposit sediment.

The design tables are based on two roughness coefficients (k_s) which are those specified in Sewers for Adoption: foul and combined k_s = 1.5mm, surface water k_s = 0.6mm.

The procedures refer the mobility of two distinct sediment particle sizes: fine particles, i.e. those smaller than 150µm and transported in suspension, and coarse particles, i.e. those larger than 300µm and transported as bedload. In the first instance, the procedures recommend that, for the sediment transportation calculations, site specific sediment characteristics are determined from sampling. In the absence of this information the procedures categorise two concentrations of these sediments as "medium" and "high" and consequently would be inappropriate for either low or very high sediment loads. Loads within highway drainage pipes would need to be categorised in accordance with the CIRIA simplified procedure; Table B1 is reproduced below, where X is the concentration and SG the specific gravity of the sediment:

Table B.1: Sediment categories for simplified procedures

Sediment transport mode	Type of sewer applicable	Parameter	Category	
			Medium	High
Suspended	Foul	X (mg/l)	350	1000
	Surface water	d_{50} (µm)	60	100
	Combined	SG	2.0	2.5
Bedload	Surface water	X (mg/l)	50	200
	Combined	d_{50} (µm)	750	750
		SG	2.6	2.6

The Report recommends that “high” sediment categories are used for surface water and combined sewers where sediment loads are “expected to be higher than usual” and give examples of where these may be:

- Existing or expected high rate of development or redevelopment in the catchment
- Presence of wind-blown sand, e.g. coastal situations
- Predominance of poorly maintained roads.

In other situations the “medium” category should be used for suspended sediment and bedload for surface water sewers.

Three sediment mobility criteria are identified:

i) transport of fine grained particles in suspension

ii) transport of coarser granular material as bedload

iii) ability to erode particles, that may have some cohesive strength, from the pipe invert or from a deposited granular bed.

The first two criteria make use of the above sediment characteristics and loads; the third is dependent on minimum velocity but independent of pipe roughness. To meet the second criterion, the designer must decide either to allow a certain depth of deposition or to ensure that the full bore flow should be capable of transporting all the sediment. The Report recommends that an allowance be made for a 2% depth of sediment in the pipe invert, with a limit of deposition (LOD) condition only where steeper gradients are achievable.

The criteria for cohesive particle erosion are: bed grain shear stress equal to 2 N/m^2, assuming 1mm cohesive particles, and an associated bed roughness of 1.2mm.

The tables relevant to surface water sewers or highway drains are Table B.6, B.7, B.8 and B.9 depending on the Sediment Category (M or H) and the Mobility (2% or LOD). These Tables are reproduced in Appendix A for the range of pipe diameters likely to be used for highway drainage.

HR Wallingford Report SR604 - Sumpless gullies for highway drainage[17], 2003

This report describes work carried out on a project commissioned by the Highways Agency to assess the viability of using sumpless gullies as an alternative to conventional gully pots. By not retaining the sediment, sumpless gullies are associated with higher levels of sediment in the discharge pipework and this aspect was investigated first by means of hydraulic calculations and then by laboratory experiments. Hydraulic tests were carried out to investigate the performance of two different configurations of sumpless gully and the conditions for sediment deposition in the downstream pipe. The work was carried out using a full scale test facility containing a typical gully grating 450mm by 450mm with diagonal bars, a sumpless gully and a 5.6m long outfall pipe with 150mm internal diameter. A clayware bend formed by two 45^o elbow bends with Perspex observation windows made the connection between the sumpless gully and the outfall pipe. In the tests granular sediment, cohesive sediment and coarse debris were used but the analysis focused on the granular sediment as this has been found by previous researchers to be most representative of sediment in highway drains.

At the time of this study, specific information on the characteristics of the sediment (such as d_{50} size, specific gravity and typical concentrations) that accumulates along high-speed roads was not available; it was later provided by the monitoring study described in the Atkins report of 2006[11]. The hydraulic assessment was therefore carried using formulae and data provided in CIRIA R141[1]. The types of sediment used in the tests were: granular sediment with d_{50} = 1.075mm, cohesive road sediment collected along the A4130, Oxfordshire, as well as debris (e.g. leaves, twigs, plastic bags and cups). The tests covered the following conditions: flow rates ranging from just over 2 l/s to 20 l/s (for a typical 200m^2 catchment size this is equivalent to rainfall intensities from 50mm/hr to 350mm/hr); different volumetric sediment concentrations (60ppm to 1000ppm) and different pipe slopes. The pipe diameter was kept constant at 150mm as it represented the upstream part of a sumpless gully system. One of the objectives of the tests was to find the minimum pipe slope for which the sediment would not tend to deposit in the pipe, but would instead be transported and discharged at an outfall chamber.

