PRACTICAL ASPECTS OF TANK SEWERS IN WASSP-SIM

By R J Long BSc MICE MIWEM

Scott Wilson Kirkpatrick & partners

Paper presented at the Wallingford Procedure Users Group
Autumn Meeting, Bournemouth, 1988

1 <u>Synopsis</u>

1.1 The paper summarises experiences gained during the design of several tank sewers.

1.2 In the first part of the paper a case study of a recent project is given. The project is followed from initial investigation, through selection of attenuation storage as the preferred option to completion of the detailed design. It is shown how at each stage of development of the design a rational selection process leads to the evolution of the final scheme offering the required degree of protection at minimum cost.

1.3 The second part of the paper is concerned with modelling and analysis techniques for tank sewers in WaSSP. Alternatives are discussed briefly and hints for trouble-free simulation are given. The interpretation of water levels output by WaSSP-SIM is described. A useful aid to analysis of volumes stored within part-full pipes is offered.

1.4 Finally, brief reference is made to the current CIRIA research proposal 1089 on Scope for Control of Urban Run-Off.

2 <u>Case Study</u>

2.1 The tank sewer discussed in this paper is part of a programme of sewer improvements planned by the Director of Technical Services of Derby City Council for the Spondon area of Derby. Following the decision of Severn Trent Water Authority to implement a programme of studies for all significant population centres in their region, Spondon was selected as one of the first areas to be examined. Spondon is a suburb bordering the River Derwent east of Derby, having a population of about 16,000. Originally a separate village, it has grown over many years to become a part of the city. Previously served by its own water reclamation works, foul flows are now intercepted and taken to the central Derby works. While the interceptor sewer is of recent origin, the collector sewers upstream are much older. A separation scheme in the 1970s gave some relief to the foul system but by the mid-1980s an unacceptably frequent level of flooding complaints was being received from a number of sources across the catchment.

2.2 An initial study of each problem area showed that in almost every case, flooding was the result of high surcharge levels in the foul system during rainfall. It was concluded that local improvements would not give effective long-term solutions. A Drainage Area Study was begun with the objective of researching the existing sewerage system, both hydraulically and structurally, and producing a verified WaSSP model of the area. This would provide a base of information on which decisions on future improvements could be based rationally in the context of other problem areas in the Severn-Trent Region.

2.3 The WaSSP model represented both the foul and surface water systems, the latter being fragmented and draining to four separate outfalls. The systems were linked via eight existing storm sewage overflows. Since rationalisation of the overflows was expected to be a feature of the improvements it was a great advantage to have foul and surface water networks together in a single model. Flow gauging for verification purposes was undertaken on all major branches and particularly in areas where it was expected that improvements would be required.

2.4 One such branch was that serving 17 hectares of mainly residential development totalling about 300 dwellings on the western edge of the study area. Older development in the sub-catchment is drained by a partially separate (roofs to foul) system, while the newer development has a separate system. Gauging in other parts of the catchment had shown that "separate" foul systems are, in fact, sensitive to rainfall. A blanket impermeable area of 5% of the gross catchment was included in normally separate foul areas to model this behaviour. Although most of the sub-catchment was free of problems, one property was experiencing recurrent foul flooding. This was the old Creamery site, now used as workshops by several small businesses. During heavy rain, foul sewage flooded from inspection chambers in the yard. Onset of flooding of buildings occurred virtually concurrently. Ground level in the Creamery is about 2.5 m lower than road level and is thus the point at which surcharge within the sewer always first translates into flooding. A flow gauge was sited a little way downstream of the Creamery connection and a good verification of peak flow from SIM was achieved.

