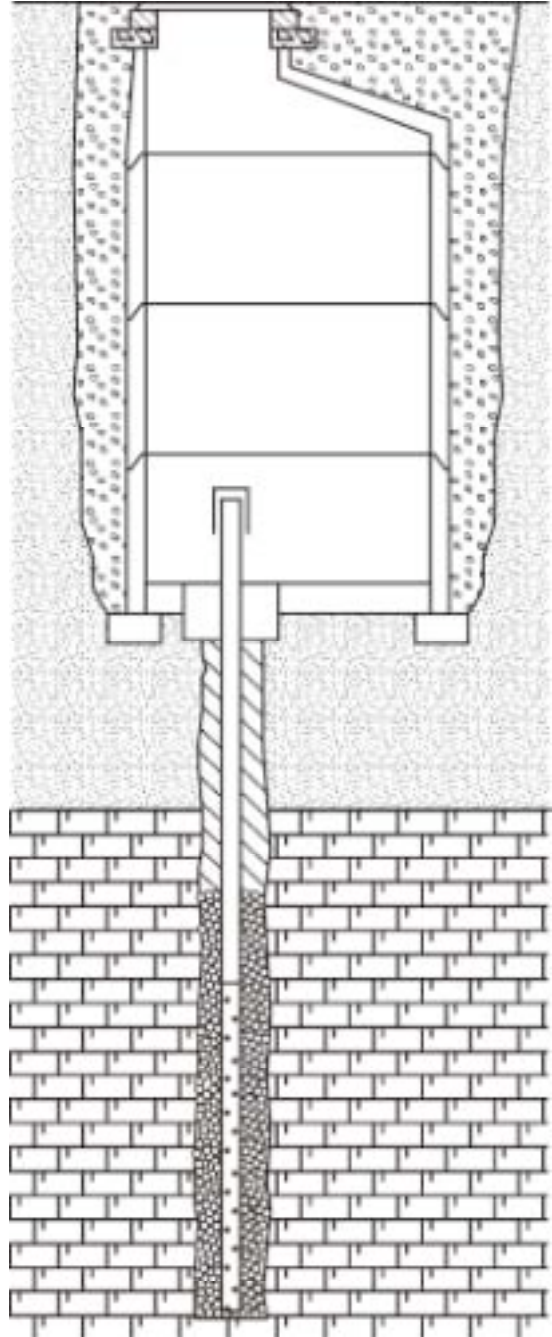
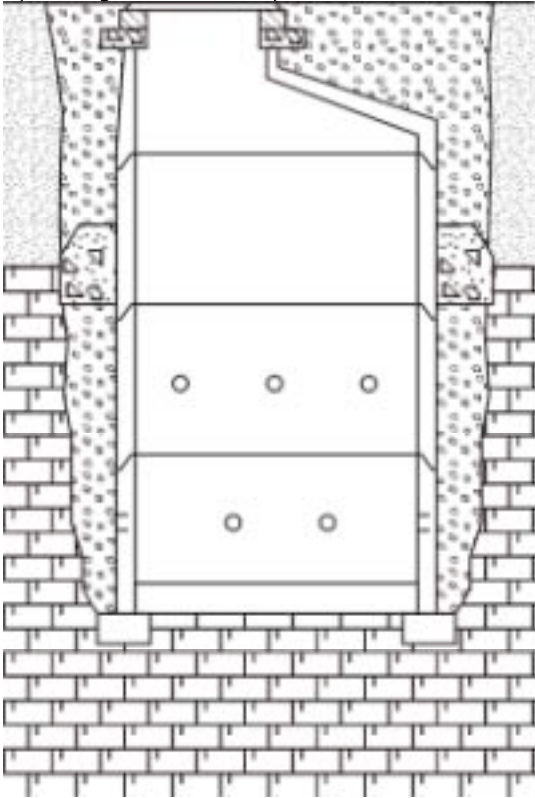


The Soakaway Design Guide (July 2000)



1b

Table of Contents

	PAGE NO.
Foreword	
Definitions and Abbreviations	
1. INTRODUCTION	1
2. THE CHOICE AND CONSTRUCTION OF SOAKAWAYS	2
2.1 The Choice of Soakaway	2
2.2 The Construction of Each Soakaway	2
2.2.1 Conventional Soakaway	2
2.2.2 Deep Bored Soakaway	3
2.2.3 Trench Soakaways	4
2.3 Influence of Geological Strata.	5
2.4 Aquifer Protection Policy	6
2.5 Influence of Ground Water Table	7
2.6 Silting-up of the Soakaway	7
2.7 Instability Around Soakaways.	9
2.8 Existing Failed Soakaways	9
2.9 Soakaway location	10
2.10 Risk Assessments	11
2.10.1 Chemical contamination	11
2.10.2 Solution Features	11
2.10.3 Impact on adjacent developments	12
2.11 Site Investigation	12
2.11.1 General	12
2.11.2 Desk Study	13
2.11.3 Ground Investigation	13
2.11.4 Proof testing of installed Soakaways	13
2.11.5 Examples of Soakaway site investigation guidance	13
3. EVALUATION OF THE FIELD SOAKAGE RATE	15
3.1 General	15
3.2 The Soakage Tests	15
3.2.1 Constant Head Test	15
3.2.2. Falling Head Test	15
3.3 Variation with Area	16
3.4 Variation with Head	16
3.5 Problems in Analysing the Soakage Rate	16
3.5.1 Saturated Stratum	16
3.5.2 Insufficient Supply of Water	17
3.6 Proof Test	17
3.7 Factor of Safety	18
4. METHODS OF DESIGNING DRAINAGE SYSTEMS	19
4.1 General	19
4.2 The Rational or Lloyd-Davies' Method	19
4.3 The Wallingford Procedure or Modified Rational Method	20
4.4 The TRL Hydrograph Method	21
4.5 Limitations to the Methods	21
5. DETERMINATION OF THE PEAK FLOW RATE.	22
5.1 General	22
5.2 Storm Frequency	22
5.3 Duration	22
5.4 Geometry of the Pipe Network	23
5.5 Gradient of the Catchment and Pipe Network	23
5.6 Impermeability of the Catchment Area	24
5.7 Comparison of the Three Methods	25
5.8 Design charts for calculating Peak Flows	25

PAGE NO.

6.	THE EVALUATION OF A SOAKAWAY'S CAPACITY	26
6.1	The effect of storage capacity.	26
6.2	Calculation of Storage Volume	26
6.3	Reduction of the Peak Flow	26
6.4	Reduction Factors	27
7.	SUMMARY OF DESIGN METHOD	28
7.1	Variability of design Information	28
7.2	Worked example (Case 4)	28
8.	REFERENCES	29
Appendix A	Examples	
Appendix B	Figures and Drawings	
Appendix C	Plates	
Appendix D	Soakaway Specifications	
D1	Specification for conventional chamber	
D2	Specification for deep bored soakaway	
Appendix E	Summary of Environment Agency Aquifer Protection Policy Requirements	
Appendix F	Current Value of Commuted Sums	
Appendix G	Highway Structure Definition and Policy for New Works	

LIST of FIGURES

Figure 1	Conventional soakaway
Figure 2	Deep bored soakaway
Figure 3	The formation of soil collapse around soakaways
Figure 4a	Examples of foundation failures caused by solution features or cavities in chalk.
Figure 4b	Typical dissolution features in chalk
Figure 5	Risk Assessment - Case history in Upper Chalk
Figure 6	Risk assessment - Case history in Hythe Beds
Figure 7	Constant Head soakage test calculations
Figure 8	Falling Head soakage test calculations
Figure 9	Extrapolation of Head versus time
Figure 10	Table 7 of Road Note 35
Figure 11	The effect of pipe network on the outfall hydrograph
Figure 12	The effect of pipe gradient on the outfall hydrograph
Figure 13	Peak flow rates for 2 year return period
Figure 14	Peak flow rates for 5 year return period
Figure 15	Definition of reduction factor
Figure 16	Reduction factor for 2 year return period
Figure 17	Reduction factor for 5 year return period
Figure 18	Geological sketch plan of Kent with deposit outcrops most suitable for soakaways

List of Plates

Plate 1	Road collapse caused by washout into gull feature in the Hythe Beds
Plate 2	Washout near soakaway in Hythe Beds
Plate 3	Solution features in Upper Chalk
Plate 4	Gravel filled reinstatements adjacent to drain runs over solution features in chalk
Plate 5	Isolated solution feature in Upper Chalk
Plate 6	Orange iron-staining to chalk close to a solution feature
Plate 7	Small gull feature in Hythe Beds showing local dip of ragstone layers

FOREWORD

This guide has been developed to be used in conjunction with the Kent Design.

The design and construction of soakaway systems outlined in "Housing Design Guide" published in 1976 by the Kent Planning Officers and "The Design and Construction of Estate Roads in the County of Kent" published in 1975 and subsequently amended were both superseded by Technical Memorandum M86/1.

The June 1987 Technical Memorandum M86/1 was prepared by Mr KS Rogers and Mr D Thomas of the Kent County Council's Highways Laboratory's Geotechnical Group. It was based upon 20 years' experience in the design and construction of soakaway systems in Kent.

This Soakaway Design Guide has been issued in July 2000 generally to update the M86/1 document. This update followed the December 1992 issue of the National Rivers Authority Aquifer Protection Policy which superseded the 1985 Southern Water Authority and Thames Water Authority Aquifer Protection policies. It also follows the March 2000 Issue of the Kent Design Guide which updated the earlier January 1977 document. This Soakaway Design Guide has been prepared by Ian Payne, formerly a member of KCC's Geotechnical Group and subsequently Babbie Geo-Engineering from January 1999. The revised document is intended to raise the profile of drainage to help prevent avoidable delays during the various stages of approval.

It is hoped that the Guide will create a better understanding for designers of soakaway systems. It must be stressed, however, that certain District and Borough Councils will have their own particular standards/requirements relating to some aspects of design/materials.

The Soakaway Design Guide is not to be regarded as a substitute for professional expertise in the preparation of particular proposals. It is always important to employ a competent designer in the first place, and all persons who are contemplating development are urged to employ expert professional advice. The illustrations accompanying the text are intended to clarify and illuminate the points made in writing.

The following companies and organisations have kindly contributed to the consultation process:-

- Local Authority Councils - Ashford BC, Canterbury CC, Dartford BC, Dover DC, Gravesham BC, Maidstone BC, Medway Towns Council, Sevenoaks DC, Shepway DC, Swale BC, Thanet DC, Tonbridge & Malling BC, Tunbridge Wells BC
- Kent County Council - KCC Network Management, KCC Transport Client Services, KCC N.Kent Area, KCC Mid Kent Area, KCC E. Kent Area
- Geotechnical Consultants - Babbie Geo-engineering (formerly Kent CC Geotechnical Group), Symonds Group Ltd, Weeks Technical Services PLC, Peter Brett Associates, Owen Williams Group (formerly East Sussex CC Geotechnical Group), WSP Graham Development Ltd, Ground Solutions Group, Southern Testing Laboratories,
- Other interested parties - Environment Agency, Health and Safety Executive

DEFINITIONS

suitable strata	strata into which the discharge of water will not cause deterioration of the inherent structure and strength (Section 2.3)
unsuitable strata	strata other than suitable strata as defined in the previous definition (Section 2.3)
The Engineer	<p>means the KCC Head of Network Management or his authorised representative, or in the case of an Agency area, the District Technical Officer or his authorised representative.</p> <p>Kent County Council Strategic Planning Network Management Invicta House Maidstone Kent ME14 1XX</p> <p>Telephone: Maidstone (01622) 221089 or 221090 Fax: Maidstone (01622) 691028</p>
Water Authority	means the Southern Water Authority or the Thames Water Authority according to the location within the County.
Environment Agency	<p>Guidance on aquifer protection zones can be obtained from:-</p> <p>Environment Agency Southern Region Guildbourne House Chatsworth Road Worthing West Sussex BN11 1LD</p> <p>Telephone: Worthing (01903) 832000 Fax: Worthing (01903) 821832</p> <p>Site specific guidance on Groundwater issues in the Kent area can be obtained from:-</p> <p>Environment Agency Groundwater Protection Kent Area Office Orchard House Endeavour Park London Road Addington West Malling Kent ME19 5SH</p> <p>Telephone: Tonbridge (01732) 875587 or 223117 Fax: : Tonbridge (01732) 875057</p>

ABBREVIATIONS

mBGL	means metres below ground level
l/s	means litres per second
l/m ² /min	means litres per square metre per minute
PPE	means Personal Protective Equipment
vs.	means versus

ORGANISATION ABBREVIATIONS

KCC	means Kent County Council
EA	means Environment Agency
NRA	means National Rivers Authority
AGS	means Association of Geotechnical Specialists
HMSO	means Her Majesty's Stationary Office
NHBC	means National House Building Council
DC	means the Council (whether District, City or Borough Council) of the administrative District in which the Development is situated.

1. INTRODUCTION

Soakaways have long been used to collect rainfall accumulating upon a catchment area prior to being discharged into the surrounding soil. In North Kent they can be found serving a large number of highway schemes and private housing developments. It is common for highways to formally channel the water into these soakaways by a system of kerbs, gullies and pipes. They are usually employed in areas where the surface run-off cannot be disposed into existing streams or ditches

A soakaway is constructed in two basic forms. A conventional soakaway comprises a partially perforated cylindrical chamber which allows the drainage of water into the surrounding soil. Where the soakage must take place at depth a deep bored liner is used. Here the water is transported from a sealed storage chamber to the soakage medium by a small diameter plastic or steel pipe, again perforated in part.

Although the construction of these systems tends to have been more unified in recent years, probably due to the standard details promoted by the Kent County Council Kent Design Guide and the former Kent County Council Geotechnical Group the same cannot be applied to their design. In the past the approach to the design of their geometry has been somewhat varied.

It is intended that this Soakaway Design Guide will pool all previous knowledge to update the existing standard details and specification to the soakaway systems.

Some clients have been unsure of what to specify when commissioning the design of a soakaway and this is especially true for the case of one with a deep bored liner. Geotechnical consultants and contractors occasionally find that they have been requested to simply install a soakaway to drain an unknown quantity of water falling upon an unknown area. Clearly this form of instruction has posed problems in determining the size of the soakaway. Other clients, more certain of their requirements, state the soakage rate they expect. Naturally there are a number of intermediate situations which arise offering different approaches to their solution.

When supplied, the soakage rate calculated by the client is invariably one directly related to the peak flow arriving at the soakaway location. Sometimes this is high and impractical for a soakaway system. The soakaway should not be simply designed to match the peak inflow of water and should allow for storage capacity of the chamber to reduce the required soakage rate.

The chamber acts as an underground storage lagoon, the function of which is to attenuate the peak flow. This can be compared with an electrical capacitor in that it stores the water at the height of the storm and discharges it after this peak. The overall effect is to reduce the required soakage rate.

The second aim of this memorandum is therefore to define a set of procedures which enable a satisfactory soakaway design to be made given any initial condition presented by the client. Moreover, the assistance offered by the storage capacity of the soakage is also considered to enable a more efficient design to be made.

The methods outlined herein consists of a set of procedures supplemented by a number of design charts.

2. THE CHOICE AND CONSTRUCTION OF SOAKAWAYS

2.1 The Choice of Soakaways

Usually the choice of alternative soakaway types in figures 1 and 2 is self-evident in that conventional soakaways can be constructed generally to a depth of up to 7 metres below ground level (mBGL) although the deeper the chambers are the more difficult and potentially more dangerous they will be to construct and maintain. If the suitable soakage medium lies at greater depths then this can only be breached by a deep bored soakaway. Therefore, the choice is generally decided upon by the depth to the suitable soakage medium subject to the approval of the Environment Agency (EA). The introduction of Building Research Establishment (BRE) Digest 365 entitled “Soakaway Design” has raised the profile of near surface trench soakaways although historically the variability and unsuitability of the near surface soils in Kent, greater maintenance liability and difficulty in maintenance has resulted in KCC only permitting trench soakaways in exceptional circumstances.

It may be evident that although the suitable soakage medium may lie close to the ground surface the discharge of drainage water into the surrounding soil at this level could promote other problems with neighbouring structures, slopes and landfills in addition to increasing risks of washout and flooding. A typical example of this would be to site a conventional soakaway a few metres away from a house or factory allowing water to permeate beneath the foundations which could induce instability as illustrated in figures 3 and 4. This is, of course, undesirable and if the soakaway cannot be relocated a deep bored liner may be necessary to channel the water out of the soakaway at a much lower level.

2.2 The Construction of Each Soakaway

2.2.1 Conventional Soakaway

The typical conventional soakaway is depicted in figure 1 with a specification in Appendix D1. It basically comprises a cylindrical chamber into which the surface run-off from the catchment area is collected. The chamber is formed from precast cylindrical rings approximately 1 metre high which are stacked upon each other to produce the required height to achieve the required effective storage volume. Typical diameters of soakaway rings are 1.2m, 1.5m, 1.8m, 2.1m and 2.4m. Chamber diameters greater than 1.5m will be classified as a Highway Structure as detailed in Appendix G which will require Technical Approval from KCC's Network Management Bridge Management through their agents Babbie Group (Highway Structures Office).

The surface run-off discharges into the surrounding soil through perforations in these rings, although not all rings supplied by the manufacturers are perforated for use in unsuitable soakage strata. It is important to prevent water seeping into unsuitable soils overlying the soakage medium for the reasons discussed more fully in section 2.7. This is normally executed by installing non-perforated rings in the region of these strata and extending a minimum of 1 metre into the suitable strata. Thorough sealing of the joints in the rings and sealing of any perforations such as lifting hook openings are also required in these non-perforated rings. Access into the soakaway chamber must be treated as a confined space and can only be undertaken with the necessary safety precautions (Gas detector, safety harness, tripod winch, backup team etc)

The rings are located upon a concrete base to prevent the soil being washed away and eventually undermining the bottom rings. If this should occur the soakaway and associated inlet pipework is likely to become unstable. The concrete base also assists cleaning out of the debris that collects in the chamber by preventing over-excavation of the chamber which could also undermine the stability of the chamber. The excavation in which the soakaway is located is backfilled with a suitable free draining granular material. This disperses the concentrated flows of water passing through the perforations and allows the soakage medium to be fully utilised.

A conventional soakaway is often more effective than a deep bored or trench soakaway system in discharging the surface run-off into the surrounding strata. It offers a large surface area through which the water can percolate and as such generally performs better than the field tests predict. This is due to two factors. The conventional soakaway exposes a greater number of joints per unit area than produced by a field test carried out in a borehole. Additionally, the borehole undoubtedly causes a certain amount of smearing on the side which masks to some extent the presence of the joints. A conventional soakaway construction does not usually promote very much smearing and what does appear can, if required, be 'plucked' or brushed away.

Whilst the base of a conventional soakaway will be in a suitable strata, loose Made Ground may be encountered close to the ground surface. Settlement of the Made Ground could result in settlement of the inlet pipe into the soakaway, opening of joints and water leakage. Greater inlet pipe gradients and greater flexibility of the inlet pipe system may also be required particularly adjacent to the rigid soakaway chamber. The National House Building Council (NHBC) Standards Chapter 5.3 "Drainage below ground" also recognises this problem where pipes pass through a rigid structure and have increased the reticulation of the pipework by introducing flexible joints and a greater frequency of joints which may need to be considered on a scheme specific basis for adoptable soakaways. Ground improvement measures such as excavation and recompaction/replacement may also be considered local to the inlet pipes.

2.2.2 Deep Bored Soakaway

The deep bored soakaway is illustrated in figure 2 with a specification included in Appendix D2. It consists primarily of two parts, a conventional chamber and a narrow bore perforated liner. The chamber is constructed in more or less a similar manner as the conventional soakaway, but here it is generally used solely for storage. The chamber is generally formed wholly from non-perforated rings. In exceptional circumstances the lower part of the chamber may be perforated provided the perforations commence a minimum of 1 metre into a suitable permeable strata.

Passing through the base, which again is concrete, is a plastic or steel liner usually 100 to 150mm in diameter although diameters of between 80 and 300mm have been used in Kent. Limitations in depth of the liner as a result of groundwater protection concerns are increasingly resulting in the use of 200mm and 250mm diameter liners. The liners are inserted into a borehole drilled normally by a light percussion or rotary auger rig depending upon the underlying geology with the diameter of the borehole a minimum of 50mm greater than the outside diameter of the liner. The plastic liner is in part perforated to facilitate discharge of the surface water run-off into the surrounding soil and as such its length is dependent upon the required soakage rate although it may be limited in depth by the requirements of EA. A section of non-perforated liner is normally installed to below the interface with the soakage medium and the overlying relatively impermeable soils. It is recommended that the non-perforated liner penetrates at least 2 metres into the soakage medium.

Between the perforated liner and the side of the borehole is a gap which is backfilled with 'pea-gravel' or similar typically 10mm in size and up to 20mm maximum size. Like the granular material surrounding a conventional chamber this helps to disperse the water evenly from the liner perforations into the soil and backfills the void between the borehole and the liner to allow the installation of the bentonite seals above the perforated section of the pipe. As the diameter of the borehole is more controlled than the dimensions of the conventional chamber excavation, the soakage area of the liner is taken as the borehole diameter.

Above this annulus of granular material lies a bentonite seal which restrains the water to below this level even when the run-off backs up into the chamber. For particularly sensitive sites it may be appropriate to introduce a reducing coupling local to the grouted area to enable a thicker annulus of grout to be obtained (eg reduce 150mm perforated pipe to 100mm plain pipe at or below the grouted zone and extend 100mm pipe into the chamber) and/or increase the depth of the seal above the standard depth increment of 2 metres.

Capping the liner which protrudes 1 metre above the base of the chamber is a syphon head placed to prevent the blockage of the deep bore. This also helps to inhibit the ingress of silt into the liner. Access into the soakaway chamber to install the syphon head must be treated as a confined space and can only be undertaken with the necessary safety precautions (Gas detector, safety harness, tripod winch, backup team etc).