In Appendix B, Tables B.1 and B.2 summarise the test results obtained for granular sediment on the two types of sumpless gully. As can be seen from the tables, for the flow conditions tested and the chosen testing set-up, the pipe was not flowing full. The tests confirmed that the sediment will tend to deposit in the pipe system unless a sufficient transport capacity is achieved by increasing the gradient (assuming that the pipe diameter remains constant). At gradients of 1/100 little deposition was found to occur in the 150mm pipe and only at pipe joints, except for the higher sediment concentrations tested (1000ppm). At a slope of 1/60 no deposition was observed for concentrations as high as 1000ppm. However, these gradients are unlikely to be economically achievable in the majority of cases.

The study also comprised tests to investigate the behaviour of deposited cohesive sediment. Road sediment collected along the A4130, Oxfordshire, was used and tested as a deposit occupying 18% of the pipe diameter at 1/100 gradient. It was found that average shear stresses of 2-3 N/m^2 were needed to erode the bed, assuming that no other sediment was being discharged into the pipe system.

Since the experimental work did not cover pipe diameters other than 150mm (which is typical only of the upstream part of a conventional road drainage system), an assessment of the transport capacity of the whole drainage system was also conducted as part of this study. As the drainage system receives flow from further gullies, the pipe diameters normally increase in the downstream direction. The assessment was carried out using numerical programs previously developed by HR Wallingford for sediment transport in pipes with deposited beds. The equation due to May 1994 was used. Verification analysis showed a very good agreement (within –2.5% and +7%) between measured and simulated pipe gradients. Some simulations were then carried out for bigger pipe diameters, up to 450mm. These showed that, for the same flow and sediment conditions, the predicted slopes were generally flatter the greater the pipe diameter. Exceptions to this were conditions where there are very small or very high relative water depths in the pipe. It was inferred from the above that achieving the transport capacity observed in the 150mm diameter pipe becomes easier as the pipe diameter is increased and meant that conclusions drawn from the150mm diameter pipe could safely be extended to larger diameters.

HA105 Sumpless gullies[2]

The use of sumpless gullies in place of conventional sumped pot gullies has the potential to introduce significantly greater volumes of sediment into the piped drainage system. Following experimental research into the movement of sediment in pipelines[17], an Advice Note HA105[2], was published to give guidance on their use.

The Advice Note states that different drainage systems may be designed for sediment retention such as the use of catchpits which will reduce the volume of sediment in the flow immediately downstream; consequently, the required minimum flow velocity is reduced. However, the incorporation of catchpits and the associated reduction in flow energy may also need to be taken into account in the hydraulic design.

Appendix B of HA105 contains tables of minimum pipe full flow velocities for self cleansing pipes in systems with sumpless gullies. The tables enable the design of the pipe system for various types of gully and location long the drainage system and are reproduced here in

Appendix C. The tables are based on an overall pipe roughness (k_s) of 0.6mm and a maximum depth of sediment deposit equivalent to 1% of the pipe diameter.

Sediment deposition is a function of the sediment load, pipe diameter, flow velocity and, consequently, pipe gradient. The trials undertaken[17] showed that only minor deposition is likely to occur if the downstream pipes are steeper than 1 in 100; this is repeated in section 5.5 of the AN. Steepening the pipe gradient is the more common method of improving the sediment transport capacity of a pipeline. Clause 5.8 contains the recommendation that for effective allowance of sediment effects in the pipeline, then an overall roughness value (ks) of 3mm should be inserted into the Colebrook-White equation.

Solids in Sewers[18], 2004

This publication, which is a compilation of selected articles presented at an international conference held in 1995, is a state-of-the-art information on the origins, occurrence, nature and effects of sewer solids, including organic and non-organic solids, for sewer design and operation. It describes sediment transport as a continuous complex process of particles interchanging between suspension, the bed and near-bed region. This document includes a graph due to May (1994), which is a good illustration of the various sediment transport regimes in circular pipes, but offers little additional information on inert sediment effects in pipes when compared with the CIRIA R141 report.

5.3 Local losses

Local losses that occur in highway drainage systems will typically be associated with the presence of bends and junctions in pipes, entry to and exit from manholes/chambers/gully pots and exit at outfalls. These losses are approximately proportional to the square of the flow velocity, V, and it is common practice to assign to them a non-dimensional loss coefficient K, which results from the division of the head loss by the velocity parameter $V^2/2g$ (kinematic head).