2.5 Having clearly identified the cause of the flooding, the search for the best solution began. To reduce computation time and to simplify the huge number of possible combinations of options which would otherwise have to be considered, each branch was split off from the main model and was considered as a separate entity. Thus, in the case of the Creamery, foul and surface water networks totalling 30 pipes were extracted from the 400-pipe full model. Checks indicated that predicted flows in the reduced model were slightly higher than in the full model, running under the same storm and were thus safe for design purposes. A range of options for restoring the required level of service was then examined. These were:

i Separation
ii Catchment transfer
iii Storm sewage overflow
iv Storage
v Resewerage

Options i - iii were quickly eliminated. Separation had already been carried out. Catchment transfer was not effective because adjacent systems had insufficient spare capacity. A new storm sewage overflow would have been ineffective because the receiving surface water sewer was higher than the ground level in the Creamery (deepening it would have been too costly), and because reverse flow in the throttle would result in unwarranted pollution. Options iv and v were then examined in detail.

2.6 In each case, the model was incrementally enlarged until flooding was just eliminated at the required design storm return period (10 years, since buildings are affected immediately when flooding from the system begins). A single storm duration (30 minutes) was used at this stage to reduce the amount of computing and analysis required.

2.7 Cost estimates were then prepared using WaSSP-CST. Resewerage proved to be about 40% more costly than storage, because of the substantial length requiring replacement, much of it at depths of 4 to 5 m. Storage had the additional advantages of reducing the amount of foul material spilled from downstream overflows and reducing the loadings on the existing surface water sewers which were to receive the overflowed sewage. Construction of a tank sewer was therefore recommended.

2.8 Following completion of the study of each branch, the proposed improvements were combined in an overall model and re-run to check for satisfactory overall operation.

2.9 The next stage of the investigation was accurately to size the tank sewer to be provided. Ultimate loadings in terms of PDWF and anticipated development were incorporated in the model. A range of storm durations from 30 to 240 minutes was then tested. Since Severn-Trent Water Authority require the operation of detention structures to be checked at 20 years return period, this was used for design purposes. From the storage requirement at each duration the peak requirement was estimated.

2.10 To this basic volume several additions were made. Firstly, an allowance for infiltration of 5 L/s during the period of operation of the tank was added. This was felt to be very much an over-allowance and was to have been checked by a night-time flow gauging exercise. Secondly, since there was to be no emergency overflow, an addition of 10% was made as an added factor of safety. It had been previously noted that under certain storm conditions the tank would operate tidally, heavy overloading downstream resulting in reverse flow into the tank, but it was believed that this would not be detrimental in service. Since the tank will operate by lowering the hydraulic grade line upstream and downstream it is not necessary to throttle the outlet. This is an advantage operationally, reducing the risk of blockages resulting from sedimentation during periods of retention.

2.11 During the detailed design process the layout of the proposed tank sewer was adjusted to achieve the required degree of protection at minimum cost. A short, large diameter tank sewer was compared with a longer length of smaller sewer. The benefits of moving the tank sewer some distance from the Creamery connections were also explored. Each configuration was costed, again using WaSSP-CST, and the maximum surcharge level in the Creamery connection was estimated. It was found that there was an inverse correlation between extra cost and surcharge level. Since no configuration offered any particular advantage of cost/benefits the cheapest option meeting the design brief was selected. This was a 75 m length of 1500 mm sewer situated close to the Creamery connection. An internal benching with 1:3 slopes and a half round dry weather channel was specified to assist in self-cleansing.

2.12 A tender of £75,000 for construction of the tank sewer has recently been accepted and work will shortly commence on site.

3 Modelling Technique

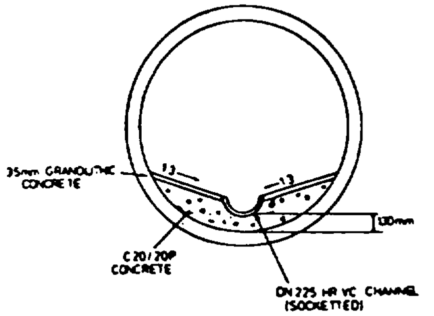
3.1 There are three ways in which a tank sewer can be modelled in WaSSP, these are:

(a) as a pipeline
(b) as an on-line tank
(c) as a pipeline with an on-line tank or pumping station as downstream control.