The deep bored soakaway intercepts far fewer joints than the conventional soakaway and therefore relies upon the driving head developed by the water column to force the run-off through the available fissures that are encountered in the side of the borehole. This installation is prone to smearing the fissures in the borehole which further reduces the soakage rate. Undertaking a series of soakage tests as a borehole is deepened and then installing a soakaway liner can increase the risk of smearing of the joints and will require testing of the installed liner to confirm the soakage rates.

The field tests do, however, satisfactorily model the capacity of the deep bored soakaway generally being of a similar diameter to the proposed liner. Field tests on the installed liners are possible and less water is generally required to demonstrate the liner capacity compared to a conventional soakaway although tankers will often be required to fully justify the capacity of the deeper liner systems in highly permeable strata.

In general terms drilling within the Alluvial gravels and chalk can be undertaken by conventional light cable percussion (shell and auger) techniques. The 'rock-like' structure of the Lower Tunbridge Wells Sands and especially the Hythe Beds often precludes the use of the light percussion rig methods. In such deposits it is necessary to utilise rotary coring drilling techniques. The costs of this specialist drilling is however quite high, typically 2 to 4 times that of light percussion rig work. In some circumstances it may be advantageous to employ a light percussion rig to drill through the overburden materials and prove the position of the underlying 'rock' strata. Once this has been undertaken the more expensive specialist rotary drilling rig can be brought onto site to complete the works.

For brownfield sites, whilst the liners themselves may be in suitable strata, the chambers and/or inlet pipes may be in loose Made Ground. Settlement of the chamber and/or opening of joints in the inlet pipes will stop some of the water reaching the soakaway liner which may result in surface settlements and/or washouts. It is therefore important to consider the need for ground improvement local to the soakaway chamber and greater flexibility in the inlet pipes, the latter being discussed in section 2.2.1.

2.2.3 Trench soakaway

The 1991 BRE Digest 365 has raised the profile of trench soakaways and whilst they may be acceptable for house drainage KCC has generally discouraged their use for surface water discharge disposal for the following reasons:-

- (a) Inability to inspect and maintain the trenches
- (b) Variability of near surface soils across the site
- (c) Greater impact on adjacent structures and services in the event of washouts
- (d) Greater risk of flooding/saturated ground.
- (e) Greater risk of damage by service trenches
- (f) Effect on vegetation as a result of this local underdrainage into the granular backfill.
- (g) Ability to sterilise these areas from future development (services, trees, street furniture etc)

KCC accept that occasionally there may be no alternative but to consider the use of trench soakaways although this will be on a scheme specific basis and only with the written consent of the adoption authority. Typical trench details are shown in BRE 365 and the concerns described above suggest that a number of issues will need to be considered in the design of the trench soakaways including:-

- (a) Fall of trench - Trench depth to have a fall into the adjacent chambers to minimise the risk of undermining of the chamber
- (b) Smearing of excavation - Roughening/brushing sides of the trench will minimise reduction in soakage potential produced by smearing of fissures etc (eg smearing of fissures in chalk by puttyfied chalk or overlying soils). Adequate safety precautions will be required for entering trenches greater than 1.2 metres deep (Shoring, Gas detector, PPE etc) and shoring may be required for unstable excavations less than 1.2metres deep.
- (c) Geotextile - consider the need for a geotextile filter fabric to line the sides of the trenches. The geotextile design must consider the pore size needed to filter the adjacent soil type, the risk of clogging of the fabric, tear resistance to allow for infilling with gravel (height of drop, angularity of gravel etc) and overlaps to ensure continuity.
- (d) Gravel infill - place uniformly graded granular fill in trench to underside of perforated pipe (eg Type B, Type 6C or single sized coarse gravel) which is generally easier to compact than a well graded granular fill and has a greater void space for storage capacity.
- (e) Place perforated pipe at a minimum of 0.9 metres below finished ground level and connect into chambers where appropriate (to allow rodding etc). The depth of the pipe must take account of external loadings (eg wheel loads) , root infiltration risks etc.
- (f) Place uniformly graded granular fill around and above perforated pipe to underside of topsoil (eg 0.35metres below finished ground level).
- (g) Place geotextile filter fabric above granular fill and extend 0.5metres beyond edges of trench to prevent ingress of fines into trench (see geotextile design comments above).
- (h) Place topsoil above filter fabric.
- (i) Location - keep away from areas sensitive to saturation, flooding or washouts (eg offset from houses, roads, services etc)
- (j) Commuted sums- consider the need for enhanced commuted sums for maintenance of the system and increased risk of the need for replacement at some stage in the future.

2.3 Influence of Geological Strata

The soakage rates depends on the ability of the surrounding soil to absorb water and hence its transmissivity/perviosity (the mass permeability). Therefore, the more open the macrostructure of the stratum the greater the soakage rate. Clearly, some soil deposits will be totally unsuitable for the installation of soakaways. These consist predominantly of the clays from the Cretaceous, Palaeocene and Eocene periods (e.g. Weald, Woolwich and London Clays respectively) together with the fine grained drift deposits such as Head and Clay-with-Flints.

The sandy materials of the strata like Folkestone and Thanet Beds soils may initially seem suitable, but in practice they provide a very poor soakage medium which often 'silts up' very quickly. Moreover these soils can promote collapse settlement at the ground surface where concentrations of water occur as discussed in section 2.7. The use of these strata will need to be reviewed with the adoption authority on a scheme specific basis and will require a careful review of the grading of the granular annulus backfill. A suitably designed non-woven geotextile filter fabric may also be required.

In Kent there remains only a few types of suitable strata into which a soakaway can be constructed. Chalk and the Hythe Beds have been used widely throughout North and Central Kent as deposits into which surface run-off can be discharged. The soakage rate depends upon the frequency of the fissures and their degree of openness. In the case of chalk, zones of poor quality often have narrow spacings and wide gaps for the fissures which offers very good soakage qualities. Conversely good quality chalk, in terms of foundation engineering, can be bad in that few fissures are intercepted and in any case are small in gap width as illustrated by the tight fissures in the Istead Rise and Ifield areas of Kent. The same can be stated for the Hythe Beds.

In the south of Kent the Lower Tunbridge Wells Sand tends to be fissured and occasionally holds water. The use of this material should, however, be seriously questioned because the soil is easily eroded and could again lead to surface collapse. Seepages have also reduced the suitability of the stratum in some locations.

Pure Alluvial gravel can be used for a soakage medium, but they tend to be limited in thickness both vertically and laterally and can have variable particle size distributions including frequent sand horizons and occasionally clay layers. They are usually close to an existing river or stream into which the drainage water can be channelled with the approval of EA.

A geological sketch plan outlining the outcrop of the major deposits potentially suitable for soakage installation is presented on Figure 18. It should be utilised as purely a preliminary guide and it is recommended that advice be sought from a Geotechnical Specialist at an early stage. The plan does not indicate those areas where unsuitable materials overlie suitable deposits in which the installation of deep bored soakaways may be feasible.

2.4 Aquifer Protection Policy

The Aquifer Protection Policy has been prepared by EA, formerly the National Rivers Authority (NRA), to protect aquifers from pollution. The aquifers in Kent provide approximately 60% of the public water supply. It is therefore important to ensure that the water mass adjacent to extraction points does not become polluted whether by discharge, soakage or spillage. The EA document is both comprehensive and technical and a full explanation of the requirements and restrictions is well beyond the scope of this design guide.

In the earlier 1985 Aquifer Protection Policy, the county was categorised into a series of five zonal areas. The five protection zones adopt a dual criteria based on the nature of the geological strata and the type/source of contamination, zone I being the most sensitive. The current 1992 NRA Aquifer Protection Policy similarly divides the county into a series of three zonal areas with up to a 50day travel time within Zone I and up to a 400day travel time within Zone II. This policy has been adopted by EA.

Certain activities in a densely populated country will by necessity present risks to the water environment and the EA aims to direct such activities to areas of least risk. EA does not wish to be unduly restrictive to development but wishes to reach the best technical solution with the available information. EA has a prime interest in protecting the water environment and therefore there will be limitations in the scope for deepening soakaways and in their plan location relative to sensitive aquifers or public water supply tunnels. With groundwater it is too late when pollution has occurred and the EA emphasis must lie with pollution prevention.

A series of ground vulnerability maps have been produced in conjunction with the 1992 aquifer protection policy in addition to the maps of the protection zones around abstraction wells. The ground vulnerability maps are available to the general public from HMSO as an aid to planning new industrial development, waste disposal and intensive agriculture to be located in areas which are less vulnerable to pollution. Local site investigation data must always be obtained for site specific assessments. The maps generally show that the lowest risk areas to EA are the least suitable for the use of soakaways being areas of low permeable soils and/or high washout potential. The fissured nature of the strata most suitable for soakaway discharges are also those which have the fastest travel times for contaminants and would be classed as high vulnerability areas on the ground vulnerability maps.

It is impossible to incorporate this information onto a plan to be presented in this document as the location of the zones is very detailed with variable requirements for each zone. It is therefore essential that the Environment Agency is contacted at an early planning stage.

The EA's "Policy and Practice for the Protection of Groundwater" includes policy statements on discharges to underground strata and should therefore be utilised when considering discharges to soakaways. Highway drainage may be opposed in sensitive protection zones around public water supply boreholes. Extracts from the document specifically related to highways are included in Appendix E.

An EA "consent to discharge" is required to discharge effluent (which includes surface water run-off from highways) to controlled waters which are defined as inland, coastal and underground waters. It is therefore a requirement of the EA to obtain a "consent to discharge" licence when constructing a soakaway which drains into an aquifer protected by the aquifer protection policy. The EA should be contacted at the design stage of a development if it is proposed to construct highway soakaways, to establish whether there is a need for a licence. If the EA requires a licence, then the Highway Authority will require the Developer to pay to the Highway Authority prior to the adoption of the works, a commuted sum to cover the future costs likely to be incurred by the Highway Authority in paying annual charges to the EA in addition to the maintenance commuted sums in Appendix F.

Applications for consent to discharge will be considerably speeded up if the following data is provided to EA's groundwater protection team:-

Information required	Example
Site location name	Culverstone Green School
National grid reference	TQ632635
Expected geology	6.5m Clay with Flints overlying Upper Chalk
Expected groundwater level	110 to 130mBGL (40mAOD)
Preferred type of soakaway	Deep bored
Preferred depth of soakaway	20mBGL

Failure to comply with the recommendations made by EA can result in prosecution.

2.5 Influence of the Ground Water Table

Soakage directly into groundwater is unlikely to be acceptable to EA except in exceptional cases as travel times for contaminants discharged directly into groundwater will be very fast. An example where discharge into groundwater may be acceptable could be local to the coastlines where the groundwater is highly saline and groundwater flows away from abstraction sources towards the coastline. Coastline locations may have more readily available surface water outfalls to remove the need for soakaways but this is not always the case.

If after consultation with EA it is deemed necessary to install a deep bored soakaway below the groundwater table this may have serious consequences in the soakage capacity if chalk is the surrounding medium as a result of the putrefaction of the chalk during drilling. The action of forming the borehole under water tends to puttify the chalk even more than if it was dry and a slurry may be produced blocking the inherent fissure system and significantly reducing the soakage rate. The chalk slurry produced during drilling will also need to be carefully controlled particularly in urban areas to minimise the impact of the spread of chalk silt on the surrounding areas.

Poor soakage is further compounded by the fact that the driving head in the soakaway is limited by the groundwater table and the required unsaturated zone above it. High water tables can appreciably restrict the driving head and hence the soakage capacity of a given system.

2.6 Silting-up of the soakaway

Whether a conventional or deep bored soakaway is installed it is prone to collect silt in the chamber. Surface run-off naturally transports fine particles of soil from the road and verges which eventually channels it into the soakaway. With time this accumulates in the bottom of the chamber and threatens the soakage capacity in two ways.

The silt can migrate through the perforations and clog the fissures of the soakage medium. This of course depends on whether the granular surround can allow the passage of the silt. If this filters out the silt and prevents its transition through to the soakage medium then the perforations become filled and the capacity of the soakaway is very much reduced. This state is generally only reached after the soakaway has partially filled in order that a reservoir of silt is available to be stored up into suspension when the water enters the chamber

In a conventional soakaway its capacity is reduced more or less immediately because the presence of the silt influences the available storage volume. With a deep bored soakaway the perforated liner is protected by fixing the tube above the base and capping it with a syphon head. This provides a volume whereby silt can collect, although excessive build up will enable the silt to become water borne and be drawn down into the liner.

New soakaways and silt traps should be jetted and cleaned out prior to adoption by the highway authority as considerable quantities of silt can be generated during construction works associated with a development. It is essential that the soakaways are then cleaned out at regular intervals during their lifetime. Linked soakaways also put a greater burden on the initial chamber in the sequence and the associated inlet pipe which will require a greater frequency of maintenance for the initial chamber in a series compared to a series of isolated soakaways. Banks of soakaways and their connecting pipes will effectively sterilise a large area from development. They are often located in areas of public open space which may still be subject to future development including childrens play areas, tree planting, lighting columns and service trenches. The depth of the connecting pipes must still consider the risk of future trenching taking place in the vicinity of the soakaways.

While it is appreciated that cleaning of the soakaways places a financial burden on the maintaining authority the consequences of not doing so is substantially more expensive. If the fissures become silted up around a conventional soakaway then it is necessary to install a deep bored liner and instead of dealing with a few tens of pounds the cost would be measured in terms of hundreds of pounds. The same could, of course, be declared for a deep bored soakaway which requires redrilling. In the extreme case of a collapsed soakaway the cost involved is thousands of pounds and if it is located close to a building the structural damage could be measured in terms of tens of thousands of pounds.

It is important to realise the necessity of maintaining these soakaways and KCC are now introducing commuted sums for all soakaways which vary in cost dependent upon the type of soakaway. These charges will vary dependent upon the ease of maintenance, longevity expectation of the system and possibly ground conditions at the site as follows:-

Type of soakaway	Surface distress if fails	Risk of activating solution features	Ease of maintenance	System replaced	Cost of decontamination	Commuted sum
Conventional chamber	moderate	Moderate	easy	Infrequent	Medium	Medium
Deep Bored soakaway	low	Low	difficult	Infrequent	High	Medium/High
Linked/ringed soakaways	moderate	Moderate	easy	Infrequent	Medium	High/Medium
Trench soakaways	moderate/high	moderate/high	difficult	Frequent	High	High

The current values of these maintenance commuted sums are contained in Appendix F. The figures in Appendix F will be periodically reviewed and may be updated in future revisions of the document so that it is important to check that the most recent update is being used.

2.7 Instability Around Soakways

Instability of the area around the soakaway occurs in the form of surface collapses. For this to manifest itself the soakage medium is usually overlain by a granular soil. This is common for chalk where either a granular Head or Thanet Beds is normally present. With Hythe Beds the limestone layers provide the soakage medium, but these usually sandwich a sandy layer.

The mechanism of instability first arises by the washout of the sandy lithologies from the drainage water being discharged into the surrounding soil. In the case of Hythe Beds the perforations allow water to permeate the sandy layers direct. Where the soakaway is located in chalk the perforations close to the interface with the overlying superficial deposits allow the water to locally wash away the sand.

The sand is transported by the water into neighbouring fissures. These fissures may be naturally open or enlarged by chemical erosion by the drainage water. This chemical effect attacks both the chalk and the Hythe Beds although this is substantially less in the fissured ragstone of the Hythe Beds.

Having been channelled away from the vicinity of the soakaway the removal of the sand forms a small void. Naturally this void enlarges until such a point where it becomes unstable and the roof of the void collapses. At this stage the collapse may not appear at the surface, but with continual erosion by the drainage water the void will eventually migrate to the surface.

A schematic diagram of this phenomenon is shown on figure 3. This is included to visually reinforce the problem of allowing water to permeate into the superficial or sandy layers. Activation of solution features can also induce instability local to soakaway chambers as discussed in section 2.7. Collapse of man made caves, tunnels and deneholes can also be triggered by concentrations of water from soakaways as illustrated in figure 4.

2.8 Existing Failed Soakaways

Existing, but failed, soakaways either in the sense of ineffective soakage or structural damage tend to be caused by constructing them in the adverse conditions outlined in the previous sections. Usually they are either wholly located in the overlying sand or close to the interface with the chalk or Hythe Beds.

Depending upon the damage involved it is normally essential to cast a concrete base to the soakaway (these are invariably never constructed in the first place), seal all the joints and holes and line the lower portion of the chamber with a 'bituthene' coating protected by some form of dwarf wall. Soakage should then be effected by a deep bored liner.

For failed deep bored liners a number of options are available:-

- (a) Rebore existing liner - a larger diameter borehole will normally be required and problems may occur if the original or new borehole is not drilled vertically. Visual inspection of soil samples and further soakage tests will be required to assess whether silt migration has affected the soakage capacity of the strata. The stability of the area local to the chamber and the risk of leakage from the chamber must also be assessed.
- (b) Install replacement borehole soakaway elsewhere in the chamber - This will either require the cover of the chamber to be rotated or a core hole through the soakaway cover to provide access to the new borehole location. Coring through the cover will require approvals from the soakaway manufacturer and the adopting authority. The stability of the area local to the chamber and the risk of leakage from the chamber must also be assessed. Decommissioning of the failed deep bored liner may also be required.
- (c) Reconstruction of the chamber and deep bore at a new location - this may be required if the damage is severe. Decommissioning of the failed deep bored liner may also be required.

2.9 Soakway location guidance

The following published information (References 1 to 4) relates to the location of soakaways adjacent to residential properties:-

- (a) Building Research Establishment Digest 151 "Soakaways" (1981)

"In land overlying chalk there may be serious risk of swallow holes and these may be activated by the concentrated discharge from a soakaway.....Because of the wide variation in soils and site conditions it is not possible to give any generally applicable guidance as to the 'safe' distance from a

- building but the local authority can usually offer advice on this based on their detailed knowledge and experience of the locality.”
- (b) British Standard BS8301 :1985 “Code of practice for building drainage” (1985)
“A soakaway is not desirable nearer to a building than about 5 metres nor in such a position that the ground below foundations is likely to be adversely affected.”
 - (c) Building Research Establishment Digest 365 “Soakaway design” (1991)
“Soakaways should not normally be constructed closer than 5 m to building foundations. In chalk or other soil and fill material subject to modification or instability, the advice of a specialist geotechnologist should be sought as to the advisability and siting of a soakaway”.
 - (d) Kent County Council “Kent Design” (2000)
“Highway soakaways shall be situated the following minimum distances from dwellings (measured from the outside edge of the soakaway):-
(1) 5 metres for conventional perforated soakaway
(2) 10 metres for deep bored soakaway”

Adjacent soakaways of a similar type shall also be spaced a minimum of 10 metres apart.

Some geotechnical consultants suggest that a 15 metre zone between a building and a deep bored soakaway is preferable. This is clearly safer than the 10 metre and 5 metre zones quoted above. It is not based upon any published guidance and a 15 metre safety zone is often impracticable with much smaller safety zones having to be adopted for many deep bored soakaways as a result of limited available space. More detailed site investigation between a soakaway and a dwelling may need to be considered for particularly sensitive sites

All developments need to consider the cost, maintenance requirements, risks and practicality of soakaway construction compared to alternative forms of surface water discharge although soakaways are often the preferred option. Ideally all soakaways should be located as far as possible away from buildings. The soakaways should also be located outside the carriageway and preferably outside the footpaths where Statutory Undertaker’s plant is located. A balance is therefore often made regarding the offset of the soakaways from buildings and offsets from carriageways where space is limited.

Geological factors including the risks of washouts, activation of adjacent solution features, activation of adjacent landslips, the location of sensitive aquifers and adjacent landfills/contaminated land must also be considered when siting a soakaway. A cone of influence of 45° from the discharge points in a soakaway surrounded by soil with similar vertical and horizontal permeability can be used to provide an initial assessment of the zone of impact of a soakaway. A more rigorous assessment will be required adjacent to sensitive sites and where horizontal fissure flows are more dominant.

Lines of adjacent soakaways of a similar type shall be spaced a minimum of 10 metres apart to reduce the risk of interaction between them which will increase the risk of instability around each chamber and will reduce the combined capacity of the system. For soakaways in uniform materials that are spaced less than 10 metres apart the following reductions in soakage area are assumed for each soakaway:-

Soakaway spacing (m)	10	8	6	4	2	1
Reduction in capacity per soakaway	0	12.5%	25%	37.5%	50%	50%

A ring of soakaway chambers will further reduce the combined capacity of the system dependent on the radius of the ring of the soakaways. A similar principle of reduced capacity will need to be applied to these soakaways. The interaction of banks of soakaways will need to be reassessed where soakaway discharge relies on isolated fissures (eg Hythe Beds).

Lines of linked soakaways with overflows into adjacent soakaways should be avoided wherever possible by intercepting the flow of water in stages. Linked soakaways inevitably result in a greater maintenance liability for the first soakaway in the chain of soakaways. This can increase the risk of leakage occurring and the implications of problems with the inlet pipe are much greater.

The locations of soakaways must also take account of the need of future maintenance requiring access for a gully emptier. Sites in back gardens or where there may be future vegetation planting are clearly undesirable for such maintenance and tree roots could also damage inlet pipes. The need for easements also needs to be considered. Drop kerbs and access tracks will highlight the location of the soakaway chamber to reduce the risk of future development obscuring access for maintenance.

2.10 Risk Assessments

To minimise the liability to the adopting authority a series of Risk Assessments are recommended including chemical contamination, solution features and the effects of adjacent developments. These assessments should be undertaken in 2 stages:-

- (a) Increase level of investigation to enable the scale of the problems to be assessed
- (b) Review the soakaway design proposals and amend to minimise or eliminate the risks.