Research studies carried out to determine the K coefficients at local features within a drainage pipeline have not tried to incorporate the effect of sediment in the flow. This issue is even not usually discussed in papers concerned with sewer pipelines which, by their nature, may contain a higher level of suspended solids and gross solids than road drainage systems. The reasons for this can be considered both on a scientific basis and at a practical level. On a scientific level the justification is based on the assumption that the flow in drainage systems is fully turbulent for the design conditions and that therefore the effect of viscosity of the flow (which is dependent on the amount of suspended sediment) is not significant (the Reynolds number of the flow is large enough for viscosity to be neglected). If this assumption is acceptable for sewer systems, it is more so for highway drainage systems where, due to lower levels of suspended solids, the viscosity of the flow is hardly different from that of clear water. On a practical level, the study of local head losses in drainage systems requires the investigation of a large number of parameters: pipe diameter, geometry of the chambers, drop height, number of pipes discharging into and out of a chamber, angle of bends, type and material of bends, etc. Simulation of sediment effects is of secondary importance and is therefore usually captured in the estimation of friction losses in the straight sections of pipe. In some circumstances it may be useful to check the effect of, for example, the accumulation of sediment in a chamber as a result of neglected maintenance. This is normally treated as a reduction in the available volume at the chamber.

When compared with the published information on head losses at local features within a drainage network, it is apparent that information relating to the specific effect of sediment is not available. The text below will therefore elaborate on the local loss coefficients recommended for newly developed highway drainage systems such as the combined channel and pipe system.

HR Wallingford Report SR624 Combined surface channel and pipe system[19], 2005

As part of a wider study for the Highways Agency aimed at developing a new type of road drainage system (the “Combined surface channel and pipe system”) tests were conducted to determine local losses at chambers. The type of chambers tested complied with recommendations given in Advice Note HA78[2] (1996). Various forms of benching were also introduced inside the chambers in order to find the configuration that would minimise head losses. These chambers, rectangular in shape, received flow at high level from the surface water channel and at a medium level from the incoming pipe; the discharge was via a pipe at the same level as the incoming pipe. The diameter of the pipes in the tests was 125mm and, because the typical pipe size for the application under study was 300mm, the tests were considered to be carried out at a scale of 1 in 2.4. The tests investigated the effects of the following parameters on the local head losses at the chambers: relative water depth in the pipe, ratio of flow from the surface channel and that in the outflowing pipe, gradient of the channel/pipe system, geometry of the chamber, and geometry of the benching in the chamber. Although the flow from these systems can carry some sediment, particularly along the surface channels, the testing to determine head losses did not consider the effect of sediment, as is customary in this type of study.

For this study two different chamber configurations were considered:

- Off-line in relation to the pipe alignment, where the chamber was set towards the verge

- In-line with the pipe alignment.

However, the position of the chamber was found to have little influence on head losses, with the internal benching being more important. Guidance on head loss coefficients were given for the following configurations:

For chamber with benching extending to half pipe height and no flow from the surface channel (Benching type I – see Figure D.1 of Appendix D)

$$K = 1.04\,\frac{y}{D} - 0.476 \qquad \text{for } y/D \geq 0.5 \qquad \text{(Eq. 1)}$$

where y is the water depth in the pipe and D is the pipe diameter. This equation is valid for $y/D \geq 0.5$; for smaller relative water depths the losses in the chamber can be estimated satisfactorily using the Colebrook-White equation along the length of the chamber.

For chamber with benching sloping towards the pipe soffit (at gradients between 1:5 and 1:10 – Benching type II – see Figure D.1), and flow from the surface channel as well as from the pipe

$$K = 0.684\,\frac{Q_C}{Q_D} + 0.220 \qquad \text{for } 0.520 \leq y/D \leq 0.739 \qquad \text{(Eq. 2)}$$

and

$$K = 0.714\,\frac{Q_C}{Q_D} + 0.407 \qquad \text{for } 0.835 \leq y/D \leq 0.940 \qquad \text{(Eq. 3)}$$

where Q_C is the flow entering the chamber from the surface water channel, Q_D is the total downstream flow produced by the combination of the surface channel and upstream pipe flow, and y/D is the relative flow depth in the pipe downstream of the chamber.

For determining the head loss at a chamber the value of flow velocity, V, should be that of the pipe downstream of the chamber. The study recommended the use of Equation 1 for when there is no flow from the surface channel and Equations 2 and 3 if there is flow from the channel. In the absence of information for ratios of y/D <0.520 the value of K can be calculated using Equation 2. To illustrate the values provided by the above equations, when applied to pipe full conditions, they give the following K values:

Pipe discharging into rectangular chamber with benching extending to half pipe height

K= 0.56

Pipe and surface channel discharging with equal flow into rectangular chamber with benching extending to pipe soffit.

K= 0.76.

6. Conclusions and Recommendations

6.1 Sediment loads

For the development of previous HA documents, such as the HA105 Design of sumpless gullies, a detailed analysis was carried out to assess the effect that sediment (no longer trapped in gully pots) would have on the performance of the piped system. Because the sumpless gullies development work was carried out before the Sediment Loads monitoring programme, there was considerable uncertainty regarding the value of sediment loads to assume for the calculations. CIRIA R141 provided some information on surface sediment supply rates that could, by association, be used to estimate highway sediment loads. Using the "Industrial" category (which was thought to be comparable to well trafficked roads) gave 10g/m^2 road per day, of which 70% is generally accepted as corresponding to coarse material. This thus gave a value of 7g/m^2 road per day, or 2,555g/m^2 per annum.