Typical SSDS for each option are included at Appendix A.

Even if no actual flow control device is to be installed at the downstream end of the tank, Option (C) will give the most satisfactory result. Performance under surcharged conditions is accurately modelled by the on-line tank routine, and the upstream pipeline provides a more realistic head/storage relationship (WaPUG User Note 7). If a flow control device is to be used a specified head/discharge relationship can be input.

3.2 The installation of a tank sewer invariably means an increase in sewer diameter. Flow velocities are reduced at all stages of flow, but especially when the tank is operating. Sedimentation within the tank will result and poor detailing may lead to unacceptable maintenance needs. Construction of a dry weather channel and benching will greatly assist self-cleansing at both high and low flows. A minimum transverse gradient of 1 in 12 is recommended by some authorities, but in the author's experience 1 in 3 slopes are likely to be more satisfactory. The benching reduces the storage capacity of the pipe and the WaSSP model should incorporate an equivalent diameter of a circular section having the same free cross sectional area. There will be a small error in the head/storage relationship. The sketch below shows a typical benching arrangement with equivalent diameters for the commonly used sizes of tank sewer.



H mm	Equivalent diameter
1800	1700
1500	1400
1350	1240
1200	1075
1050	925
900	765

3.3 An emergency overflow may or may not be provided for the actual tank sewer, depending on the availability of a suitable receptor sewer or watercourse. If provided, the relevant details can be entered on record 10. If an emergency overflow is not to be provided it is important that a dummy weir discharge coefficient and weir height are included on record 10 to avoid spurious oscillations in the computations. Defining overflow weir height as ground level has been found to be satisfactory.

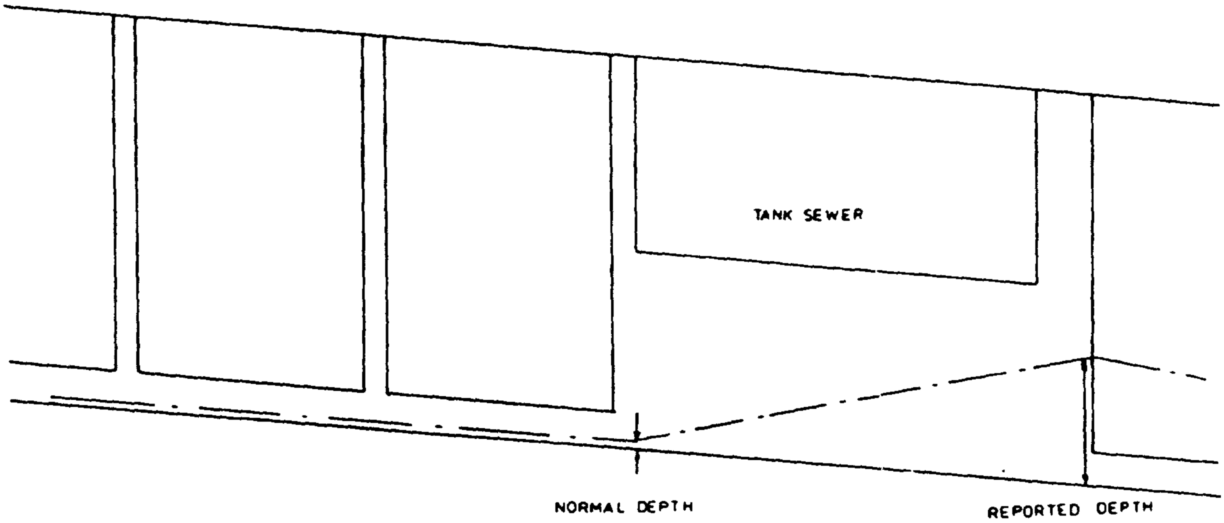
3.4 A 15-second timestep should be used as the major time increment (WaPUG user Note No 16). The tank sewer and the next pipe downstream should be specified as gauge pipes in the PCD. This will force the output of hydrographs which are useful in the subsequent analysis.