2.10.1 Chemical contamination

The adopting authority must ensure that in adopting a soakaway system the level of risk of a future groundwater pollution incident for which it will be liable is acceptable. It is therefore important that a soakaway system does not provide a pathway to connect a contamination source to a groundwater target so forming the source-pathway-target (SPT) chain to trigger further controls or remediation. It will be important to assess whether the site has a historical potential for contamination and to verify this initial assessment during later stages of investigation and construction. The potential for contamination from adjacent developments and landfills and pollutants such as oils from the road pavement can enter the soakaway system or the flow of water from the soakaway can leach or transport pollutants must also be considered.

2.10.2 Solution Features

Past experiences in Kent have shown that concentrated flows of water close to loosely infilled solution features in the Upper Chalk (eg plates 3 and 5) and Hythe Beds (eg plate 7) can result in surface settlements and instability which are costly to remediate as illustrated in plates 1,2 and 4. Stress relief associated with the formation of these features often results in fissure patterns locally directing water towards these features as illustrated in plate 7 and is often supported by more abundant orange iron-staining local to solution features as shown in plate 6. The adopting authority therefore needs to minimise these risks although to eliminate them would need the use of costly surface water outfalls or very deep soakaway liners which may not be acceptable to EA.

Understanding the geological formations in Kent, interrogation of databases, discussions with local authority/district engineers and other local knowledge from the general public can be invaluable in assessing the risks of solution features in a given location. The scale of the assessment is dependent upon the scale of the development and the perceived impact of activating a nearby solution feature. Examples of Risk Assessments in Figures 5 and 6 are described below.

Example 1: Solution features in Upper Chalk.

Figure 5 shows contours of the interface between the Upper Chalk and the overlying Clay with Flints at a site in east Kent based upon numerous boreholes on the site. Solution features were anticipated at Desk Study Stage as a result of the following:-

- (a) depressions visible in aerial photographs
- (b) location close to the edge of the Clay with Flints/Upper Chalk boundary on the geological map
- (c) an adjacent valley

Subsequent boreholes showed this assessment to be well founded.

Having assessed the risk and undertaken fieldwork to confirm the risk the soakaway design was reviewed. Whilst Upper Chalk was often encountered at shallow depths of the order of 4 mBGL the standard design philosophy for the site deep bored soakaways restricted the perforated sections between 15 and 25 mBGL although KCC's 1986 Technical memorandum M86/1 suggested that the perforated section could commence 2 metres into the chalk at higher elevations in the chalk. The decision for deeper soakage was taken as a result of the large number of solution features encountered to a depth generally no more than 12mBGL with the additional costs accepted by the Client, the deepened liners accepted by EA and the reduced maintenance liability gratefully accepted by the adopting authority. The risk was minimised but not totally eliminated as one feature elsewhere on the site had extended to 22mBGL. Perforated zones of all the liners extending below the deepest known solution feature would be unreasonably costly to the Client and pose an unacceptable risk of groundwater pollution to the EA.

Example 2: Washouts in Hythe Beds

Collapse settlements in the Hythe Beds are generally more dramatic as a result of the sand and silt sizes in the weakly cemented sandstone ("Hassock") being more readily transported by water into wide fissures in

the hard limestone (“Ragstone”). Solution features (“gulls”) in the Hythe Beds are commonly encountered in cambered valley sideslopes introducing widened vertical fissures and loose infill which can become unstable if subjected to concentrations of water.

Figure 2 shows records of subsidences in the Hythe Beds most of which are from published data that could be obtained by a Desk Study. Subsequent investigations have discovered “gull” features associated with some of these subsidences with the “gulls” generally trending parallel to the axis of the river valley at considerable offsets from the river. The solutions here are perhaps more complicated and involve a combination of deep bored soakaways and river outfalls to manage the risks subject to the approval of EA.

2.10.3 Impact on adjacent developments

Soakaway discharges must consider the impacts on adjacent structures, services and groundwaters. Typical impacts include the following:-

- (a) Chemical contamination migration - see 2.10.1
- (b) Activating adjacent solution features - see 2.10.2
- (c) Triggering slope instability - Desk Study including walkover
- (d) Washout induced settlements - Desk study to assess ground conditions
- (e) Flooding - carefully review storm return period

2.11 Site Investigation

2.11.1 General

Before any proposals are submitted for approval by the adopting authority, a geotechnical investigation should be carried out in order to assess a number of factors including the following:-

Outline Design

- (a) Depth to the water table/perched water tables (eg Published Well records)
- (b) Chemical contamination risks (eg OS Plans, Local Authority databases)
- (c) Suitability of strata for soakaway discharges (eg Geological Maps)
- (d) Consultation with EA

Detailed Design

- (e) Ground investigation to confirm expected ground conditions
- (f) Site specific soakage tests
- (g) Further consultation with EA (if requested by EA or if design proposals change)
- (h) Submission of design calculations and construction details to Local Authority for approval.

The site investigation shall be carried out in accordance with the Association of Geotechnical Specialists (AGS) Guidelines for Good Practice in Site Investigation. The scale of the investigations will be dependent upon the scale of the project but typically comprises a small Desk Study followed by ground investigation and/or proof testing of installations.

2.11.2 Desk Study

A Desk Study is beneficial for even small schemes with valuable data readily available from Well Records (items a and c in 2.11.1), published records (items a, b and c in 2.11.1), geological maps and memoirs (items a and c in 2.11.1) and aquifer protection maps (item d in 2.11.1). This literature search will help to optimise and accelerate the design and construction process (items e to h in 2.11.1). Archive records and records of the Chelsea Speleological Society may also enable the risk of tunnels, caves and deneholes to be assessed as collapse of such features can affect the integrity of nearby drainage systems and concentrated flows of water from soakaways can trigger the collapse of adjacent features.

2.11.3 Ground Investigation

Boreholes will enable the ground conditions to be confirmed and falling head tests to be performed. In some instances it may be required to prove the depth of the groundwater table and to then grout the lower part of the borehole to ensure that a safe unsaturated zone is preserved above the water table. Reuse of a borehole for a deep bored soakaway liner to save the cost of redrilling the hole at a later date will need to consider the risks of liner damage during subsequent chamber installation and the risk of smearing of the fissures during borehole progression, the latter risk probably requiring a soakage test on the installed liner to confirm discharge rates.

In some instances trial pit testing may be acceptable although the variable lithology of the near surface soils and generally unsuitable soakage potential suggest that their widespread use may be restricted (see section 2.2.3).

A separate chemical contamination survey may also be required dependent upon the results of the Desk Study.

2.11.4 Proof testing of installed soakaways.

In some instances a ground investigation can be replaced by proof testing of the soakaway system but this will require a greater level of Desk Study to have confidence in the ground and groundwater conditions beneath the site. Where borehole liners are installed the chamber will need to be designed or installed after the liner tests are undertaken or a chamber installed based upon conservative soakage rates confirmed by subsequent liner testing.

2.11.5 Examples of soakway site investigation guidance

- (a) Solution feature risks - desk study of available SI data, aerial photographs (drainage patterns/depressions), history of problems in area, adjacent valley features (contoured maps/site visit to assess risk of cambering), geological map (contacts between deposits, deepened channels of Head Deposits etc). Geophysics may be useful at some sites although the limitations of the techniques must be understood.
- (b) Underlying solution features/backfilled excavations - borehole at proposed soakaway location proving at least 3 metres of competent strata (ie not chalk fill, not isolated ragstone boulder, etc)
- (c) Adjacent solution features - in high risk areas a series of probes offset from the chamber (trial trenches may be better in areas where backfilled trenches are not a problem - ie no trenches in areas of roads, foundations etc)
- (d) Adjacent Backfilled quarries etc - Desk Study of old OS Plan, historical records and site investigation data to assess risks of adjacent man made excavations (liable to inundation induced settlements).
- (e) Groundwater level - Site Investigation data (piezometers, drillers records etc), Well Records
- (f) Soakage potential of competent strata - soakage tests at soakaway locations (rates can vary dramatically across the site). Shallow BRE 365 tests generally test unsuitable strata close to the ground surface (often have high washout potential, high variability etc)
- (g) Contamination Risks (underlying or adjacent sites) - Historical data (Maps, records etc), Site Investigation data, Environment Agency Records
- (h) Risk of tunnels, caves and denholes - Chelsea Speleological Society records and Local library records may assist in assessing the risk of encountering such features.

3. EVALUATION OF THE FIELD SOAKAGE RATE

3.1 General

The soakage capacity of any installation is predominantly dependent upon the soakage rate of the surrounding soil. Soakage testing can be implemented in two ways by either carrying out a constant head test or more commonly a falling head test.

The soakage rate used in the design of soakaways is normally evaluated by conducting small scale field tests. These are usually carried out by employing a light percussion or rotary auger rig to drill a borehole and expose the geological stratum to be tested. Water is then discharged into the borehole and its subsequent behaviour monitored, from which the soakage rate can be calculated.

The depth of the test pocket may also need to be monitored during the tests as collapse of all or part of the soakage test pocket in sands and gravels frequently occurs in Kent. Monitoring of the depth of the test pockets should be undertaken at the start of, the end of and at intervals during the tests in these soil types.

3.2 The Soakage Tests

3.2.1 Constant Head Test

Having drilled the borehole to the required depth a small section is left exposed, typically 1.5 to 2.0 metres in depth, by inserting a temporary steel casing to the top of the test zone. Clean water is placed into the borehole and its level in the casing maintained constant. The level chosen depends upon the geometry of the soakaway but should be related to the maximum driving head likely to be expected in the soakaway. This is discussed more fully in section 3.4.

The discharge of water into the borehole is measured by a flow meter and its value noted when the level is constant. From the geometrical shape of the borehole, exposed area and the discharge indicated by the flow meter, the soakage rate can be easily calculated. (See Figure 7).

3.2.2 Falling Head Test

In this case the test section is prepared as before but instead of maintaining a constant level of water it is allowed to fall, having terminated the inflow into the borehole. These tests are more frequently undertaken as they are more easily undertaken and will generally require a smaller volume of water than the constant head test.

The level of water is monitored with time typically for a period of 30 minutes or until it falls below the end of the casing. There may be occasions, however, when the level falls so slowly that the engineer may at his discretion stop the test.

The observations of level are converted to those of head by subtracting the water level from a datum set at the mid point of the exposed section. This information is plotted on a graph of Head vs. time. The method of calculating the soakage rate is rather more complex than for the constant head test but tends to be more flexible as will be demonstrated in section 3.4. The equation and geometrical definitions are shown on Figure 8.

The soakage rate is calculated by first assuming a value for the driving head which is related to the geometry of the soakaway.

A line is constructed tangent to the Head vs. time curve at the chosen value of head and continued until it intercepts both axes. The soakage rate is a function of the gradient produced by the tangent. The soakage rate obtained from these tests is normally expressed as litres/sec or litres/m²/min depending upon how the data is to be manipulated in the design of the soakaway system. It is important to try and standardise the units of the tests to avoid confusion in the subsequent design (eg If litres/m²/sec had been adopted and mis-interpreted as litres/m²/min this would result in a serious undercapacity in the system).

3.3 Variation with Area

The required exposed area will depend upon the peak flow arriving at the soakaway. However the equation balancing peak flow and the product of soakage rate and area is sometimes incorrect if the capacity of the chamber is not considered.

For conventional soakaways, the field tests provide rather an inadequate representation of the soakage because of the small test section available and the inevitable smearing that arises. The much greater area of the conventional soakage reveals a much larger number of fissures per unit area and limited amount of smearing is usually hand cleaned in any case. In service, therefore, the soakaway generally performs better than predicted by the field test.

The soakage rate can vary with depth due to the changing macrostructure of the stratum. Clearly a closely spaced open fissure system will enable the water to disperse into the surrounding soil more quickly than for a widely spaced, closed system. This variation may not occur predictably with depth. If any appreciable variation in soakage rate occurs throughout the length of the borehole then a summation technique must be employed. This is illustrated in the examples outlined in Appendix A.

While conventional soakaways are less prone to encountering this change it is quite possible that this will prevail in deep bored soakaways where the perforated section is at least 6 metres long. This poses a complication in predicting the soakage rate from field tests. Clearly in this situation it is erroneous to rely on one result. Several tests should be carried out in order to assess the variation with depth. The exact number will, of course, depend upon the anticipated depth of the borehole. A suggested maximum spacing is 4 metres. A minimum spacing of 1.5metres is recommended between the base of one test zone and the top of the next test zone to reduce the risk of smearing of the strata. The material immediately below a test pocket will be saturated and more easily smeared during subsequent drilling than material at greater depths.

3.4 Variation with Head

The discharge from a soakaway is not only affected by area it is substantially dependent upon the driving head forcing the water through the surrounding soil. This is detected in the falling head test by the curvature of the Head vs. time graph. If this is, in fact, linear, then the soakage rate would be independent of driving head.

The effect of head on the capacity is minimal for conventional soakaways and therefore is really only applicable to the design of deep bored soakaways. As the perforated section is increased the soakage area is enlarged but in addition the increased driving head generally produces an enhanced rate per unit area.

If the perviosity remains the same with depth the total soakage increases disproportionately due to the influence of the driving head. This can be predicted by again splitting the perforated section into smaller lengths and summing the individual rates.

3.5 Problems in Analysing the Soakage Rate

3.5.1 Saturated Stratum

When performing a soakage test the surrounding soil is charged with a relatively small volume of water compared with that when in service. So the test is conducted when the stratum is unsaturated. When the soakaway is subjected to the flows arising from a storm a much greater volume will accumulate and the stratum is likely to become saturated. This reduces the effective soakage rate because the water adjacent to the soakaway has to be forced into the outer regions of its sphere of influence. This should not be confused with difference in permeability between a saturated and unsaturated granular medium when the reverse could be true due to the formation of restrictive air bubbles in the soil skeleton.

It is therefore better to conduct two tests at the same section and use the second set of results for the design, particularly where significant reductions in soakage are expected following retesting (eg in sandy soils and for trench soakaways).

3.5.2 Insufficient Supply of Water

Conveniently, site investigation contractors obtain water from small 1000 litre bowsers which gravity feed it into the borehole. Although a standard means of supply it can provide unsatisfactory results when analysing the soakage rate.

It should be remembered that in order to conduct a field test it is necessary to at least partially fill the cased section of the borehole. In order to do this some of the bowser's capacity is will be used to provide this column of water before readings commence. The water entering the borehole will immediately start being absorbed into the surrounding soil and therefore a further volume of water will be lost in raising its level in the cased section before readings commence.

The volume contained in the casing is 31 litres and 18 litres for each metre length for a 200mm and 150mm diameter casing respectively. Consider, for example, a 15m length of borehole which is not untypical in Kent. The necessary storage volume for a 200mm diameter casing is 465 litres leaving only 515 remaining to be soaked while raising its level to 15 metres. Emptying the bowser of its contents normally takes 4-5 minutes and therefore, albeit crudely, the maximum soakage rate that can be directly recorded using this procedure is 103 litres/min (1.7litres/sec). Enlarged taps on the water bowser have been used to empty the water in 75 seconds which increases the maximum soakage rate that can be recorded from 1.7 l/s to 6.8 l/s

Alternatively, if the level of water barely rose above the bottom of the casing then a reasonable estimate of the discharge rate is 200 litres/min (33 litres/sec). Unfortunately, it is common to find the required soakage rate set by the engineer is well in excess of these values and as such these direct values are inadequate.

An analysis of the shape of Head vs. time can be used to evaluate greater values of soakage rate. For this it is assumed that the level of water does rise above the top of the perforated section and that at least a third of the sealed length is filled. It is recognised that flow is proportional to head and therefore it is reasonable to extrapolate the result that are recorded to establish the remainder of the curve as shown in Figure 9. This enables discharge rates to be calculated for greater heads than measured. This approach should be undertaken with caution and may require further on-site testing to demonstrate that the test result extrapolation is valid.

If the head does not rise above this one third mark then generally it will not be possible to accurately predict the soakage rates because a trend cannot usually be ascertained. A greater volume of water is required to properly conduct the test.

3.6 Proof Tests

It is usual to proof test a deep bored soakaway. However, a similar problem to that of the previous section will undoubtedly occur. The 1000 or 2000 litre bowser will not be sufficient to assess the full potential of the soakaway. This poses a problem in that the capacity cannot be directly measured and recourse must be made to the procedure previously stated.

Where a greater volume is required to establish the maximum soakage rate this is best achieved by obtaining several bowzers and pump the water into the borehole via a flow meter or by using a gully emptier. Either of these options will be more expensive to the client. For example a (2000gallon) gully emptier or (4000gallon) tanker will cost approximately £250 to £300 for half a day at 2000 prices.

3.7 Factor of Safety

The value of soakage rate is normally factored to account for the 'silting-up' of the fissures in the adjacent soil. This is done to enable the soakaway to function at a design rate for a period of time rather than quote a maximum which will deteriorate immediately.

The value chosen is normally 2 although the actual choice will depend upon the environment in which the soakaway is constructed. Clearly, a soakaway in the country will attract a greater quantity of silt than that likely to occur in the town. Consequently the life expectancy is lower in the country than the town for a similar soakaway unless a greater factor of safety is applied.

4. METHODS OF DESIGNING DRAINAGE SYSTEMS

4.1 General

Although this guide does not intentionally deal with designing the drainage system it is essential to understand the principles involved as they are heavily reflected in the analysis of the soakaway. After all the soakaway is the last link in the chain and as such is very much dependent on what goes before.

Three methods that are currently used to design drainage systems will be considered in this chapter although two are in fact similar, albeit only in procedure. The end result in all three means the calculation of a peak flow rate which can be compared with design tables to assess whether a particular pipe size is suitable for that location. The peak flow rate entering the soakaway is clearly of great importance in determining its size

4.2 The Rational or Lloyd-Davies Method

This method is described in great detail in Road Note 35(3). The Rational method has generally been employed in the past and has been found suitable for use in estate roads only where the maximum pipe size is 600mm. This is presumably why its use has been incorporated in the Kent Design Guide. The method basically evaluates the peak flow rate from a simple equation involving its dependency upon rainfall and catchment area. Obviously the greater the area and/or rainfall the larger the peak flow rate. This equation is reproduced below:

$$Q_p = \frac{A_p \times i}{0.360} = 2.78 A_p \times i \dots\dots\dots$$

4.1

where Q_p = peak flow rate (l/s)

A_p = catchment area (hectares)

i = rainfall intensity (mm/hour)

The peak flow rate given in this equation should be regarded as that entering the system from the catchment area at that point. Accumulations may arise where one or more other branches join the network.

The appraisal of the catchment area is straightforward although some complication will be added to the method if permeable regions are also considered. The major variable is that of rainfall intensity as this will change with geographical location, storm frequency and duration. Caution must be exercised, however, when using the term duration.

For a given frequency and duration the Meteorological Office at Bracknell (0845-3000300 or 01344 420242) can supply rainfall figures for any particular area in Kent.

It is perhaps interesting to note that for Kent the rainfall figures do not widely differ and indeed are not significantly different from Table 7 in Road Note 35 which has been quoted for Crowthorne. Table 7 is reproduced in Figure 10. From this it can be seen that for increasing duration the rainfall decreases. This perhaps requires some clarification. It should be remembered that the Meteorological Office collects data from rainfall gauges which are simple storage canisters. They have monitored the amount of water collected with time and produce figures like those in table 7 by dividing the height of water collected by the time. If the observations are considered over the cycle of a storm then the water accumulates much faster during the intense zone around the peak than for the periods of rise and decay. But the figures in table 7 are averages based on successive time periods. It is therefore not difficult to reason why with increasing duration the average value of rainfall drops.

The choice of frequency and duration are a little complicated and merit a section by themselves (see Sections 5.2 and 5.3).

Having established these parameters the peak flow is readily calculated. The procedure for calculating the pipe sizes is the same for the next method, but this is outside the scope of this guide.

Kent Design, for small developments such as cul-de-sacs, further simplifies the approach by stating that the rainfall intensity shall be 50mm/hour.

The equation for predicting the flow rate is reduced to:

$$Q_p = 0.015 A_p \dots\dots\dots 4.2$$

where A_p = Catchment Area (m^2)

Q_p = peak flow (litres/sec)

4.3 The Wallingford Procedure or Modified Rational Method

This analysis takes the form of the Rational Method in that the basic equation is used but modified by a dimensionless coefficient, C. This in turn is formed from the product of two separate coefficients as given below.

$$C = C_v C_r \dots\dots\dots 4.3$$

where C_v = the volumetric runoff coefficient

C_r = dimensionless routing coefficient

The volumetric runoff coefficient is simply the proportion of the rainfall on the catchment which appears as surface runoff in the drainage system. As such the ratio is dependent upon the permeability of the catchment which for rapidly draining soils is 0.6 and for clays is 0.9. Where a paved area is concerned such as a highway it would not, perhaps, be unreasonable to assume 1.0.

From an examination of the data collected by the Hydraulics Research Station a value of 1.3 is recommended for C_r . This routing coefficient depends on the shape of the time-area diagram and on the variation of rainfall within the time of concentrations.

The basic equation may therefore be re-written:

$$Q_p = 2.78 \times 1.30 \times C_v \times i \times A_p \dots\dots\dots 4.4$$

The rainfall intensity is evaluated by a totally different method than before and is described in detail in the Modified Rational Method's design guide. The value is obtained by using maps of rainfall intensity which are subsequently altered by coefficients reflecting storm frequency and duration.

4.4 The TRL Hydrograph Method

The Transport Research Laboratory's hydrograph method employs yet another different means of establishing the peak flow rate. This method is somewhat more rigorous than the previous ones and usually involves the use of a computer program. Basically the method considers an initial rainfall profile which falls upon a catchment area. Like the Wallingford approach the permeability is accounted for in a similar manner. This rainfall profile is converted to a hydrograph which is modified by the geometry and gradient of the pipe network. The peak flows are automatically compared with a set of tables describing the parameters of the pipes and the choice of pipe is given in the output. The program is consequently a complete design tool that requires little or no calculation on the part of the user. A useful feature of the program is that it can predict the hydrograph at any point on the network and therefore the flow rates arriving at, say, a soakaway can be plotted with time.