The results of the Sediment Loads study showed that urban roads, which have been found to create more sediment than rural roads, generate maximum values of the order of 200g/m^2 per annum, about 90g/m^2 per annum of which is retained in gully pots. These values are one order of magnitude lower than those given by other studies and reported in CIRIA R141. This finding indicates that previous recommendations may err significantly on the conservative side and is an important conclusion for design.

With the sediment loads obtained from the monitoring programme, it has become possible to determine typical sediment concentrations in road drainage pipework. Consider 90g/m^2 per annum for urban trunk roads and 40g/m^2 per annum for rural trunk roads which are typical values for sediment retained in gully pots. Assuming a "typical" three lane motorway and one gully per 200m^2 of road surface, an urban gully pot would receive 18kg/annum and a rural gully pot 8kg/annum. Taking the UK average rainfall of 900mm/annum and using the average particle density measured in gully pots during the Sediment Loads programme (2220kg/m^3) gives sediment concentrations of 45ppm (urban roads) and 20ppm (rural roads). The value obtained for rural roads corresponds approximately to the "medium concentration" recommended in CIRIA for sewer design, i.e. 50mg/l, whereas for urban roads it is approximately 115mg/l, a value that is still well below the "high concentration" value given in CIRIA (200mg/l). This indicates that sediment concentration levels at highways in the UK are comparable with the average values associated with surface water sewers.

It is concluded that prior to the Sediment Loads study there was limited information on the sediment load likely to be present in urban highway locations and none for rural locations. The data gathered from the Sediment Loads study may be used to develop a prediction tool for design sites for use with the design guide set out in CIRIA R141 or similar.

The following suggestions would allow full uptake of the information from the Sediment Loads Study:

- The use of data from Sediment Loads study for determining more accurate sediment characteristics for design calculations using CIRIA R141, rather than reliance on “medium” or “high” loadings for planning of maintenance operations.

- Assessment of impacts on design of operational factors or mitigation for managing such occurrences.

- Development of a design guide for maintenance purposes based on geological conditions, rather than surrounding land-use which may change during the life of the road.

6.2 Modelling of sediment transport in piped systems

Modelling of sediment transport in drainage systems is a specialist activity not usually undertaken for standard projects, and does not feature in the majority of software commonly used for highway drainage design. The presence of sediment and the need to ensure movement are indirectly taken into account by defining target minimum values of flow velocity to ensure that self-cleansing conditions are regularly achieved. It is considered that this simplified approach, common in sewer modelling, should be valid also for highway drainage design in most cases. In specific cases where sediment is perceived as being of particular concern (e.g. analysis of drainage systems with persistent blockage problems attributed to unusual rates of sediment ingress, or analysis of conditions during construction), explicit sediment transport modelling should be carried out and specialist advice should be sought.

The methodology recommended can be summarised as follows:

- Develop decision-support guidance for identifying cases where specialist sediment transport analysis is needed; if it were considered useful, develop an ad-hoc tool to model deposition/erosion in a single pipe, which could be made available to users of DMRB.

- Identify typical sediment load for the particular site

- Determine suitable friction factors that take into account level of sediment ingress into piped systems, pipe size and material

- Define minimum flow velocities associated with the values of the friction factors

- Produce tables of friction factor, pipe gradient, minimum flow velocities (for pipe full conditions).

6.3 Friction coefficients

From results of the Sediment Loads study, the levels of sediment ingress into highway drainage systems appear not to be more severe than those associated with storm water sewers. Therefore it appears reasonable to suggest that the procedure given in CIRIA R141 for calculating composite roughness values, k_c, should also be considered for highway design. The composite roughness value takes account of the pipe wall roughness and the bed roughness, and requires estimation of the depth of the deposited bed. For the typical size of sediment present in highway drainage pipes, most of the sediment transport will be in the form of bed load. As the design procedure should have an element of generality (as well as consideration of site location, type of road, pipe size and material, etc) it is suggested to assume a deposited bed depth corresponding to 2% of the pipe diameter, which is proposed in CIRIA R141 for simplified design.

6.4 Local loss coefficients

Given that highways do not appear to generate more sediment than surface water sewer networks, as shown above, there is no reasonable justification for considering the effect of sediment in local head losses, namely at manholes. Existing information, which does not account for the effect of sediment, is therefore satisfactory in this regard.

Various sources provide values of the coefficient of head loss, K, at bends, junctions, manholes, etc, and these are normally incorporated in commercial design software. The default values given can usually be changed by the user and it is suggested here to check compatibility with the values recommended in BS EN 752 or use values determined for the specific case/situation if these are available.