3.5 Some care is needed in the intepretation of water levels output by WaSSP. WaPUG User Note 12 shows that provided the tank sewer is not completely filled, the depth reported at the upstream end of the tank sewer refers to the normal flow depth, ie although WaSSP accounts for the volume stored by virtue of the level pool effect, it does not report the true upstream water level. Since tank sewers are by their nature very large in relation to the flow to be carried, the hydraulic gradient along the tank is usually small. The downstream water level may then be taken as a reasonable approximation of the upstream water level for the tank sewer itself. If substantial lengths of the upstream system are also surcharged, it may be necessay to allow for the hydraulic gradient required to pass the reported peak flow through these sewers. Head loss in manholes may be significant if flow velocities are high. Flow tables will give the required hydraulic gradient, and head loss can be estimated from

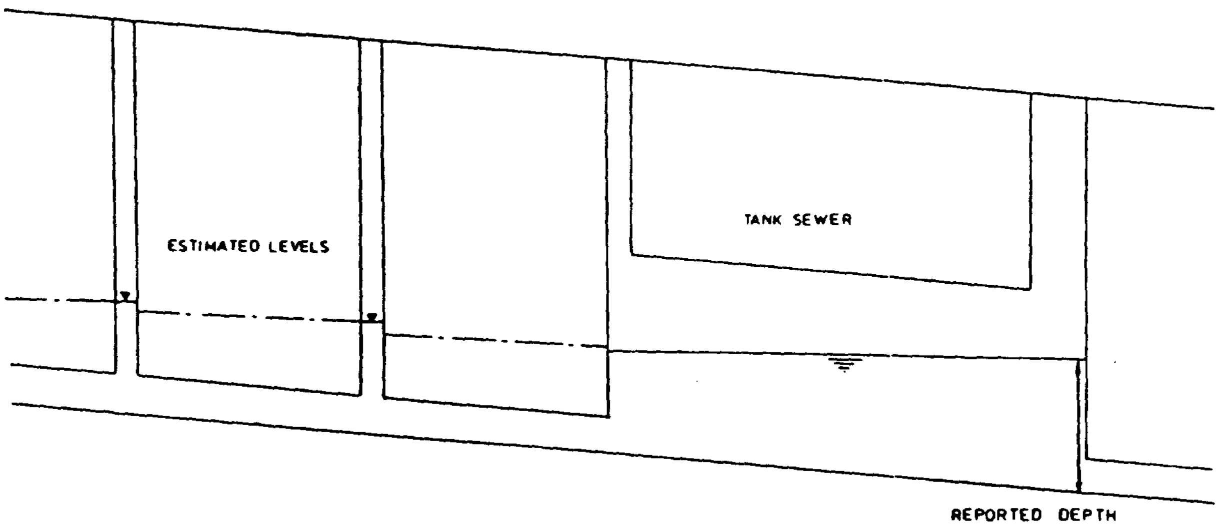
$$h_L = C \frac{V^2}{2g}$$

where V is the velocity in the incoming sewer and C is a coefficient varying between 0.25 and 2 depending on manhole layout [1].

This manual correction is vitally important for an understanding of the operation of the system. In a typical case where a tank sewer is to be used to reduce surcharge levels, reliance on reported water levels could lead to a false conclusion that the tank sewer had successfully eliminated flooding, whereas after manual correction it might be found that upstream water levels were dangerously high.