The use of the TRL/HCSL approved computer program HYDRAN in the 1987 document was far more flexible than the previous method and for these reasons was utilised in providing the basic design charts which were published in M86/1 issued in June 1986. These charts have been reproduced in this design guide Figures 13, 14, 16 and 17 given later.

4.5 Limitations to the Methods

It is accepted that the Rational Method is the most erroneous, but all three methods suffer from inaccuracies in the prediction of the actual volumes passing through a drainage system. Indeed only the HYDRAN method offered a possible solution by manually integrating the area beneath the hydrograph. The other two only compute the peak flow.

The HYDRAN method suffers further in that the hydrograph is a simplified model of an actual storm which in reality can be appreciably different.

5. DETERMINATION OF PEAK FLOW RATE

5.1 General

As previously mentioned it is the peak flow rate which governs the choice of pipe size and to some extent the design of the soakaway outfall. However, the peak flow rate is sensitive to a number of variables, not the least of which is the catchment area. This, of course, is self evident and no further comments will be made.

The peak flow rate is dependent upon the following parameters.

- (i) Storm frequency
- (ii) Duration
- (iii) Geometry of pipe network
- (iv) Gradient of pipes

Other minor effects exert some influence, but can be disregarded for the purpose of designing soakaways. While the first two items can be dealt with some success by the Rational and Modified Rational Methods it is with some difficulty that they can assess the effects due to the latter two. The program HYDRAN requires that all four form part of the input and consequently the individuality can be readily analysed.

5.2 Storm Frequency

The rainfall intensity depends upon the frequency of the storm. A low intensity rainfall returns at frequent intervals, for example 1 year, while greater rainfalls do not appear so often. So the rainfall intensity increases with storm frequency.

Road Note 35 assesses the choice of storm frequency, or a return period as it is otherwise known, in the following way:-

“Suitable design frequencies range from once per year for separate surface-water sewers in modern estates or roads, to once per 50 or 100 years for combined sewers in old developments with basements. The factors that must be taken into account in estimating the seriousness of a given frequency of flooding are always local in character and cannot be determined by a general formula. For example, at one area a given volume of floodwater might merely hinder traffic for a short period, but at another it would cause serious harm by flooding property, particularly where the floodwater contains foul sewerage”.

For schemes to be adopted by Kent County Council it would appear that drainage systems for roads are designed with frequencies of either 2 or 5 years. Storm return periods of 1 year, 10year and 50 years may be required on a scheme specific basis.

The procedure outlined in this Design Guide provides design guidance for storm frequencies of 2 and 5 years. If the design of a system is required outside these storm frequencies, specialist advice should be sought.

5.3 Duration

Duration is the subject of some misconception. It has been noted that in some cases the duration used in a design is more or less arbitrarily set at 10 minutes for a 2 year storm and 20 minutes for 5 year storm. The misconception is that duration is not the length of the storm. As can be seen from Figure 10, the rainfall intensity is very sensitive to duration for a given storm frequency and consequently its choice is crucial.

Road Note 35 states:-

“For use with 'rational' (Lloyd-Davies) formula it is appropriate to use a mean rate of rainfall during a storm. The duration of the storm should be taken as being equal to the time of concentration of the drainage area to the point for which the calculation is being made. The time of concentration is simply the sum of the time of entry (normally 2 minutes for roads) and the travel time along the drainage system to that point. The conclusion to be drawn from this is that the durations for most pipe lengths are 3 minutes or less. The use of 10 minutes and 20 minutes are highly suspect which can lead to a severe under design”.

The use of the concept of duration is irrelevant to the hydrograph method as it evaluates its own duration in the computer programme. It has been necessary to include this section in the guide as the term may occur in discussions with the client.

5.4 Geometry of the Pipe Network

It would seem obvious that the geometry of the pipe network would seriously affect the outfall hydrograph. Indeed it does when the network is a sprawling spider's web. However, such a occurrence is rare because several soakaways would serve such an area and the complexity of the network would be reduced.

As a comparison the effect on the peak flow rate of serving the catchment with two pipes instead of one, as might occur in a road, was quoted as a difference of approximately 7% in M86/1. This is graphically illustrated in Figure 11. Here, the reduction is compared with the rainfall hydrograph. This particular case is for a very large intensity storm of frequency well in excess of 20 years over an area of 1000m².

5.5 Gradient of the Catchment and Pipe Network

The gradient of the pipe network affects the time of concentration in that water travelling along a shallow gradient pipe takes appreciably longer than in a steeper gradient. The effect of this is to attenuate the peak flow rate with decreasing gradient.

This is illustrated in Figure 12 where the peak flow is related to the gradient of the pipe network. As can be seen the peak flow rate decreases with decreasing gradient, but not linearly. The rate of decrease rapidly diminishes. Indeed, for small areas, that is no attenuation and conversely for large areas the attenuation occurs over a significant range of gradients.

For most design purposes the gradient of the network reflects the gradients of the topography and although the pipe geometry may be unknown a reasonable estimate can usually be made from the contours. There is of course a limit in that the pipe requires the velocity to be self cleansing and a minimum gradient of about 1 in 200 is normally used. A further complication lies in the time of entry. For normal roads this is 2 minutes. However, wide areas of catchment would naturally increase this time.

Road Note 35 states:-

“It is recommended that a time of entry of two minutes should be used for normal urban area, increasing up to four minutes for areas with exceptionally large paved surfaces with slack gradients”.

It further adds that a change in the entry would affect the peak flow rate by only 15%.

5.6 Impermeability of the Catchment Area

In considering the design of a sewer system for a highway scheme the catchment area can be regarded as impermeable. This is not the case when the catchment area includes portions of field and garden and certainly is not the case when assessing the drainage of valleys.

The permeability of the underlying ground is therefore important in determining the percentage of water reaching the drainage system. It is interesting to note, however, that the Hydraulics Research Station consider that clayey sub-soils only marginally influence the permeability factor reducing it from unity to 0.9. Sandy or gravelly sub-soils will to some degree decrease this factor as discussed in section 4.3.

The charts forming part of this Design Guide are based upon a permeability factor of unity.

5.7 Comparison of the Three Methods

The following presents a comparison of the methods for predicting the peak flow rate for the same case. The case consists of the following:-

- (a) Area (A_p) = 1000m²
- (b) Return Period = 2 years
- (c) Network Gradient = 1 in 60
- (d) Network = Single pipe
- (e) Time of concentration = 3 minutes

The time of concentration and hence duration used in the rational and modified rational method was obtained from the output of the HYDRAN computer program.

<u>Method</u>	<u>Peak Flow Rate (l/s)</u>
TRL Hydrograph	22
Rational Method	23
Wallingford	41

It is interesting to note that the minimum duration which is used in the Wallingford procedure is 5 minutes and as such the flow rate calculation predicts it to be higher than it should. The Wallingford procedure is more favourable for durations in excess of 5 minutes. Although the Rational Method is insensitive to changes in gradient or roughness coefficient, simply because they are not directly accounted for, the error is not significantly increased by considering different values for these parameters.

5.8 Design Charts for Calculating Peak Flows

The computer program HYDRAN has been used to construct design charts to enable an assessment to be made of the peak flow arriving at a soakaway. These design charts cater for the major variables affecting the value of peak flow. These are as follows:

- (i) Area (m²)
- (ii) Return period
- (iii) Gradient of Pipe network

There are two charts, one each for a 2 and 5 year return period (Figures 13 and 14 respectively). The peak flow is estimated by taking the catchment area and pipe gradient to form an ordinant on the y axis.

6. THE EVALUATION OF A SOAKAWAY'S CAPACITY

6.1 The Effect of Storage Capacity

The design of a soakaway must include both aspects of storage volume and storage rate. While a peak flow rate may enter the soakaway, as predicted by the hydrograph method, this does not necessarily have to be catered for by the soakage into the surrounding soil strata. In fact, it is uneconomical to do so. It is clear that in considering the storage volume of the chamber with the required soakage rate being less than the peak flow volume of the chamber the soakage rate will be less than the peak flow rate. By how much less will depend upon a number of factors. Therefore a balancing equation can be compiled to account for both aspects. The form of this equation is simple as shown below.

$$\text{Total volume} = \text{storage volume} + [\text{soakage rate} \times \text{time}] \dots\dots\dots 6.1$$

While being simple it is inaccurate to use directly. The major problem is that it would assume the storm be of uniform intensity rather than rising to a peak and decaying symmetrically with time.

Even if the second term could be predicted with some accuracy, it is not possible, as yet, to accurately assess the total volume of water entering the soakaway. Certainly this is true with the Wallingford Procedure. The Rational Approach and the HYDRAN approach does enable the user to evaluate the total water reaching the soakaway but not with any accuracy although the values of peak flow rate are reliable,

As will be demonstrated in Section 6.3 equation 6.1 can be replaced by treating the affect of the storage volume in a different way.

6.2 Calculation of Storage Volume

For the proposed method the storage volume will still need to be calculated. Storage chambers are usually constructed from precast concrete rings having standard diameters. In reality the smallest normally chosen is 1.8 metres although up to 3.0 metres can be obtained (in intervals of 300mm).

The volume of the chamber is readily calculated from equation 6.2.

$$\text{Storage volume} = \frac{\pi d^2}{4} \times \text{effective height} \dots\dots\dots 6.2$$

The term 'effective height' is the depth of the chamber less the depth to the inlet pipe's invert and for the case of a deep bore soakaway the height from the top of the syphon. In the standard details in Appendix D2 the syphon head height is 1.0metres above the base of the chamber. For design purposes the invert pipe depth should be at least 1 metre although this may be increased in light of actual drainage information. the design process specifically discourages backing up of water in the inlet pipes in the calculations and therefore a knowledge of the proposed invert levels is essential.

6.3 Reduction of the Peak Flow

The modification of the peak flow rate to account for storage volume is given by the following equation.

$$\text{Required Soakage Rate} = \text{Peak flow rate/reduction factor} \dots\dots\dots 6.3$$

The reduction factor is evaluated by considering the volume of water beneath the inflow hydrograph symmetrical about the peak. Although it has been stated that the total volume predicted by the hydrograph method is not accurate it is perhaps reasonable to assume that the volume about peak is appreciably less erroneous. It should be remembered that the peak flows are calculated with reasonable accuracy.

The reduction factor is simply the ratio between the flow rate given by the hydrograph (taking the storage into account) and the peak flow rate. This is graphically illustrated in Figure 15.

The Reduction Factor RF is given by the equation

$$\text{RF} = Q_p/Q \dots\dots\dots 6.4$$

6.4 Reduction Factors

A series of reduction factors have already been calculated for a given set of conditions which will enable their use without recourse to the original hydrograph. These are reproduced on two graphs outlined on Figures 16 and 17. The reduction factor is predominantly dependent upon two variables, that of catchment area and the return period of the storm. Both of these affect the peak flow rate in the same way. Larger catchment areas and greater return periods increase the peak flow rate and in doing so raise the level of the adjusted flow rate taking storage value into account.

The effect of network geometry and pipe roughness also influence the magnitude of the peak flow rate but to a much lesser extent. It is reasonable to ignore the slight differences brought about by these minor effects and produce design charts which concentrate on the major variables. With this in mind Figures 16 and 17 provide reduction factors for different storage volumes for a limited number of catchment areas for a two and five year return period respectively. For areas other than those depicted upon the graphs interpolation between the curves is sufficiently accurate.

The required soakage rate necessary from the surrounding soil is simply the peak flow rate obtained from Figures 13 and 14 divided by the relevant reduction factor.

7. SUMMARY OF DESIGN METHOD

7.1 Variability of Design Information

The method of design of a soakaway will initially depend upon the information supplied by the client. Clearly there is a significant difference in approach if the Engineer supplies either the peak flow rate or merely states he wants a soakaway installed into which he can discharge an unspecified amount of water. It is therefore necessary to consider a number of different initial conditions.

Case 1 Engineer supplies the soakage rate he requires (the storage volume adjusted flow rate)

For the condition no other information is necessary as the Engineer has calculated the soakage rate.

Case 2 Engineer supplies the peak flow rate.

In situations where the Engineer has completed a surface water drainage design, typically in the case of a proposed new road scheme, the peak flow rate will be known.

Known parameters	(i) Catchment Area
	(ii) Return Period
Unknown parameters	(iii) Storage volume

This in fact may be a known parameter especially if the storage chamber is already in existence.

Case 3 Engineer supplies only the area to be drained.

The additional information necessary is as follows:

(i)	Return Period
(ii)	Storage Volume
(iii)	Gradient

Case 4 Engineer requests a soakaway to be installed but offers no other information.

The information required is as follows:

(i)	Catchment Area
(ii)	Return Period
(iii)	Storage Volume
(iv)	Gradient

In deciding the additional variables necessary to complete the design it will be essential to seek the approval of the engineer in selecting this information. It may perhaps be a process of discussion between the two parties from which will eventually evolve these parameters. As case 4 involves the entire design process the summary for this will be outlined first. The remaining conditions utilise similar segments of the analysis and as such will be very much abbreviated. For case 3 the evaluation of the catchment size has already been completed by the engineer and therefore the design procedure adopted in steps 7.2.2 to 7.2.7 for Case 4 will be followed. For case 2 steps 7.2.1 to 7.2.4 steps have been completed by the Engineer and so the design procedure adopted in steps 7.2.5 to 7.2.7 for Case 4 will be followed. Further worked examples are contained in Appendix A.

7.2 Worked example (Case 4)

7.2.1 . Evaluation of Catchment Area (At)

This is best achieved from considering a plan of the area. For the case of highway drainage it may involve the entire area as might result from a new estate road. Alternatively, where a flooding problem needs to be resolved the watersheds may require determining on site before measuring the area on plan.

7.2.2. Return Period

The return period is normally chosen from either 2 or 5 years the exact value be governed by local conditions. (See section 5.2 - storm frequency).

7.2.3. Gradient of the Pipe Network

As stated in Section 5.3 this is likely to reflect the topographical gradient and that may be necessary to consider the longitudinal profile of the road. Where such a profile does not exist it may not be necessary to conduct a string of levels on site. If the area is flat the sensitivity to change in gradient is small if less

than 1 in 60. This is true for most catchment areas. A simple visual estimate of slope may only be necessary. However it is important to remember that there is usually a minimum gradient which can be employed in a drainage system to ensure that self cleansing velocities are maintained. This is usually in the order of 1 in 200.

7.2.4. Peak Flow Rate

Using the above three pieces of information the peak flow rate can be calculated from figures 13 and 14 depending upon the return period.

7.2.5. Storage Volume

The evaluation of storage volume is most significant when the soakage rate is poor and the system relies on as much stored water as that can be made available. It should be remembered that the volume can be increased without excavating deeper simply by using a larger diameter ring or at the extreme by constructing a 'one-off' rectangular chamber.

7.2.6. Required Flow Rate

The required soakage rate is determined by obtaining the reduction factor RF from either figure 16 or 17 and dividing this into the peak flow rate (Q_p).

Required soakage rate = Q_p/RF

7.2.7. Calculation of Perforated Area

The required soakage rate is then compared with the actual values assessed by the field tests and the perforated area calculated.

8. REFERENCES

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APPENDIX A

Worked Examples

APPENDIX A - EXAMPLES

The following examples will illustrate the complete procedure.

Example 1 - Conventional Soakaway

A new estate road is to be constructed to serve a proposed housing development. The new carriageway is 5 metres wide and 200m long lying on relatively level ground. It can be assumed that the consequences of flooding is minor and that a 2 year return period is adequate. The surface run off is to be discharged into a conventional soakaway (chalk was encountered from the ground level). The unit soakage was established by the field test to be 40 litres/m²/min.

1. The Catchment Area (A_c) is $5 \times 200 = 1000\text{m}^2$
2. The return period is set at 2 years.
3. The gradient of the pipe network may in all probability be disregarded.
4. From Figure 13 the peak flow rate is $0.022 \text{ m}^3/\text{sec}$ (a limit of 1 in 60 has been chosen for the pipe gradient)
5. The storage volume is unknown so consider a number of alternatives in terms of depth based upon a 1.8m. diameter ring. Choice of depths being 3, 4, 5 and 6 metres. From these depths it is assumed that the top 1 metre cannot be considered due to the invert of the inlet pipe. The storage volumes are as follows:

Soakaway Depth (mBGL)	Effective Soakaway Depth (m)	Effective Storage Volume (m ³)
3	2	5.09
4	3	7.63
5	4	10.18
6	5	12.72

6. From Figure 16 the following reduction factors are evaluated. These are utilised to reduce the peak flow rate to an actual discharge rate required from the soakaway.

Effective Storage Volume (m ³)	Reduction Factor	Required Discharge rate (l/s)
5.09	3.9	$22/3.9 = 5.64$
7.63	7.0	$22/7 = 3.14$
10.18	12.0	$22/12 = 1.84$
12.72	22.8	$22/22.8 = 0.96$

7. The rate required from the unit can now be compared with the field test to evaluate the required configuration/depth. The field test gives $40 \text{ l/m}^2/\text{min}$ which should be factored by 2 to provide a design rate of $20 \text{ l/m}^2/\text{min}$.
8. The following table can thus be produced to provide the relationship between the required and achieved discharge rate.

Soakaway Depth (mBGL)	Effective Storage Volume (m ³)	Soakage Depth (m)	Soakage Area Provided (m ²)	Discharge Rate Provided * (l/s)	Required Discharge rate (l/s)
3	5.09	2	11.3	3.76	5.64
4	7.63	3	17.0	5.67	3.14
5	10.19	4	22.0	7.53	1.84
6	12.72	5	28.3	9.43	0.96

Note * - Discharge rate provided = soakage area x design soakage rate. The design soakage rate is in litres/m²/minute and will need to be divided by 60 to obtain discharge values in units of seconds (e.g. discharge rate provided = $11.3 \times 20/60 = 3.76 \text{ l/sec}$).

9. From the above table it can be seen that the 3.0 metre deep soakaway is insufficient in that despite there being a discharge rate of 3.76 l/s available from the unit, the required discharge from this size chamber is 5.64 l/s. It will therefore be necessary to install 4.0 metre deep soakaway. Refining the calculations shows that the use of a 3.5metre deep soakaway will be adequate.

Example 2 - Linked Conventional Soakaways

The developer of the estate in Example 1 has been restricted to soakaways to a depth of 3 metres as a result of groundwater protection concerns and with limited space available on the site is considering conventional soakaway options including a line of small diameter soakaways constructed at 5 metre spacings.

1. Reiterating the information obtained in the earlier example.
 - (a) Area $A_t = 1000\text{m}^2$
 - (b) Return Period = 2 years
 - (c) $Q_p = 0.022\text{m}^3/\text{sec} = 22\text{litres/sec}$ (figure 14)
2. The storage volume is unknown so consider a range of chamber diameters from 1.8 metres to 3.0metres. From these depths it is assumed that the top 1 metre cannot be considered due to the invert of the inlet pipe. The storage volumes and required discharge rates (Figure 16) are as follows:-

Soakaway diameter (m)	Soakaway Depth (mBGL)	Effective Soakaway Depth (m)	Effective Storage Volume (m^3)	Reduction Factor	Required Discharge rate (l/s)
1.8	3	2	5.09	3.9	5.64
2.1	3	2	6.93	6.5	3.38
2.4	3	2	9.05	8.7	2.53
2.7	3	2	11.45	17	1.29
3.0	3	2	14.14	33	0.67

3. The rate required from the unit can now be compared with the field test to evaluate the required configuration/depth. The field test gives $40\text{ l/m}^2/\text{min}$ which should be factored by 2 to provide a design rate of $20\text{ l/m}^2/\text{min}$.
4. The following table can thus be produced to provide the relationship between the required and achieved discharge rate.

Soakaway Depth (mBGL)	Soakaway diameter (m)	Effective Storage Volume (m^3)	Soakage Area Provided (m^2)	Discharge Rate Provided * (l/s)	Required Discharge rate (l/s)
3	1.8	5.09	11.31	3.76	5.64
3	2.1	6.93	13.19	4.40	3.38
3	2.4	9.04	15.08	5.03	2.53
3	2.7	11.45	16.96	5.65	1.29
3	3.0	14.14	18.85	6.28	0.67

Note * - Discharge rate provided = soakage area x design soakage rate (convert units of time to seconds).

5. From the above table it can be seen that the 1.8 metre diameter soakaway is insufficient in that despite there being a discharge rate of 3.76 l/s available from the unit, the required discharge from this size chamber is 5.64 l/s . It will therefore be necessary to install a 2.1 metre diameter soakaway.
6. The alternative solution is to provide a series of smaller diameter closely spaced linked 1.5 or 1.8 metre diameter soakaways with the table below showing that 2No 1.5metre diameter linked soakaways will be adequate. The cost of the additional chambers and pipework will need to be reviewed in relation to the buildability and working area available for a single larger diameter chamber.