However, adequate account is currently not made of losses caused by multiple connections at manholes and catchpits, as loss coefficients only relate to single entry and exit straight-through manholes and manholes with bends. Experimental research into losses at catchpits carried out as part of the development of the combined surface channel and pipe system suggested K values of the order of 0.76 for flow discharged from both the surface channel and the pipe. A very comprehensive source of information on head losses is the book by Idelchik[20] but guidance on manholes receiving multiple entry pipes is not specifically covered. Approximate values are sometimes obtained from data for four-way wye pieces which are provided for angles of approach of the side pipes of 15°, 30°, 45°, 60° and 90°. However these values do not take account of the enlargement at the catchpit or at a non-benched manhole. Also, all the configurations covered in Idelchik assume that the incoming pipe is of the same size as the exit pipe and that they are aligned. For a particular configuration (i.e. angle of approach of the side pipes), the head loss coefficients will depend on the ratios of flow rate in the various pipes and the ratios of their cross-sectional areas but, as an indication, values of K as high as 3.4 are proposed. Experimental research into losses at catchpits carried out as part of the development of the combined surface channel and pipe system suggested K values as high as 0.76 for flow discharged from both the surface channel and the pipe.

It can be concluded from the literature review that specific guidance on head losses at manholes and catchpits with multiple entries is not available and experimental research would therefore be useful for increasing the accuracy of the design calculations.

6.5 Hydraulic roughness of new types of surfacing

The Advice Notes HA102: Spacing of road gullies and HA37: Hydraulic design of road edge surface water channels both give guidance on the Manning's n value to be used to assess surface flows, but only contain values for concrete and black top in average or poor condition. There are new types of surfacing being used on the network, such as Stone Mastic Asphalt, Thin Surface Courses etc, that may have different n values. Maybe this guidance needs to be revised to take account of these alternative surfacings. Additional values may be determined in Stage 2 and text produced.

6.6 Load predictor as a maintenance tool

The sediment load prediction spreadsheet developed under the monitoring project may be more appropriate as a maintenance tool than for design purposes. Primarily, this is because during the life of the road, land use can change significantly often driven by market forces or subsidies applied in the agricultural industry.

Land use maps are available; these generally indicate that south east of a line from The Wash to the Severn Estuary, the land use is predominantly arable. A map of land use may be more appropriate for determining sediment loads than the Maintenance Area map used for the Sediment Loads Predictor.

An indicative geographical comparison of predicted loads is shown below, based on the assumptions that the landuse (grassland), road size (3 lane motorway) and profile (embankment) remain constant and only the geographical location (areas 3 and 10) vary. Area 3 being Wiltshire/Hampshire and Area 10 being Cheshire/ Greater Manchester.

Primary Bed Load $B_p = 0.05$ kg/m^2

Total Site Specific Bed Load $B_s = B_p(F_l + F_g + F_r + F_p)$ (Eq.4)

Where $F_l = 0.5$ x land use factor (grassland = 0.9)

$F_g = 0.35$ x geographical factor (Area 3 = 0.1, Area 10 = 6.2)

$F_r = 0.1$ x road size (3 lane = 0.1)

$F_p = 0.05$ x profile (Embankment = 1.2)

Inserting the factors into the above equation the predicted sediment loads are:

Area 3 32 g/m^2

Area 10 139 g/m^2

7. Stage 2 Document development

There is considered to be insufficient data in the following areas and this will be addressed in Stage 2 of the project.

The guidance on design of pipelines for sediment transportation given in CIRIA R141 requires that the flow be parallel to the pipe invert and that backing up does not occur due to downstream constrictions, tidal flows etc. Highway drainage outfalls can be surcharged by flows within the receiving watercourse/ditchcourse and hence guidance will be required on overcoming this effect.

The data collected in the Sediment Loads study can be used for more accurate determination of the likely effect of sediment in highway drainage. A review of the bedload predictor is recommended and consideration for the augmentation of raw data in a national database to be developed on sediment characteristics for geographical locations including surrounding land use data. The data set collected by the Sediment Loads study is limited in its extent considering the number of variables that can affect the sediment bed load. Consideration should be given for further data gathering from site sample collection for design sites in future with a data storage repository made available for sediment information gathered from sites.

Stage 2 of the project will consider how sediment load, land use and k_s value are to be incorporated into the design and the maintenance of piped systems.

With regard to the design of pipes two alternative approaches can be considered:

Approach 1 - by prediction of sediment load

This approach is based on estimating the total sediment load for a specific site and the proportion entering the gully and then using formulae given in CIRIA R141. The methodology in CIRIA R141 is however rather complex and cannot easily be simplified for incorporation in an Advice Note if the sediment loads are not averaged values but specific to each site. It is considered unrealistic to expect that a highway drainage engineer will embark on such an exercise. CIRIA R141 provides a simplified method which is based on average sediment loads (low, medium and high) but this negates the use of detailed sediment rates.