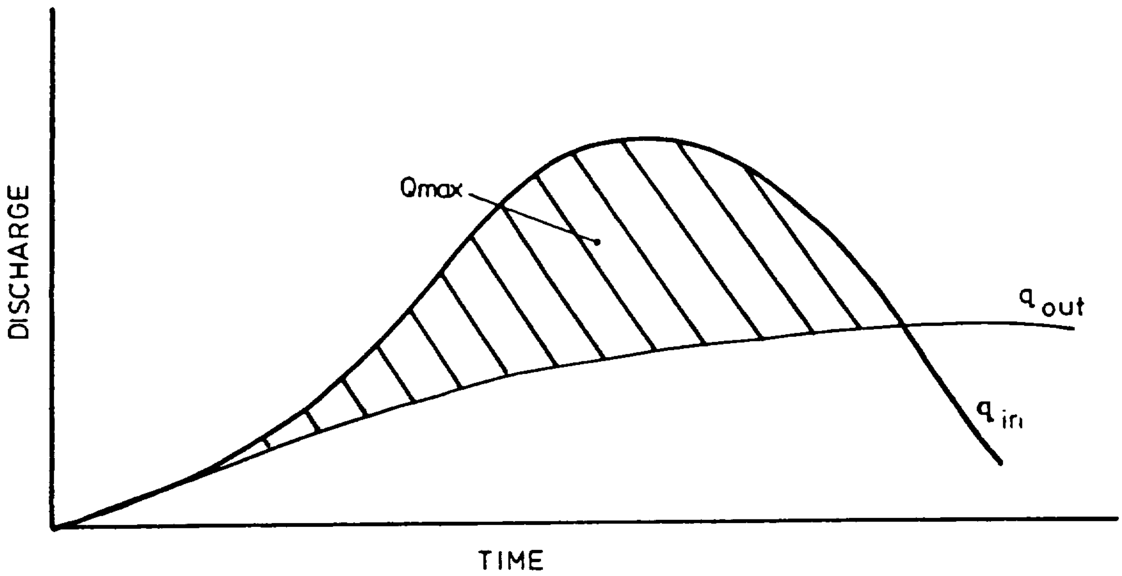
UNCORRECTED LEVELS



LEVELS AFTER MANUAL CORRECTION

3.6 During development of the design of a tank sewer it is an advantage to know the volume of storage occupied within the system. If an initial under-estimate of the storage requirement is made and the tank sewer becomes full, its effect on the performance of the system is difficult to interpret. It is better to over-estimate the volume required, thus ensuring that the tank sewer only partially fills. If the volume taken up can be easily estimated, the modelled tank sewer may be adjusted in length and diameter to give the actual volume required, and a re-run will check the effects of the altered head/storage relationship. The volume stored in the on-line tank is easily calculated from the product of base area and maximum depth above outlet invert. The head/storage relationship of a part-full circular pipe laid to falls is more complex and is not easily solved.

3.7 The peak storage requirement can be deduced in several ways. Plots of the inflow and outflow hydrographs will yield the storage volume as shown below.



Hydrograph can be plotted manually or directly from WaSSP results files using GURVIL. Area between the plots is measured with a planimeter. Note that GURVIL does not plot reverse flow correctly. For reliable results neither the tank sewer nor the throttle should have a contributing Ap in the SSD.

3.8 A quicker alternative is to use the graphical solution given as Appendix 2. Peak depth h at the outlet from the on-line tank is read from the WaSSP results file. A worked example illustrating the use of the graph is given. A strict analysis of storage taken up requires the volume occupied by the normal depth of flow to be deducted from the overall volume within the tank sewer. If required, the graph may be used to estimate the normal depth volume by setting h = normal depth (from WaSSP output if the tank sewer is specified as a gauge pipe and g = infinity (ie a flat pipe). Storage used may then be calculated by adding the volume within the on-line tank and the tank sewer and deducting the volume at normal depth.

4 CIRIA proposal on Scope for Control of Urban Run off

The Construction Industry Research and Information Association and the promoters of the CONFLO initiative are collaborating in a research consortium to review the factors which influence the utilisation of urban run-off control in the UK. CONFLOs 'Guidelines for the Design of Attenuation Storage' are eagerly awaited. The distillation of experiences gained nationally will be of great benefit to designers of storage facilities in the future. The report of Research Proposal 1089 is expected late in 1989.