Soakaway Depth (mBGL)	Soakaway diameter (m)	Number of chambers	Effective Soakage Area Provided (m^2)	Discharge Rate Provided (l/s)	Effective Storage Volume (m^3)	Reduction factor	Required Discharge rate (l/s)
3	1.5	1	9.42	3.14	3.53	2.5	8.80
3	1.5	2	14.14*	4.71	7.07	6.3	3.49
3	1.5	3	18.85*	6.28	10.60	13.2	1.67
3	1.8	1	11.3	3.76	5.09	3.9	5.64
3	1.8	2	17.0*	5.64	10.18	12.1	1.81
3	1.8	3	22.6*	7.52	15.27	40	0.55

Notes * - ignore 25% of surface area of each chamber for adjacent linked soakaway due to the potential interaction(For 3No chambers the central soakaway will lose 25% to both the adjacent soakaways so that only 50% of the area will be available)

Example 3 - Deep Bored Soakaway

The same estate is to be sited upon 8 metres of Thanet Beds overlying chalk. Design a suitable deep bored soakaway having a liner diameter of 100mm in a 150mm diameter borehole. The Soakage Test results are as follows:-

Test1: exposure 12 to 14mBGL		Test 2: exposure 18 to 20mBGL	
Head (m)	Field Soakage Rate (l/m ² /min)	Head (m)	Field Soakage Rate (l/m ² /min)
2	5	3	10
3	10	5	20
5	15	8	30
7	20	11	40
9	25	14	50
11	30	17	60
13	42		
15	54		

- Reiterating the information obtained in the previous example.
 - Area $A_t = 1000\text{m}^2$
 - Return Period = 2 years
 - $Q_p = 0.022\text{m}^3/\text{sec} = 22\text{litres/sec}$ (Figure 14)

In this case however the effective depth of the soakaway unit is reduced by 2.0 metres. It is assumed, as before, that the upper 1.0 metre cannot be considered due to the invert of the inlet pipe and the lower 1.0 metre above the soakaway base is also ignored as a result of the syphon head upstand. The required discharge rates using the Reduction Factors in Figure 17 are as follows:-

Soakaway Depth (mBGL)	Effective Soakaway Depth (m)	Effective Storage Volume (m ³)	Reduction Factor	Required Discharge rate (l/s)
3	1	2.54	1.1	20.0
4	2	5.09	3.9	5.64
5	3	7.63	7.0	3.14
6	4	10.18	12.0	1.84

- To establish the correct length of soakaway liner involves a process of trial and error. However, first, the underlying chalk should be divided into zones of perviosity. Assume first set of results applies between 8 and 18 metres below ground level (mBGL) and the second set below 18 mBGL.
- The perforated section of the liner should not commence say above 12 metres below ground level. For convenience consider depths of 15, 18, 21 and 24 metres. The second set of soakage test results would suggest that there is a linear increase in soakage rate with increasing head and therefore it would be reasonable to extrapolate the results beyond the head = 17m value.

Head (m)	Extrapolated Field Soakage Rate (l/m ² /min)
20	70
23	80

- The soakage areas generated by the length of perforated section are as follows:

Depth to base of liner (mBGL)	Perforated Length (m)	Soakage Area* (m ²)
15	3	1.41
18	6	2.83
21	9	4.24
24	12	5.66

Note * - this relates to borehole diameter and not liner diameter (eg $1.41\text{m}^2 = \pi \times 0.15\text{m} \times 3\text{m}$)

5. Soakage obtained by these lengths of liner are given below. It is important to remember that for each section the soakage rate is different. It should be remembered that the unit soakage rate has been factored by 2 and the head has been evaluated by taking the depth to the centre of the soakage section less 1 metre to allow for the invert pipe

For example for a liner to 15mBGL relates to an exposed section 12 to 15mBGL (centre 13.5mBGL), a head of 12.5metres (13.5mBGL - 1.0m for pipe inlet invert), an extrapolated field soakage rate of 39litres/m²/min (Pro rata between 30 l/m²/min for head of 11m and 42 l/m²/min for head of 13m) giving a design soakage rate of 19.5 l/m²/min (Field rate divided by a factor of Safety of 2).

Depth to base of liner (mBGL)	Increment of Discharge (mBGL)	Maximum driving Head (m)	Area of exposed chalk * (m ²)	Unit Field Soakage rate (l/m ² /min)	Unit Design Soakage rate (l/m ² /min)	Increments of available discharge (l/min)	Total available discharge (l/min)	Total available discharge (l/sec)
15	12 to 15	12.5	1.41	39 ♠	19.5	27.5	27.5	0.46
18	12 to 15 15 to 18	12.5 15.5	1.41 1.41	39 ♠ 57 ♠	19.5 28.5	27.5 40.2	67.7	1.13
21	12 to 15 15 to 18 18 to 21	12.5 15.5 18.5	1.41 1.41 1.41	39 ♠ 57 ♠ 65 ♥	19.5 28.5 32.5	27.5 40.2 45.8	113.5	1.89
24	12 to 15 15 to 18 18 to 21 21 to 24	12.5 15.5 18.5 21.5	1.41 1.41 1.41 1.41	39 ♠ 57 ♠ 65 ♥ 75 ♥	19.3 28.5 32.5 37.5	27.5 40.2 45.8 52.9	166.4	2.77

Note * - ignore 1 metre of chalk exposed between base of bentonite seal and top of perforated section

- Areas have been divided and related to their respective unit soakage rates.

- For depths of upto 18mBGL use test 1(♠). For liner depth below 18mBGL use Test 2(♥)

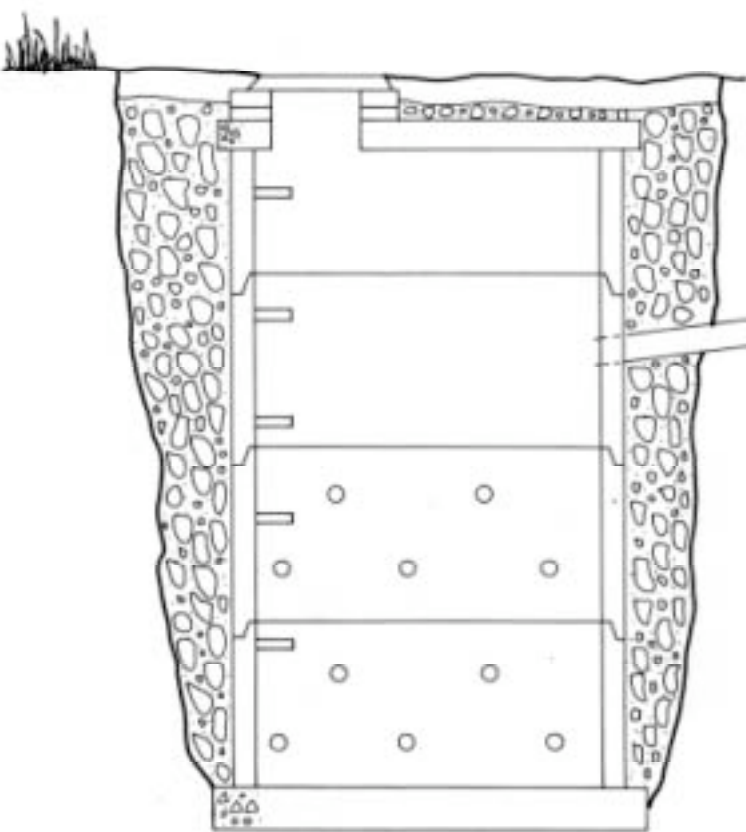

These storage results can now be combined with the storage volumes and checked against the required soakage rate as summarised in the following table.

Soakaway Depth (mBGL)	Effective Storage Volume (m ³)	Required Discharge rate (l/s)	Depth to base of liner (mBGL)	Total available discharge (l/sec)
3	2.54	20.0	15 18 21 24	0.46 1.13 1.89 2.77
4	5.09	5.64	15 18 21 24	0.46 1.13 1.89 2.77
5	7.63	3.14	15 18 21 24	0.46 1.13 1.89 2.77
6	10.18	1.84	15 18 21 24	0.46 1.13 1.89 2.77

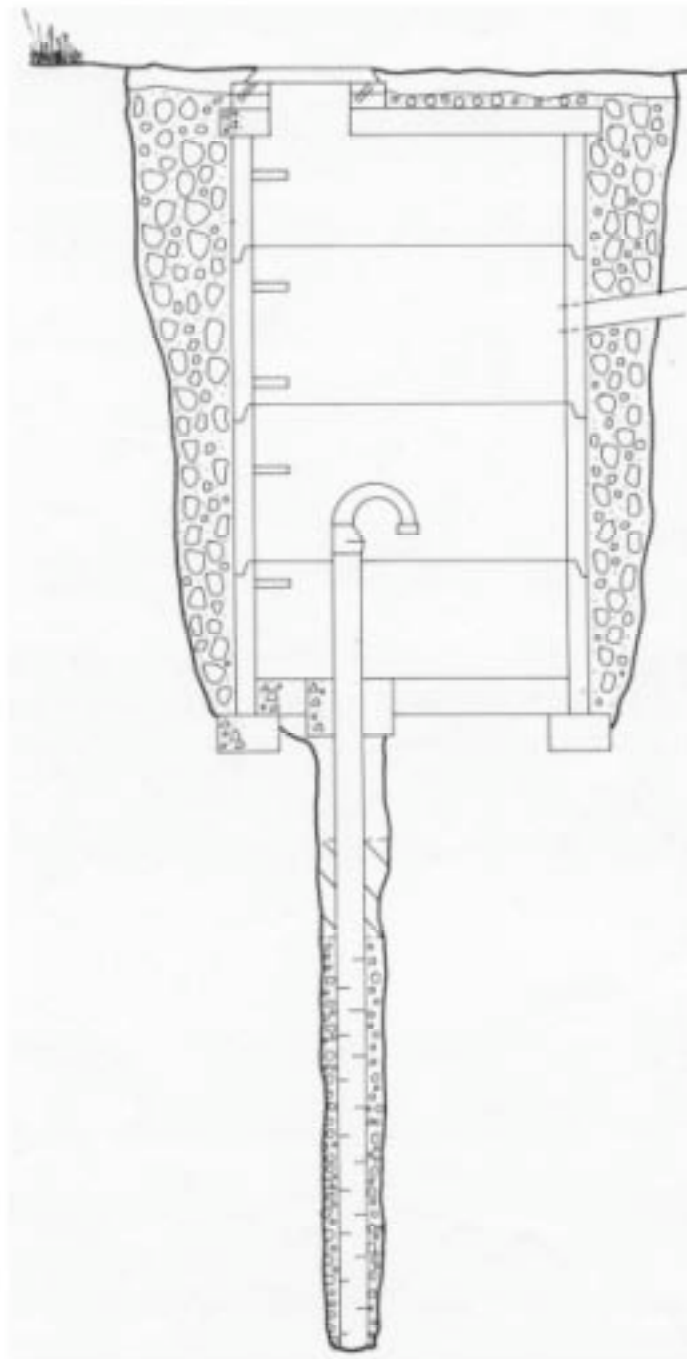
From these results the soakaway chamber should be 6 metres deep and the liner installed to 21 metres below ground level. However, by increasing the perforated section it will be possible to reduce the depth of the chamber. By inspection a 5m deep chamber could be used if the perforated section was increased by 3 metres, that to 27 metres below ground level.

APPENDIX B

Figures and Drawings

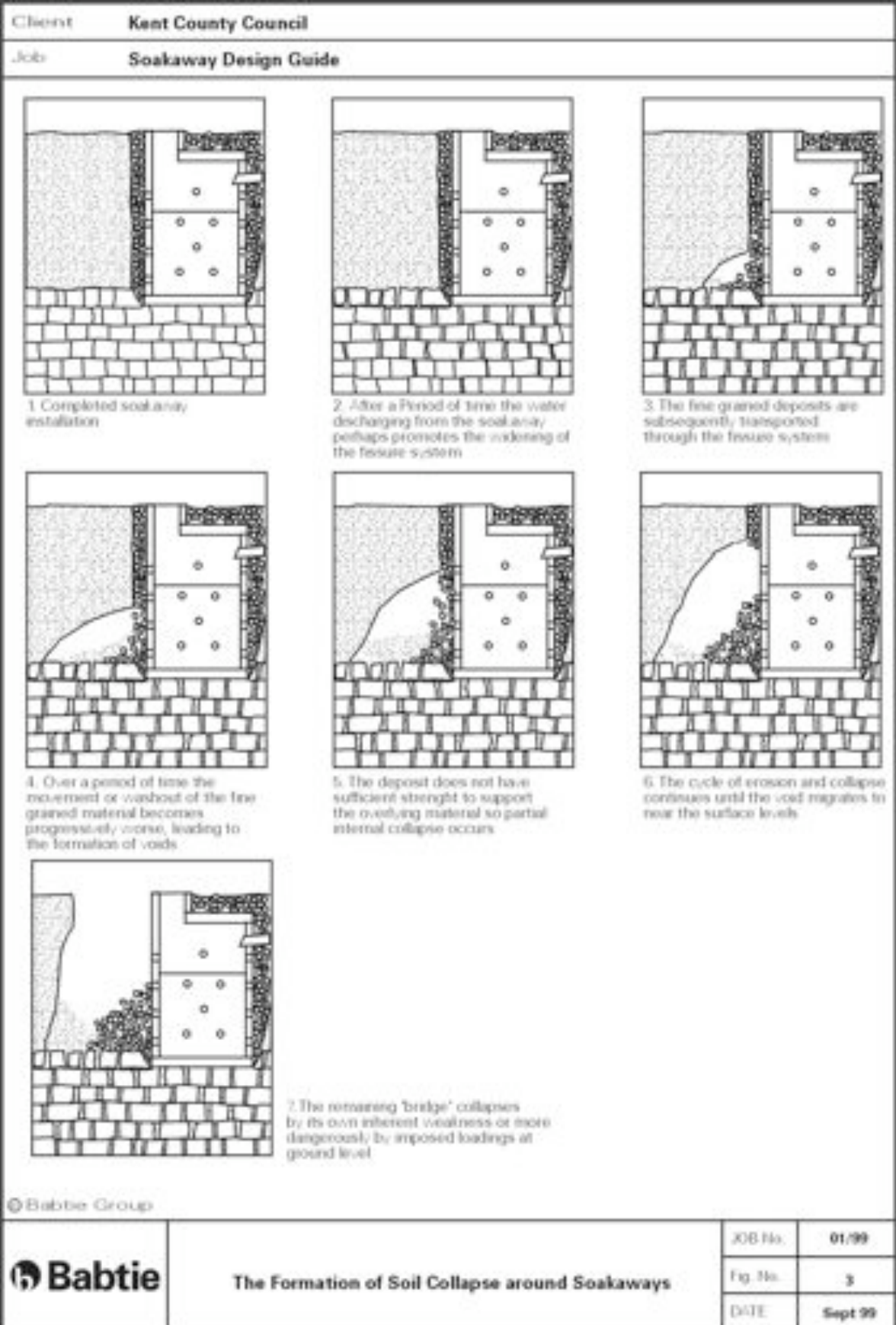
Client	Kent County Council		
Job	Soakaway Design Guide		
			
© Babbie Group			
	Conventional Soakaway	JOB No.	1/99
		Fig. No.	1
		DATE	Sep 99

Client	Kent County Council
Job	Soakaway Design Guide



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	Deep Bored Soakaway	JOB No.	1/99
		Fig. No.	2
		DATE	Sep 99



Client	Kent County Council
Job	Soakaway Design Guide

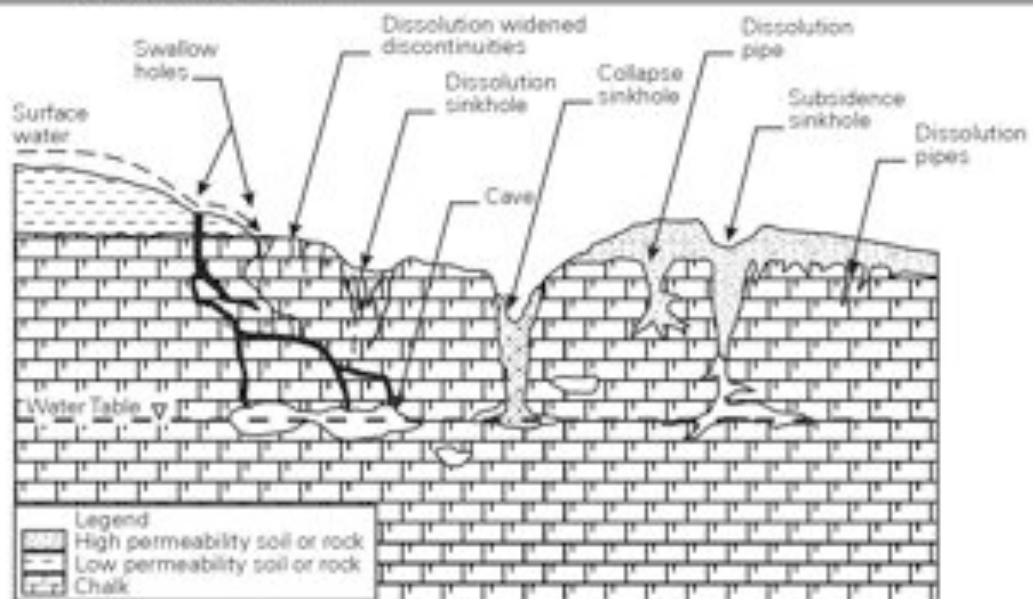
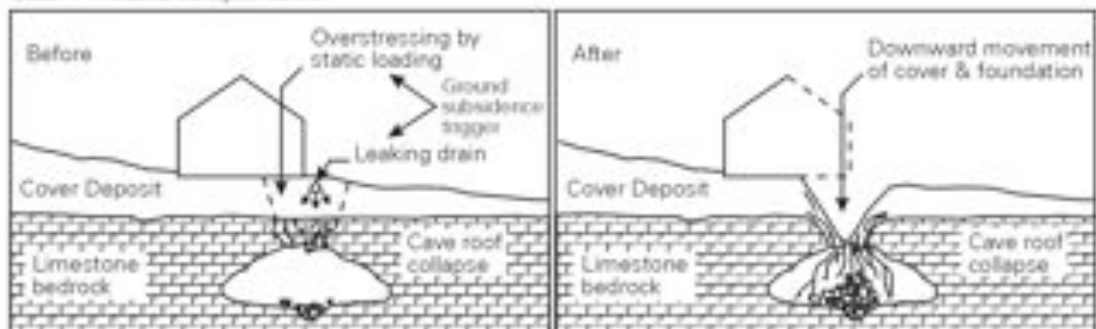


Figure 4a Examples of Foundation Failures Caused by Solution Features or Cavities in Chalk (based on Applied Geology, 1993)