Approach 2 - by adjusting the roughness coefficient, k_s.

This approach is based on determining composite values of the roughness coefficient, k_c, that take into account the effect of sediments associated to a certain type of location, as given by the Sediment Loads study. The roughness coefficients selected for design can allow for the effects of a loss of 2% of pipe diameter due to sediment deposition by using the methodology given in CIRIA R141 for calculation of the composite roughness. The roughness coefficient used for the design of new build assumes that the pipe remains in "good" condition and this may not be appropriate for use in assessing the capacity of existing piped systems to determine the capacity to accommodate future works. Stage 2 will determine whether a separate value can be included in a table of k_s values. Stage 2 development will confirm the k_s values to be included in any design tables that will be part of the design document. The tables could also include k_s values for misaligned pipes for use in the assessment of existing pipe performance. The tables would quote data (i.e. required gradients, flow velocity) for approximately 19 diameters in the range 150 to 900mm. The need for inclusion of data for pipes of greater diameter will be determined during Stage 2.

Given that sediment loads from high-speed roads are expected to be well within the range of values provided for the design of sewers, it is suggested that the prediction of sediment load may be more appropriate as a maintenance tool than incorporated into the document as a design factor. The sediment loads predictor developed during the sediment loads study will be revisited to adapt this as a maintenance tool - see previous chapter.

The frequency of drain cleaning is dependent on the rate of sediment accumulation in the drainage pipes. The sediment loads study showed that sediment load and the particle size vary on a geographical basis, therefore the map showing the geographical variations that will be produced during Stage 2 of this project can be used with the Sediment Loads Predictor developed during the sediment loads study.

The Advice Note while concentrating on the design aspects will include a section giving guidance on maintenance issues and could include or refer to the Sediment Loads Predictor.

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APPENDICES

Appendix A: Simplified design tables from CIRIA R141

CIRIA R141 Design Tables

Table B.6: Design table for surface water sewers with medium sediment loading and 2% allowable sediment bed depth

Sewer diameter D(mm)	Minimum velocity V_m (m/s)	Governing criteria	Deposited bed depth Y_s/D (%)	Composite roughness K_c (mm)	Discharge capacity Q (l/s)	Minimum gradient i_m (1/xxx)
150	0.67	iii)	0	0.6	11.8	217
225	0.72	iii)	0	0.6	28.6	317
300	0.75	iii)	1	1.9	52.9	313
450	0.79	iii)	1	2.1	126	470
600	0.85	iii)	1	2.1	231	630
750	0.85	iii)	1	2.2	375	780
900	0.87	iii)	1	2.3	552	930

Table B.7: Design table for surface water sewers with medium sediment loading at limit of deposition

Sewer diameter D (mm)	Minimum velocity V_m (m/s)	Governing criteria	Discharge capacity Q (l/s)	Minimum gradient i_m (1/xxx)
150	0.67	iii)	11.8	217
225	0.72	ii) / iii)	28.6	317
300	0.86	ii)	60.8	322
450	1.11	ii)	177	325
600	1.33	ii)	376	325
750	1.53	ii)	676	325
900	1.72	ii)	1090	322

Table B.8: Design table for surface water sewers with high sediment loading and 2% allowable sediment bed depth

Sewer diameter D(mm)	Minimum velocity V_m (m/s)	Governing criteria	Deposited bed depth Y_s/D (%)	Composite roughness K_c (mm)	Discharge capacity Q (l/s)	Minimum gradient i_m (1/xxx)
150	0.67	iii)	1	1.8	11.8	161
225	0.72	iii)	1	1.9	28.6	235
300	0.75	iii)	1	1.9	52.9	313
450	0.79	ii) / iii)	2	3.0	125	423
600	0.90	ii)	2	3.1	253	467
750	1.06	ii)	2	3.2	466	445
900	1.22	ii)	2	3.4	774	421

Table B.9: Design table for surface water sewers with high sediment loading at limit of deposition

Sewer diameter D (mm)	Minimum velocity V_m (m/s)	Governing criteria	Discharge capacity Q (l/s)	Minimum gradient i_m (1/xxx)
150	0.77	ii)	13.6	165
225	0.99	ii)	39.4	169
300	1.20	ii)	85.0	167
450	1.56	ii)	248	165
600	1.88	ii)	532	164

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Appendix B: Test results for sumpless gullies (from HR Wallingford Report SR604)