Reference

(1) 'The effect of surcharging on discharge through a pipe'. P J Colyer. Chartered Municipal Engineer Vol 104 April 1977.

Acknowledgement

The author is grateful to R Jackson MSc CEng MICE FIHT FIAS MBIM, Director of Technical Services for Derby City Council for permission to use the case study material in this paper.

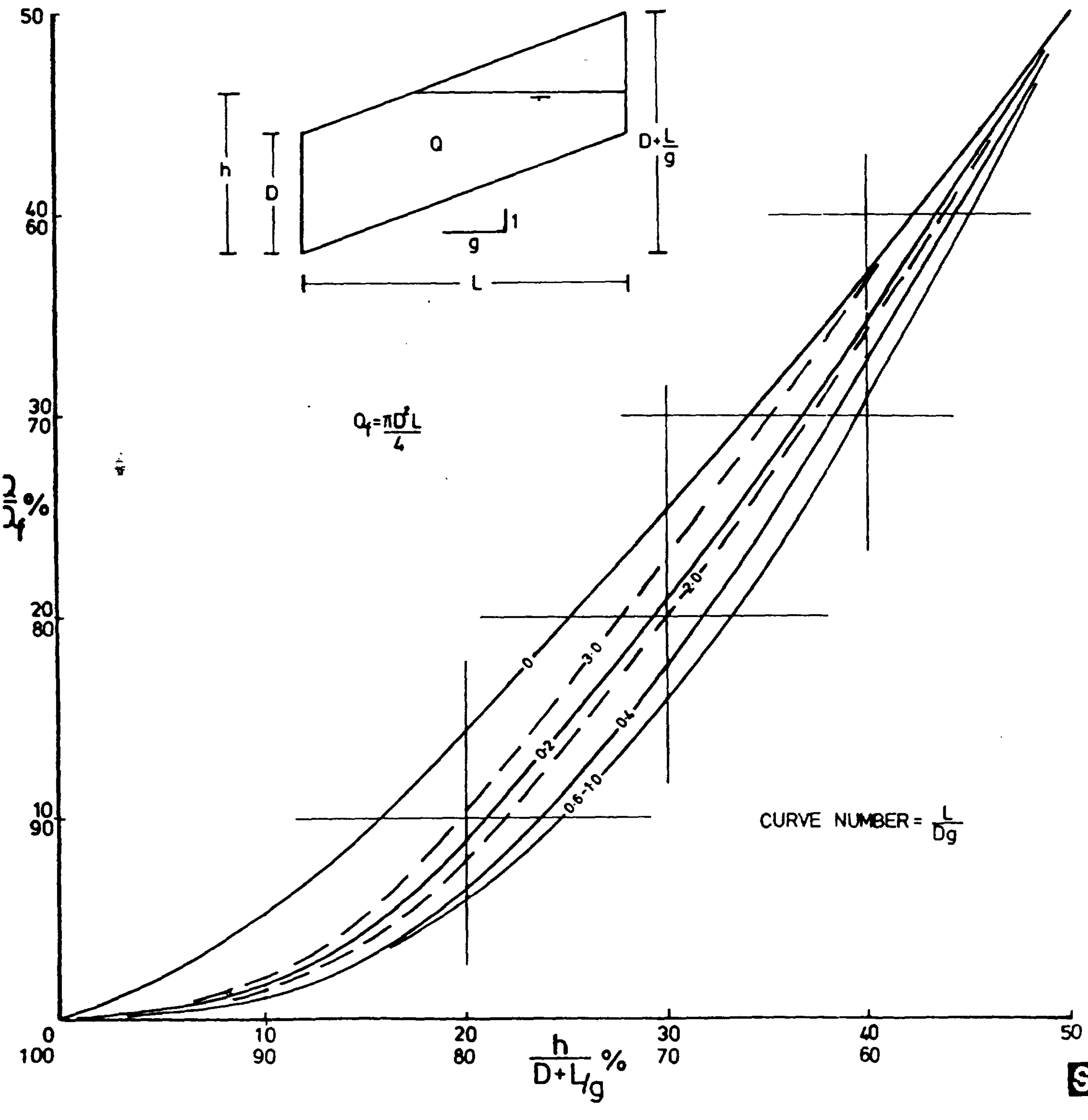
SAMPLE SSDs

```
TANK   B  -  MODELLED AS SEWER                                                      1
   5    1                                                                           2
  1 010 0   100100.00 97.000 96.000     225      0         1.000 10 10       .001 4
  1 020 0   100 99.00 96.000 95.000     225      0         1.000 10 10         -1 4
  1 030 0   100 98.00 95.000 94.000    1500      0         1.000 10 10         -1 4
  1 040 0   100 97.00 94.000 93.000     225      0         1.000 10 10         -1 4
  1 050 0   100 96.00 93.000 92.000     225      0         1.000 10 10         -1 4
 -1               96.00  92.00                                                     15
```

```
TANK   C  -  MODELLED AS ON-LINE TANK                                               1
   5    1                                                                           2
  1 010 0   100100.00 97.000 96.000     225      0         1.000 10 10       .001 4
  1 020 2   100 99.00 96.000 95.000     225      0         1.000 10 10         -1 4
   1          1                 1       133    93.990                               9
      0.8                                 2      97.00      2.4                    10
  1 040 0   100 97.00 94.000 93.000     225      0         1.000 10 10         -1 4
  1 050 0   100 96.00 93.000 92.000     225      0         1.000 10 10         -1 4
 -1               96.00  92.00                                                     15
```

```
TANK   D  -  MODELLED AS SMALL ON-LINE TANK CONTROLLING A LARGE TANK SEWER          1
   5    1                                                                           2
  1 010 0   100100.00 97.000 96.000     225      0         1.000 10 10       .001 4
  1 020 0   100 99.00 96.000 95.000     225      0         1.000 10 10         -1 4
  1 030 2   100 98.00 95.000 94.000    1500      0         1.000 10 10         -1 4
   1          1                 1         4    93.990                               9
      0.8                               2.0      97.00      2.4                    10
  1 040 0   100 97.00 94.000 93.000     225      0         1.000 10 10         -1 4
  1 050 0   100 96.00 93.000 92.000     225      0         1.000 10 10         -1 4
 -1               96.00  92.00                                                     15
```