Case 1: Induced collapse doline



Case 2: Induced subsidence doline

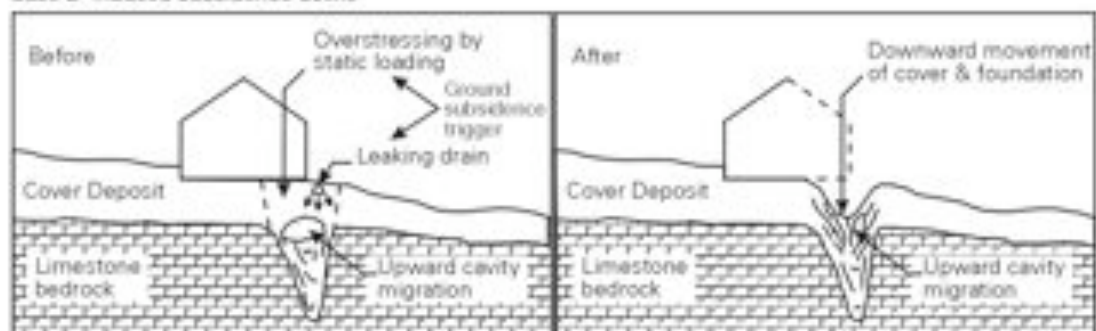
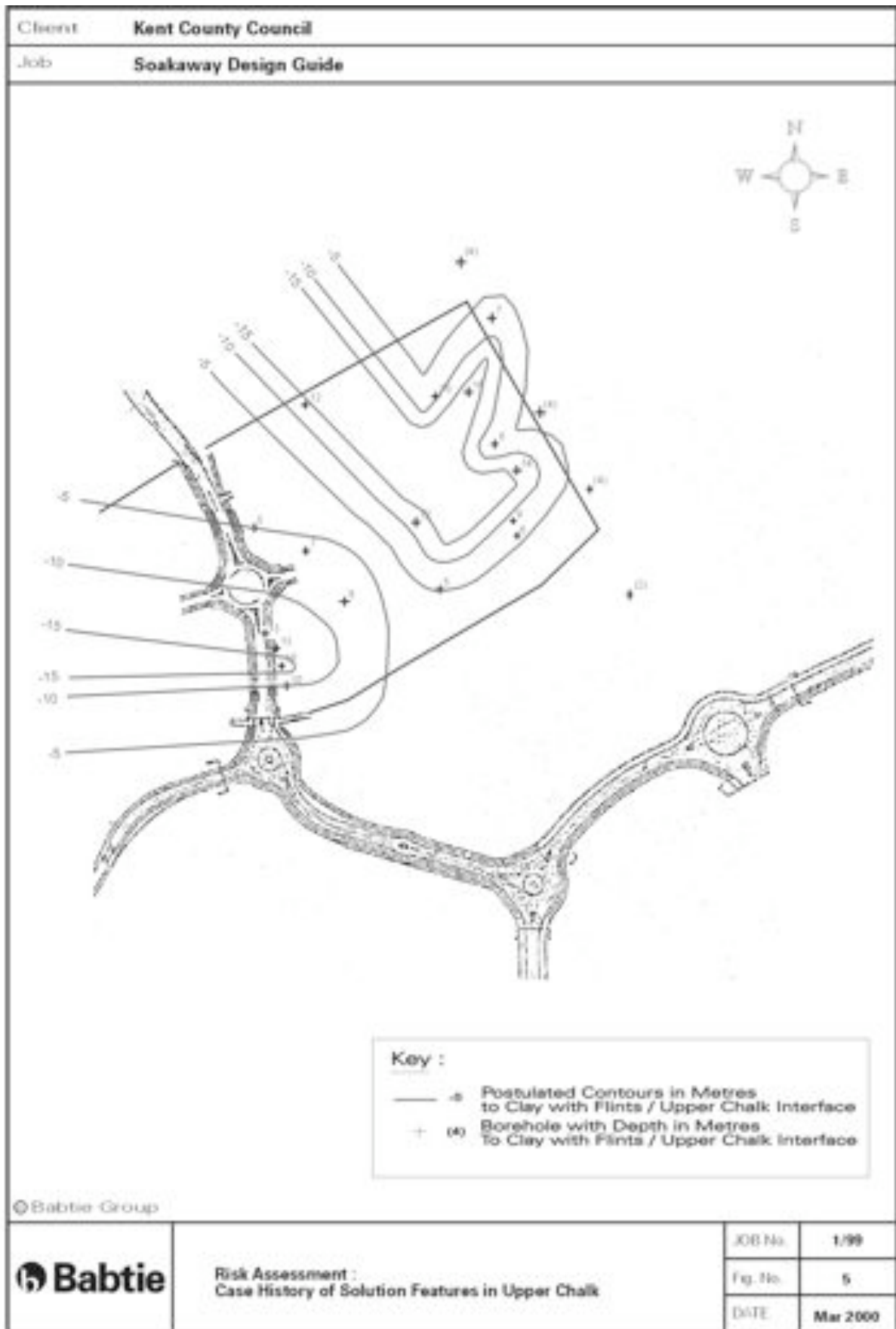
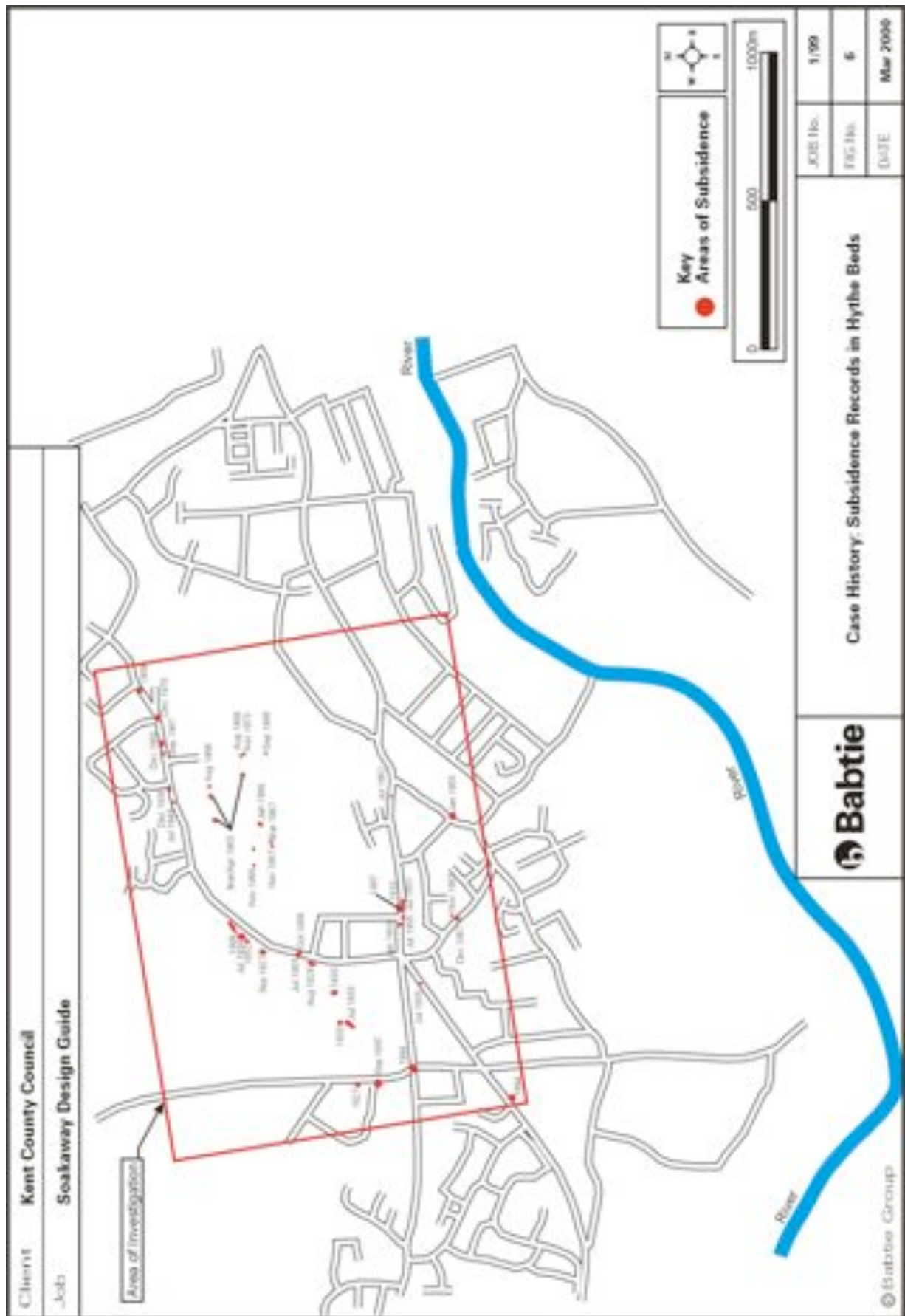


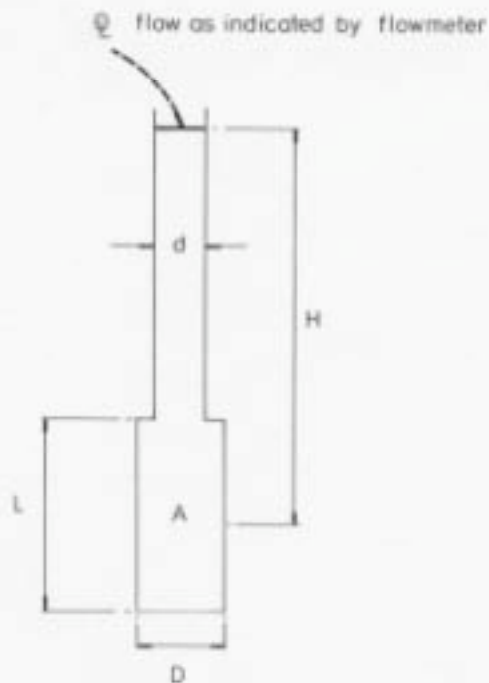
Figure 4b Typical Dissolution Features in Chalk

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	Figure 4a Examples of Foundation Failures Caused by Solution Features or Cavities in Chalk (based on Applied Geology, 1993) Figure 4b Typical Dissolution Features in Chalk	JOB No.	1/99
		Fig. No.	4a & 4b
		DATE	Sept 99







The soakage rate as calculated from this type of test is relevant only for one particular value of head, H .

The soakage rate is simply equal to the flow divided by the exposed area.


$$R = \frac{Q}{A}$$

In this case the exposed area is simply the area of the cylinder. The base is usually ignored due to the debris which accumulates at the bottom.

Area of the exposed cylinder $\pi D \times L$

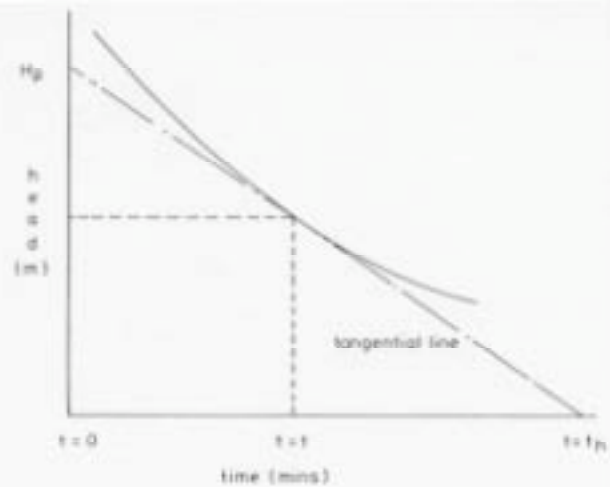
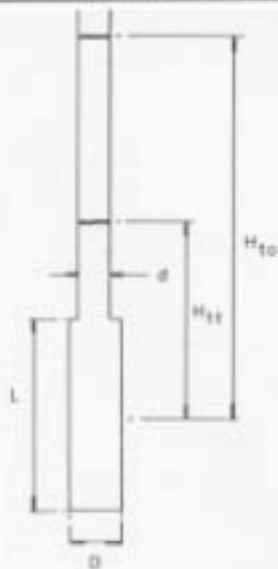
The soakage rate $\frac{Q}{\pi D \times L}$

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	Constant Head Soakage Test Calculations	JOB No.	1/99
		Fig. No.	7
		DATE	Sep 99

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Job **Soakaway Design Guide**



The soakage is calculated, like the constant head test, for a given value of head which in this case is designated H_{tt} .

This is done by first constructing a line tangent to the head vs time graph at the particular value of head under consideration. The tangent is extended until it intercepts both the x and y axis the ordinates of which are t_h and H_p respectively

The soakage rate is again $\frac{Q}{\text{area.}}$

The Q is given by the absorbed water in time $t = t_h$

$$Q = \frac{\pi d^2 \times H_p \times 1000}{4 t_h}$$

The exposed area is $\pi D L$

The soakage is thus:

$$\frac{1000.H_p.\pi d^2}{4 \pi D.L.t_h} = \frac{250.H_p.d^2}{D.L.t_h} \{l/m^2/min\}$$

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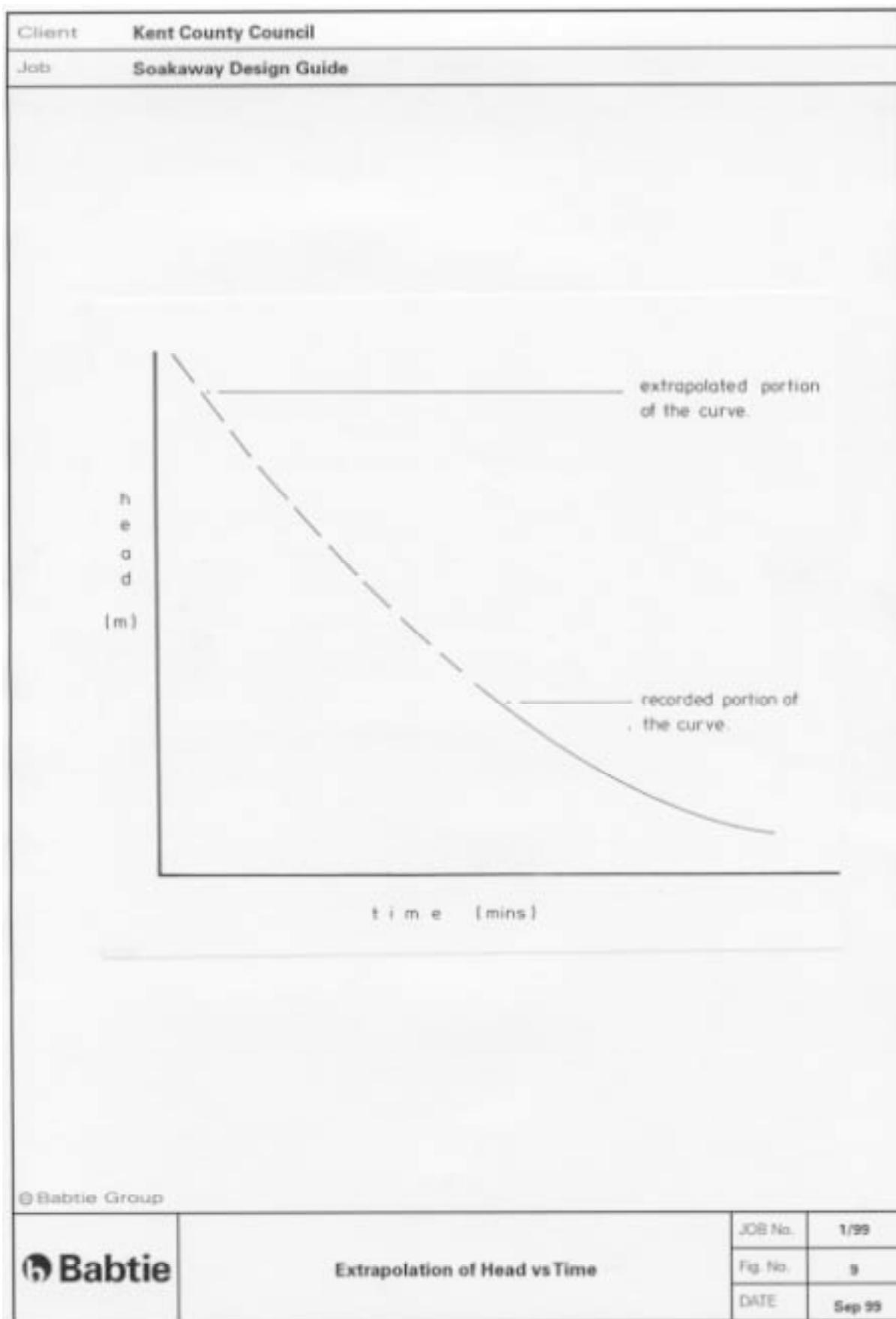
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Falling Head Soakage Test Calculations

JOB No. **1/99**

Fig. No. **8**

DATE **Sep 99**



Client	Kent County Council						
Job	Soakaway Design Guide						
Rates of Rainfall in mm/h for a range of duration and return period for a specified location in the United Kingdom							
National Grid Reference 4833E 1633N							
RETURN PERIOD (YEARS)							
DURATION							
2.0 Mins	75.6	93.4	120.5	138.3	158	187	213
2.5 Mins	76.5	87.5	113.4	130.4	149	177	202
3.0 Mins	66.3	82.3	107.2	123.4	141	168	192
3.5 Mins	62.8	77.8	101.7	117.3	135	161	184
4.0 Mins	59.6	73.8	96.8	111.8	128	154	176
4.1 Mins	59.1	73.1	95.9	110.8	127	152	174
4.2 Mins	58.5	72.3	95.0	109.8	126	151	173
4.3 Mins	57.9	71.6	94.1	108.8	125	150	172
4.4 Mins	57.4	71.0	93.2	107.9	124	149	170
4.5 Mins	56.9	70.3	92.4	106.9	123	148	169
4.6 Mins	56.3	69.6	91.6	106.0	122	146	168
4.7 Mins	55.8	69.0	90.8	105.1	121	145	166
4.8 Mins	55.3	68.3	90.0	104.2	120	144	165
4.9 Mins	54.8	67.7	89.2	103.4	119	143	164
5.0 Mins	54.3	67.1	88.5	102.5	118	142	163
5.1 Mins	53.9	66.5	87.7	101.7	117	141	162
5.2 Mins	53.4	65.9	87.0	100.9	116	140	160
5.3 Mins	53.0	65.4	86.3	100.1	115	139	159
5.4 Mins	52.5	64.8	85.6	99.3	115	138	158
5.5 Mins	52.1	64.3	84.9	98.5	114	137	157
5.6 Mins	51.7	63.7	84.2	97.8	113	136	156
5.7 Mins	51.2	63.2	83.5	97.0	112	135	155
5.8 Mins	50.8	62.7	82.9	96.3	111	134	154
5.9 Mins	50.4	62.2	82.3	95.6	110	133	153
6.0 Mins	50.0	61.7	81.6	94.9	110	132	152
6.2 Mins	49.3	60.7	80.4	93.6	108	130	150
6.4 Mins	48.5	59.8	79.2	92.2	107	129	148
6.6 Mins	47.8	58.9	78.1	90.9	105	127	146
6.8 Mins	47.1	58.0	77.0	89.6	104	125	144
7.0 Mins	46.4	57.2	75.9	88.4	102	124	143
7.2 Mins	45.8	56.4	74.9	87.3	101	122	141
7.4 Mins	45.2	55.6	73.9	86.1	100	121	139
7.6 Mins	44.5	54.8	72.9	85.0	99	119	138
7.8 Mins	44.0	54.1	71.9	84.0	97	118	136
8.0 Mins	43.4	53.4	71.0	82.9	96	117	135
8.2 Mins	42.8	52.7	70.1	81.9	95	115	133
8.4 Mins	42.3	52.0	69.3	81.0	94	114	132
8.6 Mins	41.8	51.4	68.4	80.0	93	113	131
8.8 Mins	41.2	50.7	67.6	79.1	92	112	129
9.0 Mins	40.8	50.1	66.8	78.2	91	110	128
9.2 Mins	40.3	49.5	66.0	77.3	90	109	127
9.4 Mins	39.9	49.0	65.3	76.4	89	108	125
9.6 Mins	39.4	48.4	64.6	75.5	88	107	124
9.8 Mins	39.0	47.9	63.8	74.8	87	106	123
10.0 Mins	38.6	47.4	63.1	74.0	86	105	121
10.5 Mins	37.6	46.1	61.5	72.1	84	102	118
11.0 Mins	36.7	44.9	59.9	70.2	82	100	116
11.5 Mins	35.8	43.8	58.4	68.5	80	97	113
12.0 Mins	35.0	42.8	57.0	66.9	78	95	111
12.5 Mins	34.2	41.8	55.7	65.4	76	93	108
13.0 Mins	33.4	40.8	54.4	64.0	75	91	106
13.5 Mins	32.7	39.9	53.3	62.6	73	89	104
14.0 Mins	32.0	39.1	52.1	61.3	72	87	102
14.5 Mins	31.4	38.3	51.0	60.0	70	86	100
15.0 Mins	30.8	37.5	50.0	58.8	69	84	98
16.0 Mins	29.6	36.1	48.1	56.6	66	81	94
17.0 Mins	28.6	34.8	46.3	54.6	64	78	91
18.0 Mins	27.6	33.5	44.7	52.7	62	76	88
19.0 Mins	26.7	32.4	43.2	51.0	60	73	85
20.0 Mins	25.9	31.4	41.8	49.3	58	71	83

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Table 7. of Road Note 35

JOB No.