Table B.1 Test results with chute gully; granular sediment

Test no.	Slope	Rainfall intensity (nominal) mm/h	Flow rate l/s	Conc. ppm	Average water depth in the pipe mm	Average sediment depth in the pipe mm	Sediment weight in the pipe g	Observations
1 T2A	1/200	50	2.78	61	45	2	524	Test duration: 32min Sediment deposition more than 5m d/s of bend
1 T2B	1/200	100	6.50	60	Not measured	0	84	Test duration: 32min Sediment deposition more than 5m d/s of bend
1 T2C	1/200	50	2.75	307	40	Not Measured	1683	Test duration: 33min Dune (0.22m long) formed 0.5m from downstream end of transparent pipe
1 T2D	1/200	100	5.51	306	86	9	5145	Test duration: 1h 31min Deposited bed
1 T2E	1/200	150	8.43	299	101	4	Not measured	Test duration: 31min Deposited bed
1 T2F	1/200	300	16.7	500	141	0.7	Not measured	Test duration: 14min Very turbulent conditions in the gully (water level=0.48m above invert level of outlet pipe) Deposited bed
1 T2G	1/200	50	2.69	605	59	11	6620	Test duration: 30min Deposited bed starting 0.5m d/s of bend (continuous narrow strip)
1 T2H	1/200	100	5.36	606	87	17	9451	Test duration: 30min Deposited bed (continuous narrow strip)
1 T2I	1/200	100	5.56	1001	92	18	Not measured	Test duration: 19min Deposited bed, also in bend
1 T2J	1/200	150	8.33	1001	113	30	Not measured	Test duration: 23min Test started with deposited bed from Test 1T2I

Table B.1 Test results with chute gully; granular sediment (continued)

Test n.	Slope	Rainfall intensity (nominal) mm/h	Flow rate l/s	Concent. ppm	Average water depth in the pipe mm	Average sediment depth in the pipe mm	Sediment weight in the pipe g	Observations
1 T2K	1/200	300	16.7	1001	141	17	Not measured	Test duration: 15min Deposited bed Water level in gully = 0.49m above invert level of outlet pipe
2 T2A	1/100	50	2.78	500	43	0	Not measured	Test duration: 30min Deposition about 3m d/s from bend
2 T2B	1/100	100	5.56	500	63.3	0	Not measured	Test duration: 30min Deposition about 3m d/s from bend
2 T2C	1/100	50	2.90	959	44	0	Not measured	Test duration: 35min Deposition about 3m d/s from bend
2 T2D	1/100	100	5.56	1001	66	4	Not measured	Test duration: 33min Deposited bed along the whole pipe length
3 T2A	1/60	50	2.78	500	39	0	Not measured	Test duration: 30min No deposition
3 T2B	1/60	100	5.56	1001	53	0	Not measured	Test duration: 31min Flume traction (fully moveable sediment bed)

Table B.2 Test results with sumpless pot; granular sediment

Test no.	Slope	Rainfall intensity (nominal) mm/h	Flow rate l/s	Concent. ppm	Average water depth in the pipe mm	Average sediment depth in the pipe mm	Sediment weight in the pipe g	Observations
1 P2A	1/100	50	2.78	60	43	0	0	Test duration: 30min No sediment in collecting bucket Sediment deposition more than 5m d/s of bend
1 P2Aa	1/100	50	2.78	200	42	0	0	Test duration: 30min No sediment in collecting bucket Sediment deposition more than 5m d/s of bend
1 P2B	1/100	100	5.56	60	62	0	0	Test duration: 30min No sediment in collecting bucket or in the pipe
1 P2C	1/100	150	8.33	60	78	0	0	Test duration: 30min No sediment in collecting bucket or in the pipe
1 P2D	1/100	300	16.7	60	113	0	0	Test duration: 30min Water level inside the sumpless pot = 320mm above outlet invert level Considerable turbulence inside sumpless pot No sediment in collecting bucket or in the pipe
1 P2E	1/100	50	2.78	100	45	0	0	Test duration: 37min No sediment in collecting bucket Sediment deposition more than 5m d/s of bend
1 P2F	1/100	100	5.56	100	62	0	0	Test duration: 30min No sediment in collecting bucket or in the pipe
1 P2G	1/100	150	8.33	100	83	0	0	Test duration: 31min No sediment in collecting bucket or in the pipe
1 P2H	1/100	300	16.7	100	117	0	0	Test duration: 38min Water level inside the sumpless pot = 320mm above outlet invert level Considerable turbulence inside sumpless pot No sediment in collecting bucket or in the pipe

Table B.2 Test results with sumpless pot; granular sediment (continued)