STORED VOLUMES IN PART-FULL CIRCULAR PIPES LAID TO FALLS



SWK

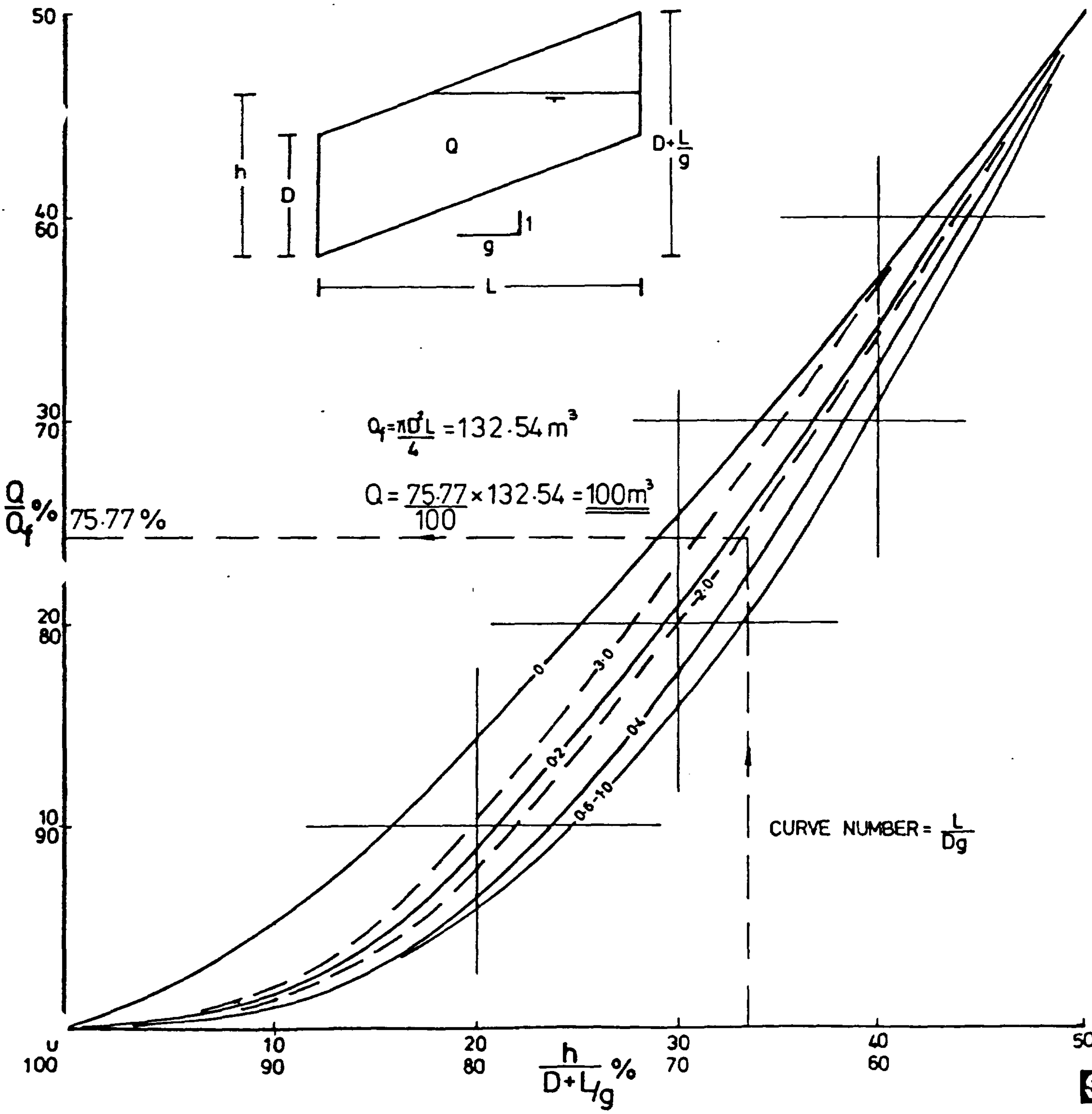
STORED VOLUMES IN PART-FULL CIRCULAR PIPES LAID TO FALLS

WORKED EXAMPLE

$D = 1{\cdot}50\,m$
$L = 75{\cdot}0\,m$
$g = 187$
$h = 1{\cdot}263\,m$

$\dfrac{h}{D + L/g} = 66{\cdot}4\,\%$

$\dfrac{L}{Dg} = 0{\cdot}267$



SWK

DISCUSSION NOTES

Technical Session 3
Paper 3.2 Discussion

D.Balmforth ; Sheffield City Polytechnic

Did you use different duration storms to give the largest volume ?

R.Long ; Scott Wilson Kirkpatrick

Yes.

****

P.Sunderland ; Weymouth & Portland B.C.

Could you elaborate on costs ?

R.Long

We used WASSP-CST and ascertained that the storage option would be around 40% cheaper than conventional resewerage.

****

M.Osborne ; HR

I am pleased to hear that somebody is using WASSP-CST. Does anybody else feel it needs further development ?

R.Long

We use it frequently and usually get good correlation with tenders returned. The program always assumes levels specified are soffit levels.

D.Beale ; Howard Humphries

We have also used it but have found its estimate to be only 50% of the final cost. Users must be aware of what it doesn't take account of. It would be useful if HR could allow us to vary more Baxter indices than just the one at present.

D.Balmforth

Cost comparison is the key point, and not absolute cost. It would be useful if it could cost rectangular tanks.