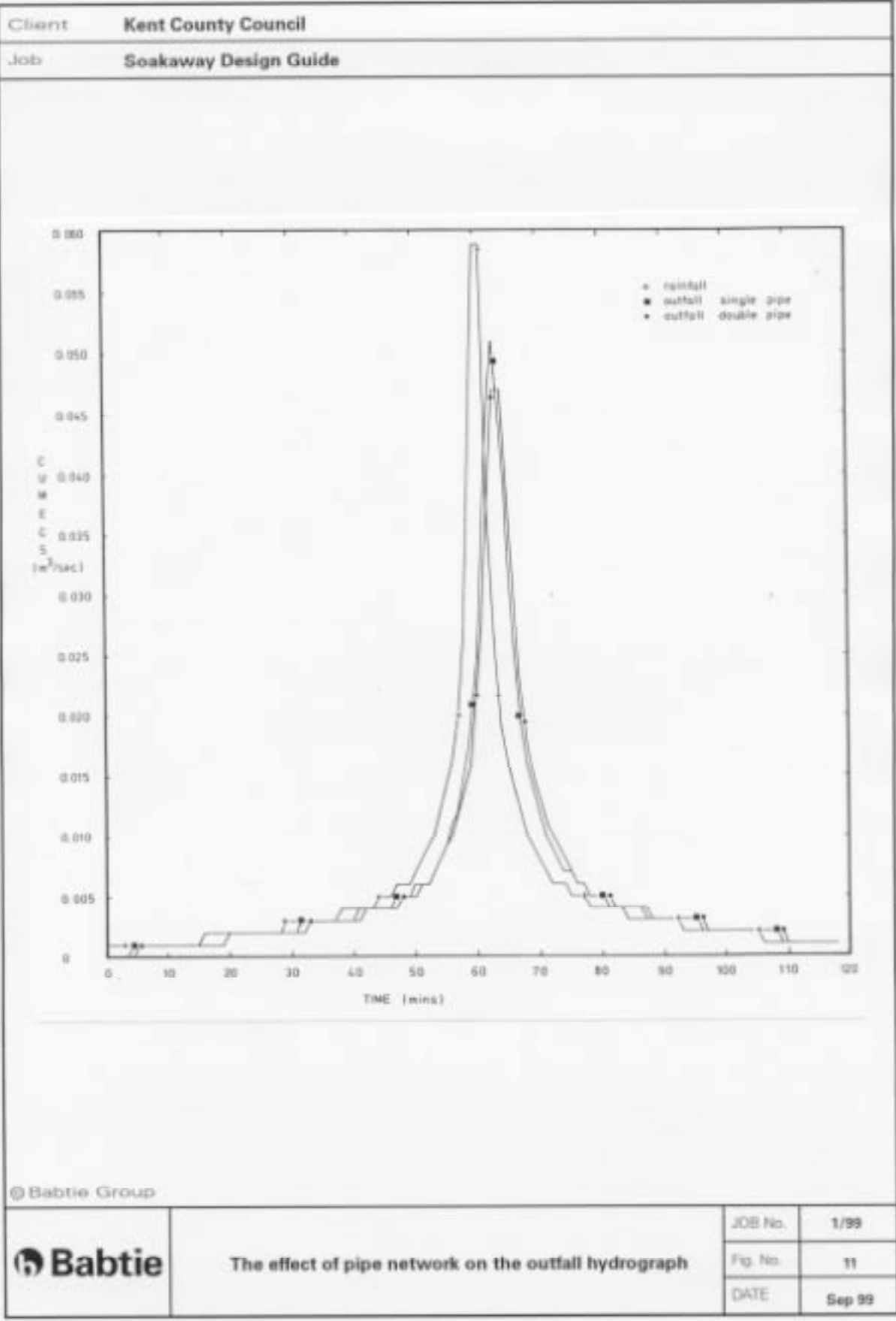
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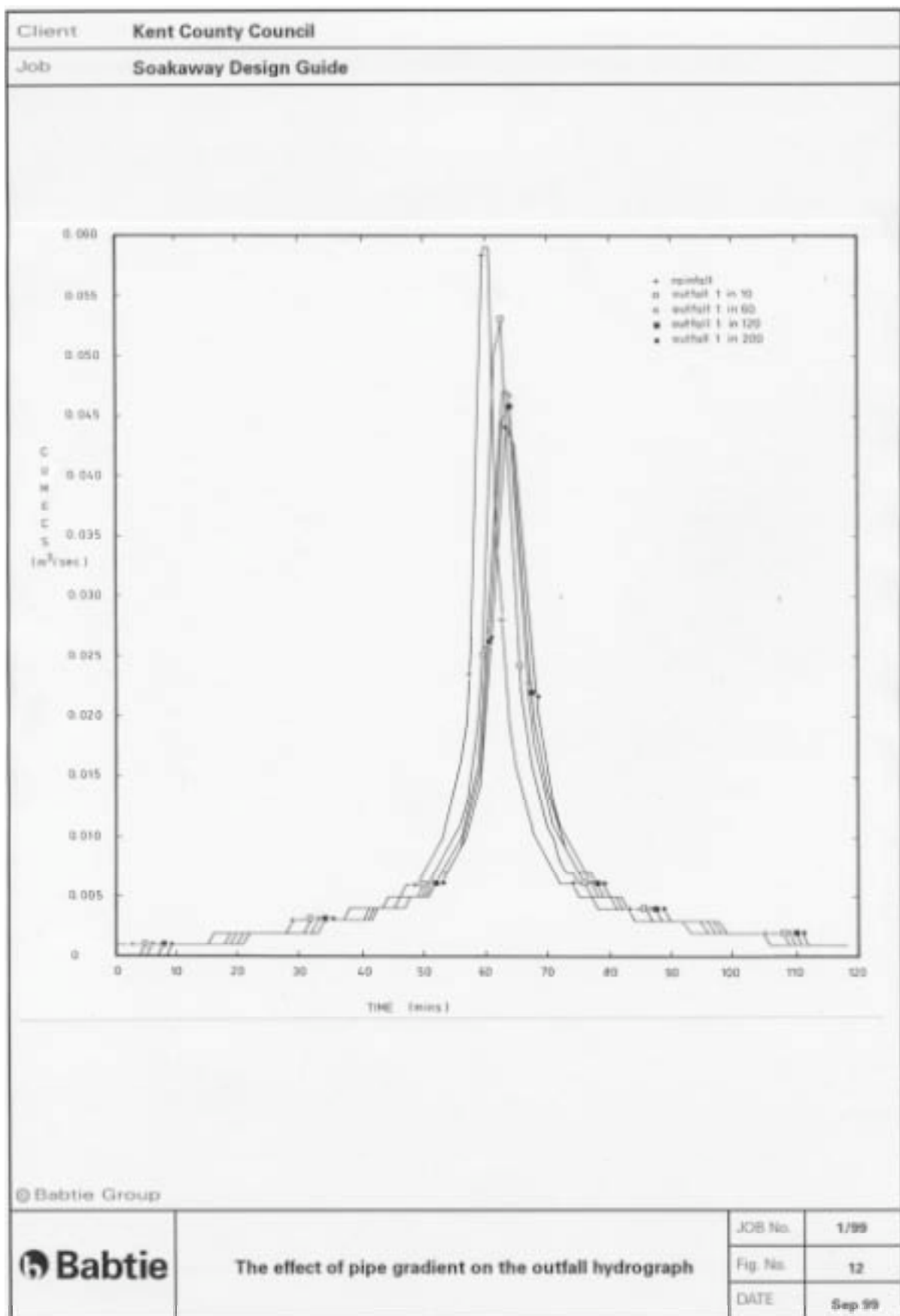
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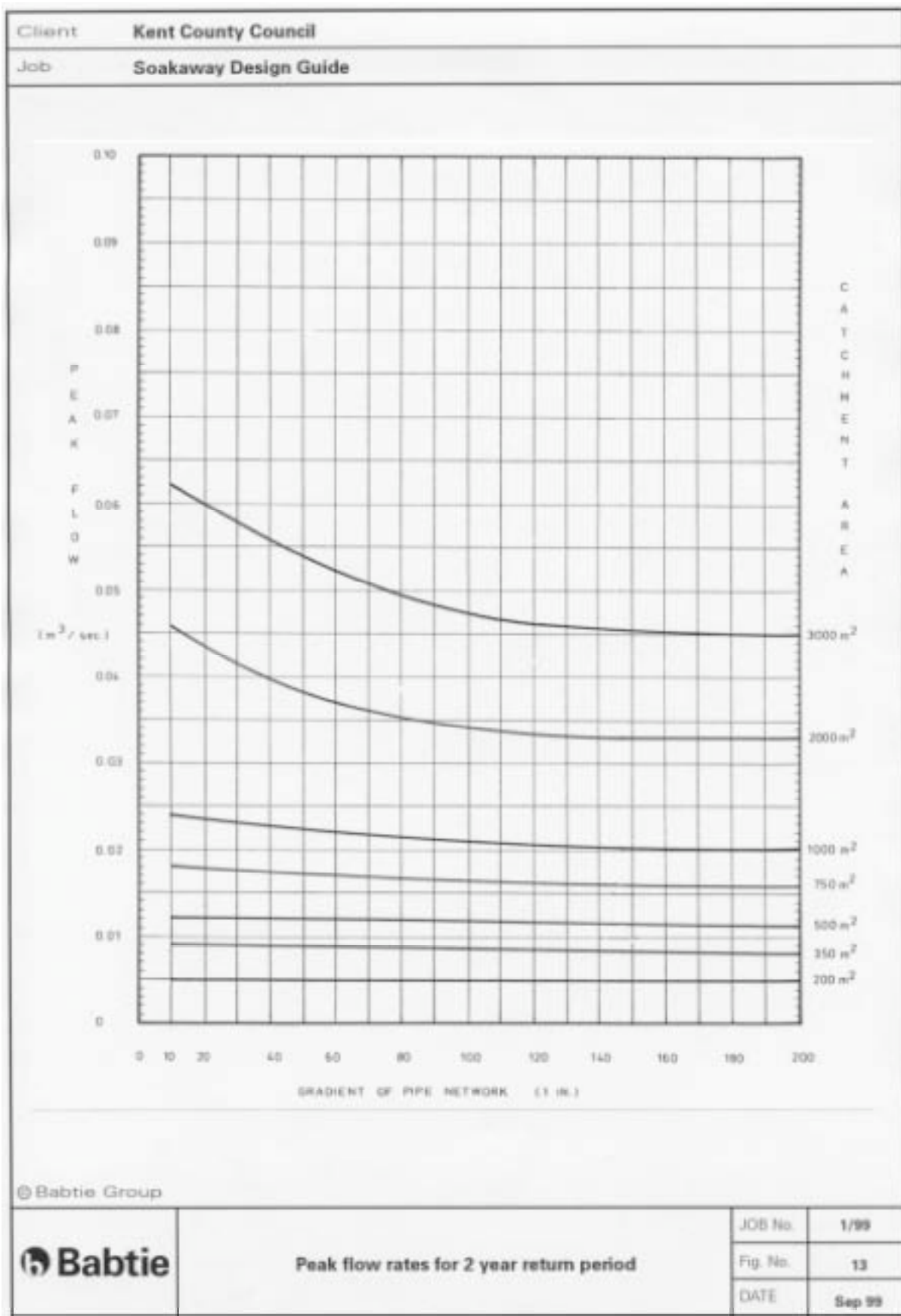
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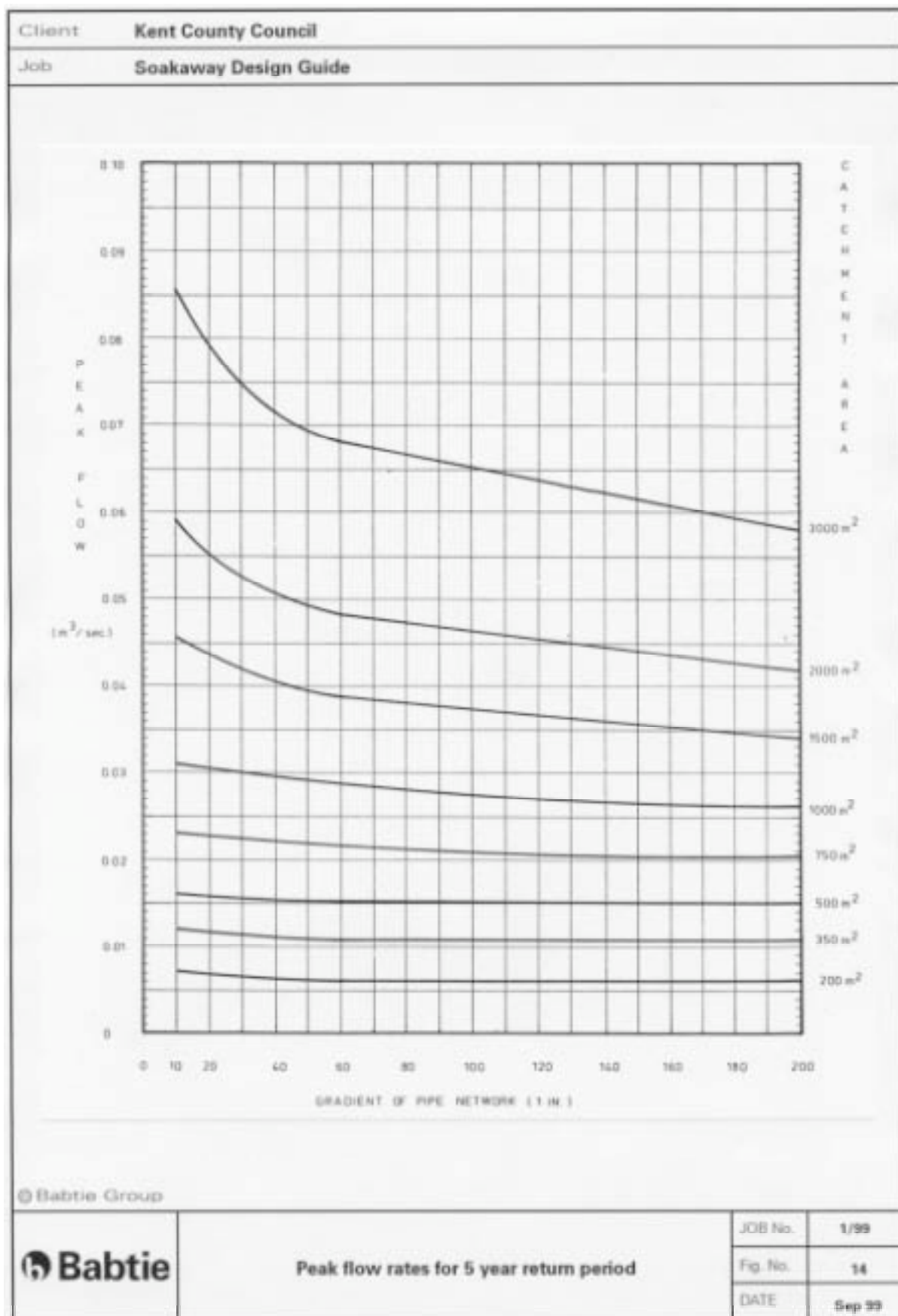
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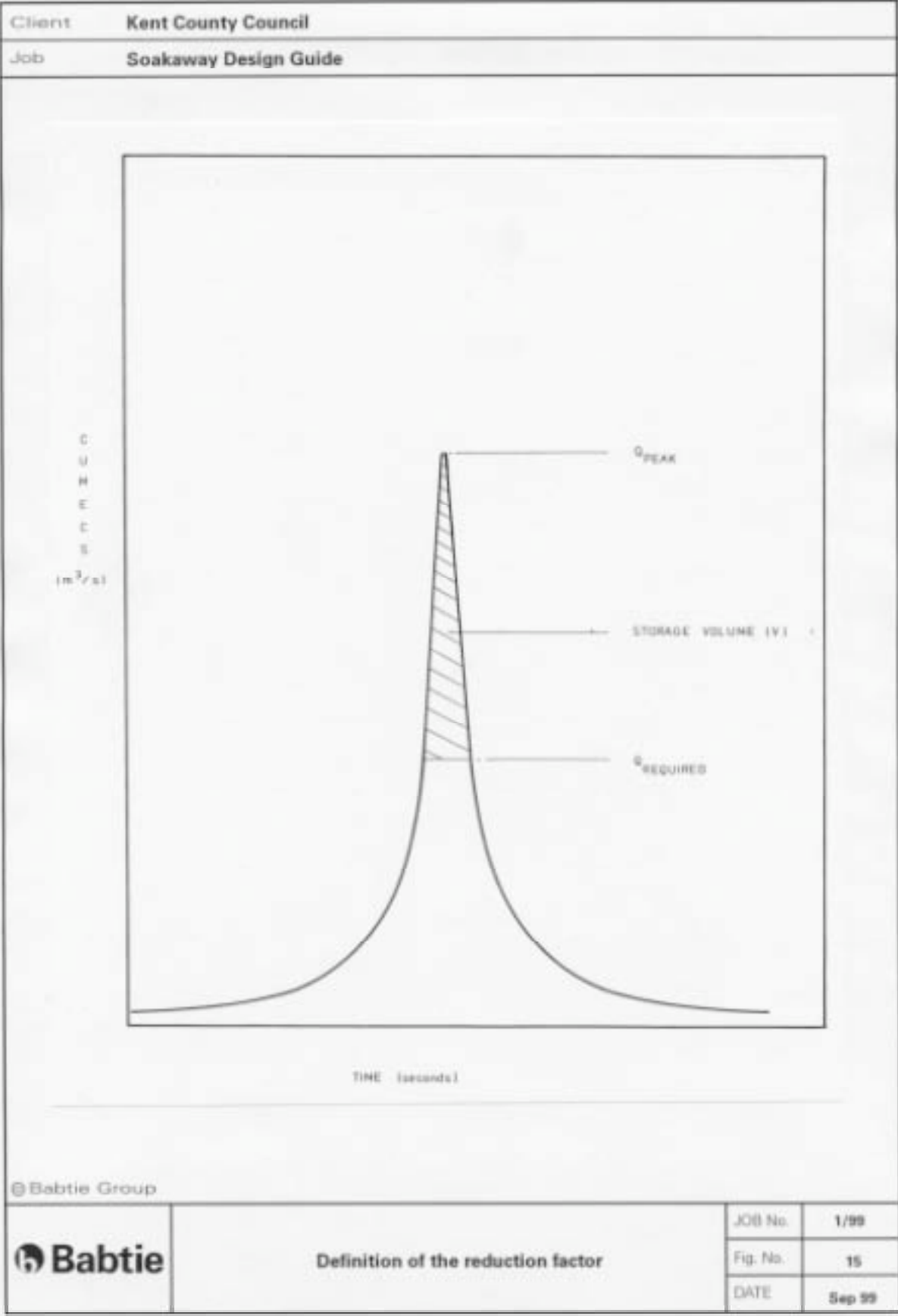
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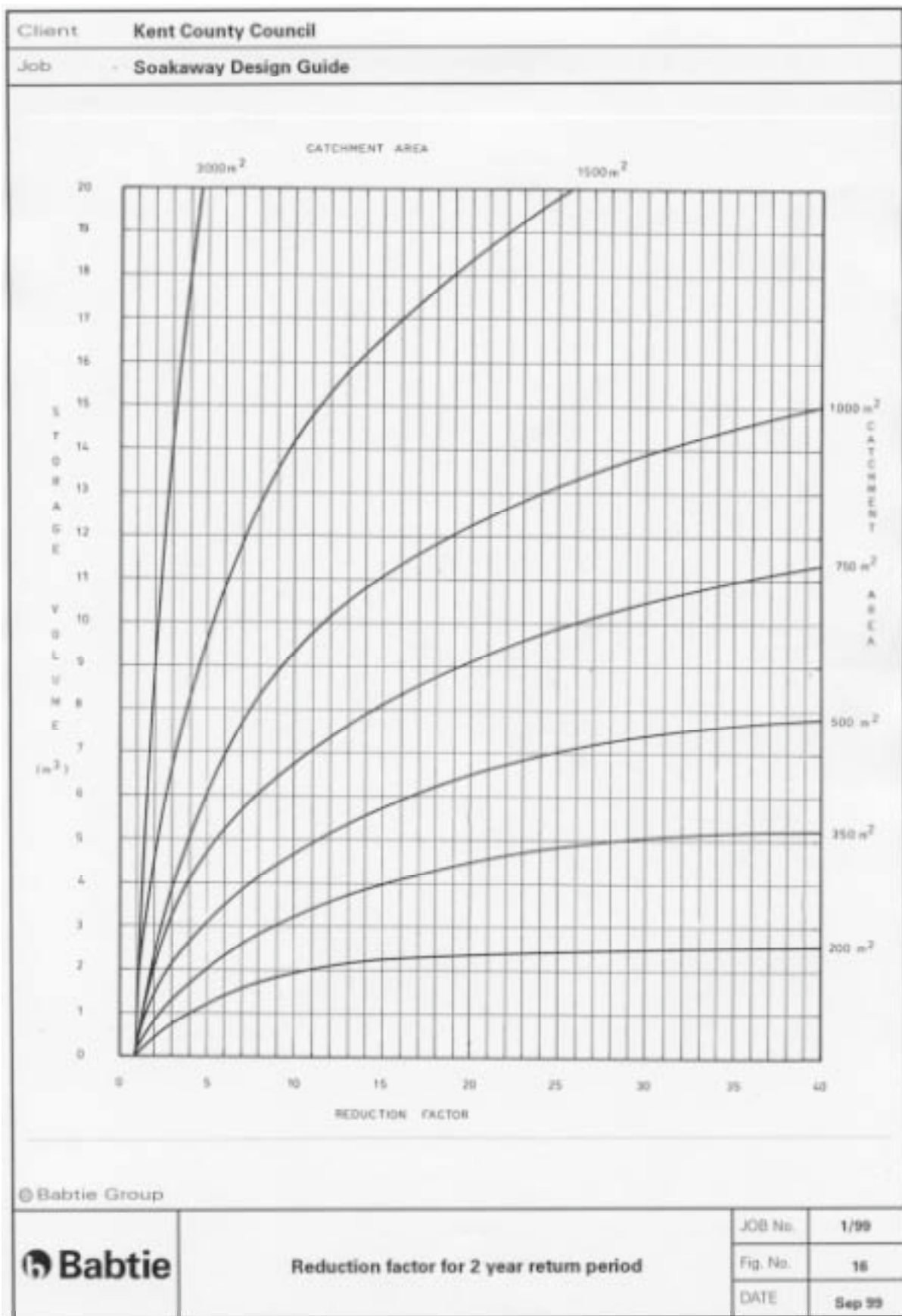


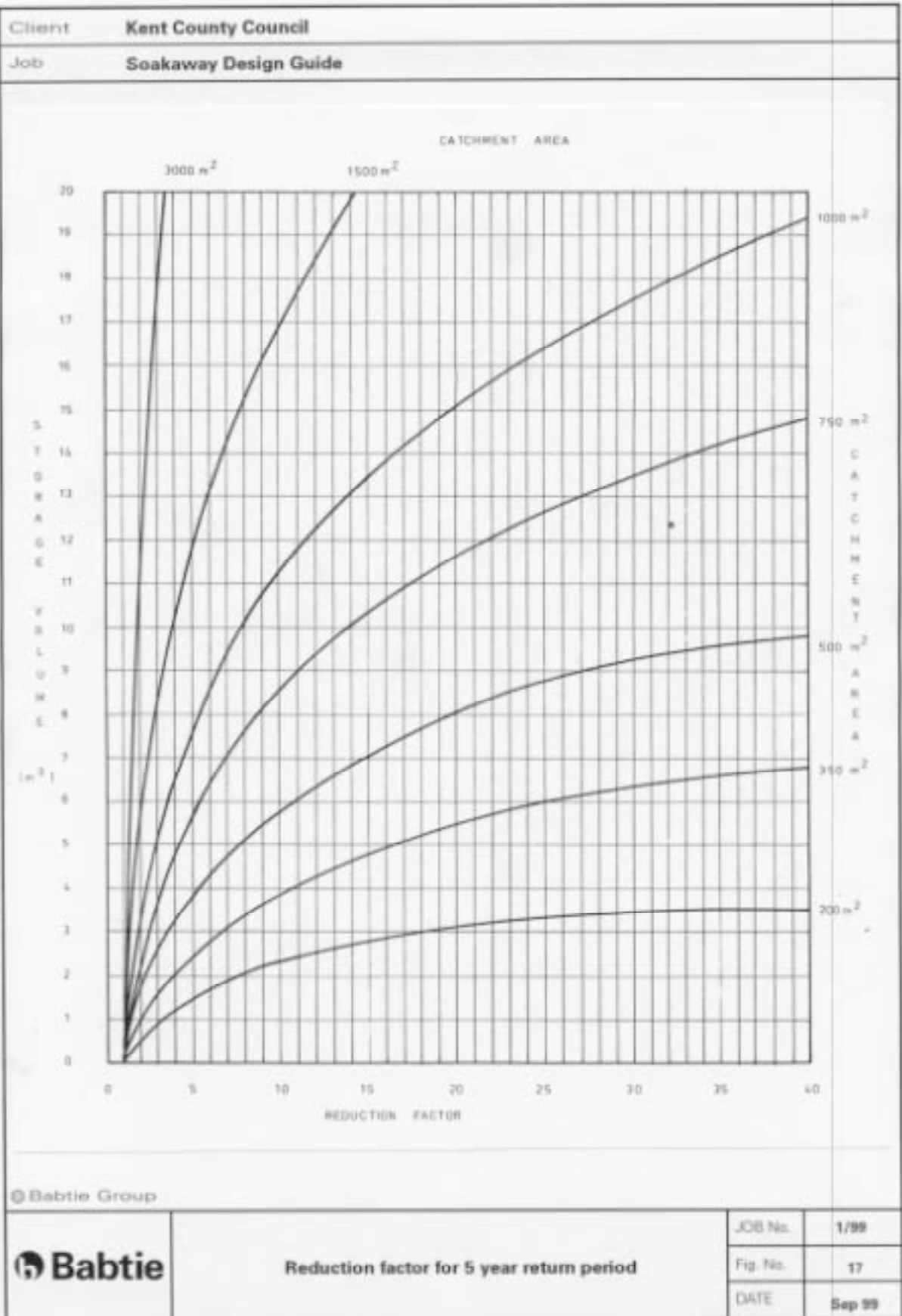












APPENDIX C

Plates



Plate 1 Road collapse caused by washout into gull feature in the Hythe Beds



Plate 2 Washout near soakaway in Hythe Beds



Plate 3 Solution features in Upper Chalk



Plate 4 Gravel filled reinstatements adjacent to drain runs over solution features in chalk