Test no.	Slope	Rainfall intensity (nominal) mm/h	Flow rate l/s	Concent. ppm	Average water depth in the pipe mm	Average sediment depth in the pipe mm	Sediment weight in the pipe g	Observations
1 P2I	1/100	50	2.78	300	45	0	0	Test duration: 30min No sediment in collecting bucket Sediment deposition more than 5m d/s of bend
1 P2J	1/100	100	5.56	300	69	0	0	Test duration: 33min No sediment in collecting bucket or in the pipe
1 P2K	1/100	150	8.33	300	79	0	0	Test duration: 30min Water level inside the sumpless pot 210mm above outlet invert level No sediment in collecting bucket or in the pipe
2 P2L	1/100	50	2.78	500	42	0	0	Test duration: 30min No sediment in collecting bucket Sediment deposition more than 5m d/s of bend
2 P2M	1/100	100	5.56	500	62	0	0	Test duration: 30min No sediment in collecting bucket Sediment deposition more than 5m d/s of bend (more than in test 2P2L
2 P2N	1/100	50	2.78	1000	46	0	Not measured	Test duration: 32min A small amount of sediment around the walls of the collecting bucket Some deposition in the bend Flume traction (fully moveable sediment bed)
2 P2O	1/100	100	5.56	1000	65	1	Not measured	Test duration: 30min A small amount of sediment around the walls of the collecting bucket Deposited bed along the whole pipe length

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Appendix C: Tables of minimum pipe-full velocities for self cleansing with sumpless gullies – reproduced from HA105

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Table 5.1 of HA 105 - Design options and choice of appropriate tables below (A to D) for minimum flow velocities

Type of gully	**Downstream pipe system**			
	With Catchpits		**No Catchpits**	
Sumpless gullies (pot or chute) with basket	Upstream of Catchpit 1	Table A	Whole system	Table A
	Between Catchpits 1 and 4	Table B		
	Between Catchpits 4 and 10	Table C		
	Downstream of Catchpit 10	Table D		
Sumpless gullies (pot or chute) with bucket	Upstream of Catchpit 2	Table C	Whole system	Table C
	Downstream of Catchpit 2	Table D		
Conventional gully pot	Whole system	Table D	Whole system	Table D

Table A

Pipe Diameter (internal)	Minimum Velocity (m/s)
150mm	0.69
200mm	0.78
225mm	0.82
250mm	0.89
275mm	0.96
300mm	1.03
350mm	1.17
375mm	1.24
400mm	1.32
450mm	1.48
500mm	1.63
600mm	1.84

Table B

Pipe Diameter (internal)	Minimum Velocity (m/s)
150mm	0.67
200mm	0.71
225mm	0.72
250mm	0.73
275mm	0.76
300mm	0.79
350mm	0.84
375mm	0.87
400mm	0.89
450mm	0.97
500mm	1.05
600mm	1.21
700mm	1.38
750mm	1.47
800mm	1.56
900mm	1.76

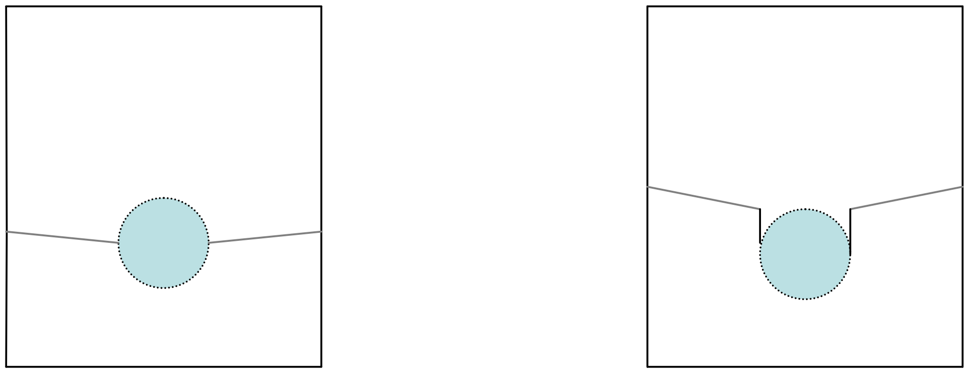
Table C

Pipe Diameter (internal)	Minimum Velocity (m/s)
150mm	0.67
200mm	0.71
225mm	0.72
250mm	0.73
275mm	0.74
300mm	0.75
350mm	0.76
375mm	0.77
400mm	0.78
450mm	0.79
500mm	0.81
600mm	0.85
700mm	0.91
750mm	0.93
800mm	0.96
900mm	1.00

Table D

Pipe Diameter (internal)	Minimum Velocity (m/s)
150mm	0.67
200mm	0.71
225mm	0.72
250mm	0.73
275mm	0.74
300mm	0.75
350mm	0.76
375mm	0.77
400mm	0.78
450mm	0.79
500mm	0.81
600mm	0.82
700mm	0.85
750mm	0.86
800mm	0.86
900mm	0.87

Appendix D: Types of chamber/benching covered in HR Wallingford Report SR624



Benching type I

Benching type II

Figure D.1 Schematics of the benching tested in the development of the Combined Surface Channel and Pipe system