Plate 5 Isolated solution feature
in Upper Chalk



Plate 6 Orange iron-staining to chalk
close to a solution feature



Plate 7 Small gull feature in Hythe Beds
showing local dip of ragstone layers

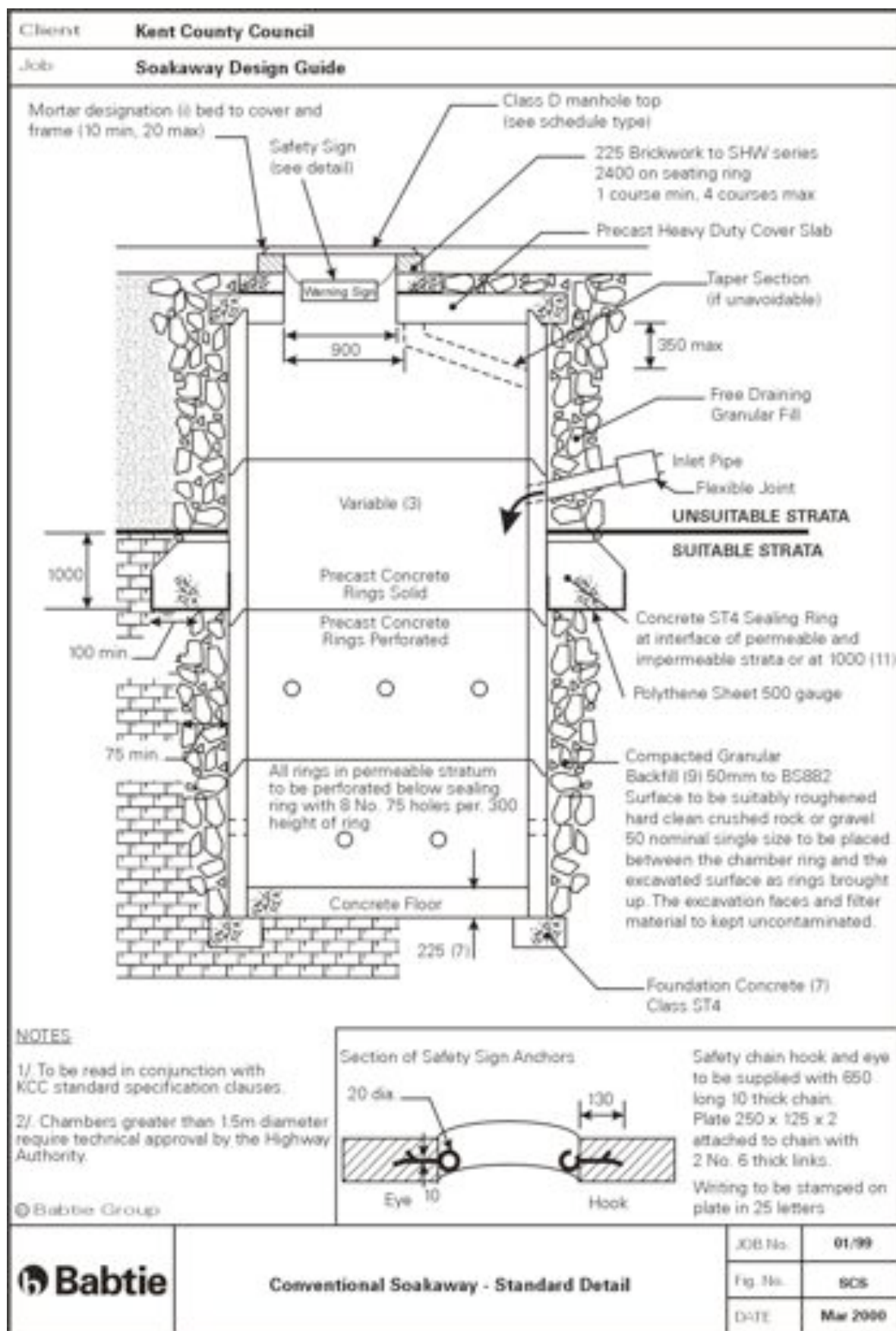
APPENDIX D

Soakaway Specifications

<u>Appendix D1</u>	Specification for conventional chamber
<u>Appendix D2</u>	Specification for deep bored soakaway

Appendix D1 Kent County Council Specification: Conventional Soakaway

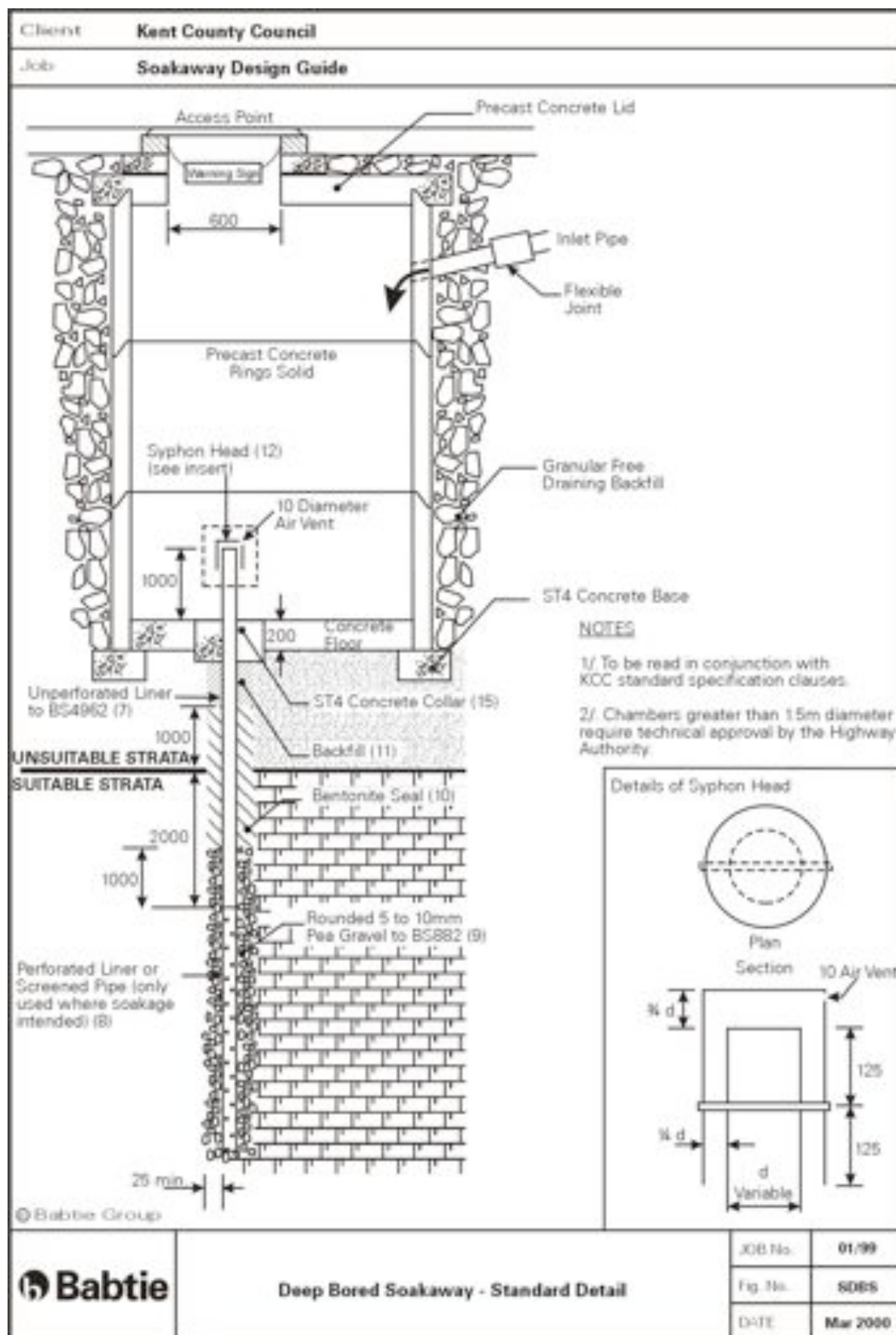
1. These notes shall be read in conjunction with drawing SCS with all dimensions in millimetres.
2. Each location considered for the installation of a soakaway should be investigated by means of constant or falling head soakage tests.
3. All precast units shall be in accordance with BS 5911. The size and depth of the installation shall be dependent on the design of the system.
4. All brickwork and chamber covers shall comply with Clause 507 of the Department of Transport's "Specification for Highways Works".
5. All inlets to the soakaways shall pass through a silt trap.
6. Pipeline connections to the chamber shall be in accordance with Clause 507 (3 and 15) of the Department of Transport's "Specification for Highways Works".
7. The soakaway rings shall be supported on a concrete base not less than 200mm thick with a diameter of not less than the external diameter of the rings themselves. The concrete used shall be Class ST4.
8. The perforated rings shall not commence less than 1 metre below the interface between the overlying unsuitable strata and the suitable strata with 8No 75mm diameter holes per 300mm height of soakaway ring.
- 9- Except where the Engineer permits, the granular backfill to the chamber shall be 50mm nominal single sized hard, clean, crushed rock or gravel complying to BS 882 and compacted in accordance with the 600 series clauses of the Department of Transport's "Specification for Highway Works".
10. The granular backfill shall be sealed at the level of the highest perforated ring by a layer of 500 gauge polythene. This shall be abutted and extended 450mm up the sides of the chamber and excavation.
11. The polythene shall be capped with Class ST4 concrete to a nominal thickness of 0.15m, or as directed by the Engineer, Where the concrete is positioned such as to conflict with Note 6, the final details are to be at the direction of the Engineer.
12. The remaining excavation shall be backfilled according to the 600 series clauses of the Department of Transport's "Specification for Highway Works".
13. All joints, soakaway holes and lifting eyes in the precast concrete rings that are located in unsuitable strata and extending 1 metre into the suitable strata below the interface with the unsuitable strata should be sealed with mortar or other sealing materials approved by the Engineer.
14. A safety sign shall be anchored below the manhole cover comprising a 650mm long by 10mm thick safety chain with a 250mm by 125mm by 2mm thick plate attached to the chain with 2No 6mm thick links. The writing stamped onto the plate in 25mm letters shall be "Soakaway x metres deep".
15. Chambers (manholes, shafts & soakaways etc) greater than 1500mm internal diameter and located within the highway boundary will require Technical Approval by KCC Network Management Bridge Maintenance through their agents Babbie Group (Highway Structures Office). The design and detailing shall be in accordance with the Approval In Principle (AIP) submitted to the Technical Approval Authority (TAA) for each chamber. The use of precast products to BS5911 will not necessarily be accepted. Type approval is available for the design of some pre-cast supplier's products with further details of these from the TAA. The following additional requirements are required for TAA approval:-
 - (a) The chamber cover slab shall be set a minimum 450mm below the finished surface level.
 - (b) The buried surfaces of the cover slab shall receive two coats of bituminous emulsion.
 - (c) Non perforated shaft rings shall receive a minimum 150mm ST2 concrete surround.
 - (d) The base slab thickness shall be determined by the design criteria contained in the AIP



Appendix D2 Kent County Council Specification: Deep Bored Soakaway

1. These notes should be read in conjunction with drawing SDBS with all dimensions in millimetres.
2. The conventional soakaway shall be constructed in accordance with the relevant specification (see Appendix D1 and Drawing SCS) .
3. Where the conventional soakaway is to act purely as a storage chamber, it shall be backfilled in accordance with the 600 series clauses of the Department of Transport's "Specification for Highway Works"
4. In all installations the inspection cover must be sited vertically above the borehole to facilitate future maintenance.
5. Each location considered for the installation of a soakaway should be investigated by means of a constant or falling head soakage test. These tests should be carried out in the deposits considered suitable and at vertical intervals as agreed with the Engineer. Alternatively a proof test may be undertaken on the installed liner.
6. When the soakage tests are complete, the borehole could be deepened, if necessary, to provide the required length within the soakage layer and a deep bored soakaway liner installed.
7. The borehole liner shall have an internal diameter of at least 100mm but normally not more than 250mm. It may be UPVC in accordance with BS 4962 or other durable material agreed with the Engineer. It must be capable of insertion in the borehole without risk of breakage, or damage to the joints.
8. Perforated slotted or screened pipe shall only be used in that section where soakage is intended. The apertures in such a pipe shall be smaller than any aggregate placed between the liner and the borehole wall. The perforated pipe shall commence a minimum of 2 metres below the interface with the overlying unsuitable strata.
9. After installation of the lining tube, rounded 5mm to 10mm pea gravel shall be placed in the annulus between the borehole wall and the lining tube to a level 1 metre above the perforated pipe as the casing is withdrawn. Care must be taken to ensure that the level of the aggregate is maintained just above the bottom of the casing to prevent collapse of the borehole. In addition excessive heights of aggregate above the bottom of the casing could jam the casing during withdrawal and risk lifting the liner to the detriment of the installation.
10. A bentonite seal shall be placed on top of the aggregate to extend nominally 1.0 metres above and below the interface with the overlying unsuitable material. Care should be taken to prevent contamination of the gravel.
11. As the casing is withdrawn the remaining annulus shall be filled with suitable backfill material or grout as directed by the Engineer.
12. The removable syphon head shall be in any approved non-rusting material. A suitable design in UPVC is illustrated. It shall have a 10mm diameter air vent at its highest point. It shall be fixed to the liner in a manner agreed by the Engineer. Correctly fitted push fitting syphon heads are acceptable to remove the need for entry into a confined space. .
13. On completion, if requested, the soakaway shall be tested to the satisfaction of the Engineer.
14. Where the deep bored liner has to be installed within an existing chamber without a concrete foundation, a Class ST4 concrete floor, 225mm thick, shall be cast to avoid any necessary erosion of the soil at the base of the chamber.
15. Where the deep bored liner has to be installed within a chamber in which a hole has been made during drilling, or preformed in the base, the annulus remaining after completion of the installation shall be filled with a 225mm thick concrete plug Class ST4.
16. Where the deep bored liner has to be installed within an existing chamber which does not have a concrete base and which is showing signs of instability then the bottom rings should be underpinned and a concrete floor (Class ST4), 225mm thick be cast beneath them. More serious instability will require additional ground improvement and/or relocation of the soakaway.
17. All joints, soakaway holes and lifting eyes in the precast concrete rings located in unsuitable strata should be sealed with mortar or other sealing materials approved by the Engineer. This shall prevent seepage of water by any other means than through the deep bored liner.

18. Where an existing soakaway shows signs of instability it is recommended that after the remedial measures have been completed the bottom two metres of the chamber be sealed with a bituthene coating protected by a 150mm thick dwarf wall formed from Class ST4 concrete. More serious instability may require additional ground improvement and/or relocation of the soakaway.



APPENDIX E

Summary of Environment Agency Aquifer Protection Policy Requirements
(“Policy and Practice for the Protection of Groundwater” 2nd Edition 1998)

Matrix 3c: Discharges of surface water to underground strata

Note 1. This matrix is a summary only and should be read in conjunction with the policy statements and the key in the Environment Agency “Policy and Practice for the Protection of Groundwater”

Note 2. Direct discharge into groundwater of surface water run-off is normally not acceptable

SOURCE PROTECTION

Activity	1 Inner Zone	2 Outer Zone	3 Catchment Zone
Discharges of surface water to soakaway			
Roof drainage	No objection (R5) (Provided for sole use of roof drainage)	No objection (R5)	No objection (R5)
Impermeable areas - public/amenity - large car parks - lorry parks - garage forecourts - major roads	Not Acceptable (R1)	Acceptable (R4)	Acceptable (R4)
	Not Acceptable (R1)	Acceptable (R3/R4) (with interceptor)	Acceptable (R4) (with interceptor)
	Not Acceptable (R1)	Presumption against (R2)	Acceptable (R3/R4) (with interceptor)
	Not Acceptable (R1)	Presumption against (R2)	Acceptable (R4) (with interceptor)
	Not Acceptable (R1)	Presumption against (R2) Acceptable only in exceptional circumstances	Acceptable only if investigation favourable and with adequate precautions (R4)
Industrial sites	Not Acceptable (R1)	Presumption against (R2)	Acceptable only if investigation favourable and with adequate precautions (R3/R4)

RESOURCE PROTECTION

Activity	Major Aquifer	Minor Aquifer	Non-Aquifer
Discharges of surface water to soakaway			
Roof drainage	No objection (R5)	No objection (R5)	No objection (R5)
Impermeable areas - public/amenity - large car parks - lorry parks - garage forecourts - major roads	Acceptable (R4)	Acceptable (R4)	Acceptable (R4)
	Acceptable (R4) (with interceptor)	Acceptable (R4) (with interceptor)	Acceptable (R4) (with interceptor)
	Acceptable (R4) (with interceptor)	Acceptable (R4) (with interceptor)	Acceptable (R4) (with interceptor)
	Acceptable (R4) (with interceptor)	Acceptable (R4) (with interceptor)	Acceptable (R4) (with interceptor)
	Acceptable (R4) (subject to investigation and with interceptor)	Acceptable (R4) (subject to investigation and with interceptor)	Acceptable (R4) (with interceptor)
Industrial sites	Acceptable only if investigation favourable and with adequate precautions (R3/R4)	Acceptable (R4) (subject to investigation with interceptor)	Acceptable (R4) (subject to investigation with interceptor)

APPENDIX F

Current Value of KCC Commuted Sums

APPENDIX F: Current Value of Commuted Sums payable to Kent County Council (May 2000)

(excludes charges levied by Environment Agency)

Soakaway Type	Basic Commuted Sum (BCS)	Enhanced Commuted Sums (if applicable)	
		<u>EITHER</u>	<u>OR</u>
		(a) Difficult Access (b) Rural location susceptible to silt collection (c) Chamber in rock (d) Two-thirds or more of the chamber/trench situated in unsuitable strata	Exceptional circumstances (eg - soakaway located above or in made ground, above or near sensitive aquifer, adjacent to a landfill, adjacent to contaminated land, adjacent to a landslip)
Conventional Soakaway	£4000 each	BCS x 1.5	BCS x 3.0
Deep Bored Soakaway	£4000 each	BCS x 1.5	BCS x 3.0
Trench Soakaway	£750 per linear metre	BCS x 1.5	BCS x 3.0

APPENDIX G

Highway Structure Definition and Policy for New Works

Highway Structure Definition and Policy
for New Works



(including drainage and statutory
undertakers structures under the NRSWA)

KCC Network Management
Bridge Management
Invicta House (1)
County Hall
Maidstone
KENT ME14 1XX

The definition for a highway structure can fall into one of two types, either:

- a) any structure built in, under, or over, the highway where the least internal span dimension is equal to, or exceeds 900mm. This includes amongst others, bridges, footbridges, pipe gantries, culverts, pipes, runnels, chambers, cellars, shafts, manholes, soakaways and Storm water balancing tanks etc. or,
- b) any structure built in, or within 3.66m of, the highway which supports the highway, or ground above it, where the retained height either above or below the highway is 1400mm, or more. This includes amongst others, retaining walls, headwalls, basements, cellars and reinforced earth where the face is at an angle of 70° or more, etc.

The definition of 'highway' used above is within the highway boundary and includes the carriageway, footway and all verges if there is any doubt as to the classification of a structure please contact KCC's agents Babbie Group (Highway Structures Office) at the address given below for adjudication-

All structures that are classified as highway structures under the above require Technical Approval of the design and construction as set out in the Technical Approval Procedure, which is administered by Babbie Group (Highway Structures Office) at the address Given below. This applies, without exception, to all structures regardless of promoter whether it be the Highway Authority, County Council, District Council, Parish, a private individual, or company (including Statutory Undertakers under the NRSWA). This procedure may also be used to regulate other structures for which approval is required including buildings spanning, or protruding (regardless of overhang dimension) into the highway.

Approval to structures that fall below the above classification (i.e. <900mm span and <1400mm retained height) is a matter for the local Highway Management Unit, although technical advice on these is available from Babbie Group if required.

The only exception to the above for highway structures are chambers built to the Highway Construction Detail Drawings, No. F3 to F 12, where the designs are accepted as standard, appropriate to the use of BS 5911 and technical approval is already accepted. All chambers which do not conform to the Highway Construction Detail Drawings do require full technical approval and need to be designed to current highway loading and design standards. The following table summarizes the requirements:

Shaft Type	Design Standards	Technical Approval
Any Shape < 900 mm	BS 5911	No
Circular I.D. 900, 1050, 1200, 1350 & 1500 mm	BS 5911	No
Circular I.D. > 1500 mm	Highway Standards	Yes
Any Shape ≥ 900 mm (except circular)	Highway Standards	Yes

The list of standard design and detailing codes is contained in Appendix A of the Approval In Principle document which forms part of the Technical Approval Procedure. The only exception to the detailing requirements of BD 24/92 is that for factory produced reinforced concrete products for chambers (excluding cover slabs), tunnels and pipes (excluding box culverts), the cover requirements may be reduced to a minimum 20mm where the manufacturer can demonstrate that their typical concrete strength is actually greater than 60N/mm². This relaxation must be clearly identified in paragraph 4.3 of the Approval in Principle document, if proposed.

If it is hoped to use a code not included in Appendix A of the Approval In Principle document please discuss with Babbie (Highway Structures Office) at an early stage. Unless specifically allowed above the following codes are not appropriate for the design and detailing of highway structures, and should not be used:

- BS8110 Structural use of concrete.
- BS5950 Structural use of steelwork in building.

Babbie Group, Highway Structures Office (TAA),
Sandling Block, Springfield, Maidstone, Kent, ME14-2TG (Tel01622-666000)

