Detention Tanks –
Modelling Exceedance Failure

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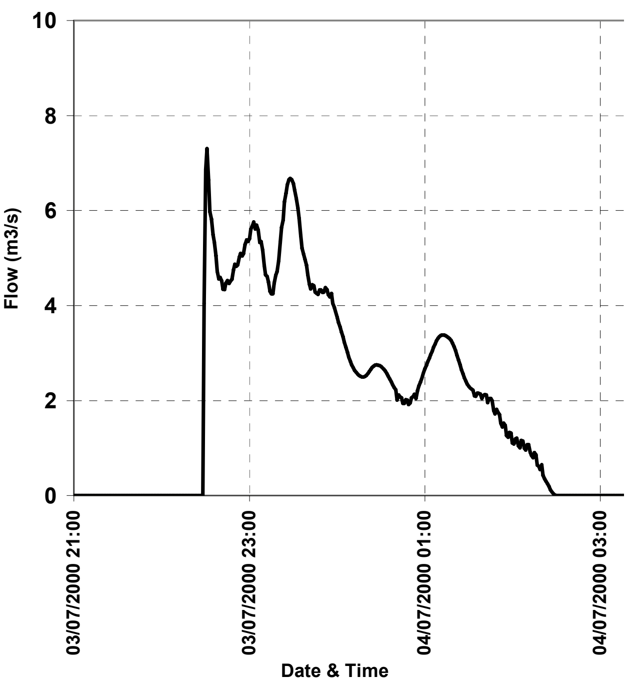
1. Introduction

There is a growing recognition that in the future there will as a result of climate change be more days with heavier rainfall. It is also becoming recognised that it is unlikely that storage options will be sustainable and that increased conveyance capacity will be required to solve flooding problems. However, we now have a legacy of detention tanks which will become increasingly overloaded and their failure mechanism will need to be better understood.

This paper explores how detention tanks behave during storm events which are greater than those for which the tanks were designed (ie exceedance storms). On-line and off-line detention tanks behave in different ways, generally have different control strategies and usually have different overflow configurations and therefore their behaviour during exceedance storms can be notably different.

2. Reasons for Investigations

The reasons why the behaviour of detention tanks and overflows during exceedance events became of particular interest was the spill hydrograph which was simulated for the overflow from a large detention tank which we were investigating. We were using actual recorded rainfall data for this study and the particular storm used in this analysis had a return period of 1 in 126 years. The hydrograph shown to the right shows the hydrographs for one of the two similar overflows on this tank. The overflow reached its maximum spill rates of 7,300 l/s within a period of 3 minutes. The second overflow discharged at the same time but with a slightly higher peak discharge of 10,580 l/s.



The big question is can the model simulations in these circumstances be relied upon? Such massive flows occurring almost instantaneously and probably without any warning could be the cause of considerable alarm and concern about the potential risks to life. Fortunately in this case the spills occurred during the early hours of the morning when it is unlikely that any members of the public would have been in the vicinity but this would not always be the case. The risk was also found to be even greater in this case when it was determined that the spills occurred some 2½ hours after the peak rainfall.

3. Design Criteria for Detention Tanks

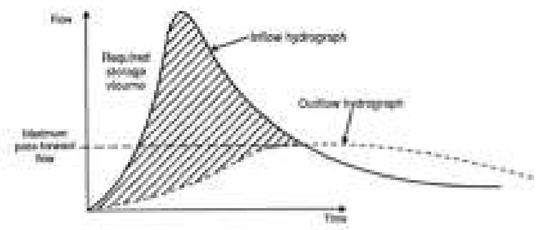
Detention tanks are constructed for three main purposes:-

- To reduce the frequency at which CSO's spill or
- To reduce the flooding risk to properties;
- To control the discharges from particular areas (specifically open balancing ponds at new housing developments).

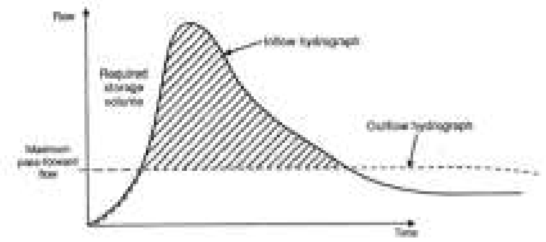
In the first case the design criteria is to provide sufficient storage that the spills at a certain CSO remain within a predetermined spill frequency (eg 3 times per Bathing Season for Bathing Waters) or to achieve certain water quality objectives in the receiving water (eg Shellfish Waters). Detention tanks built in these circumstances are not always required to fulfil any particular function during major storms though it is desirable that they do not cause an increase in flood risk to properties. Unfortunately the performance of such tanks during major storms is not always modelled.

In the second category the detention tanks are sized to prevent flooding at specific locations up to certain pre-determined return periods. Many such tanks are nowadays built to alleviation flooding for 1 in 30 year or 1 in 50 year return periods. However, many tanks constructed in the late 1980's and early 1990's were probably only designed for 1 in 10 year return periods.

The third category is predominantly used for new housing and other developments where the intention is to keep discharges to local watercourses to about the same rate as would have happened without the development. Many of these are designed as open ponds sufficient to contain 1 in 20 year floods but with sufficient additional freeboard that with 1in 50 year floods there are no properties flooded.



On-line Storage



Off-line Storage

The next consideration is whether detention tanks should be on-line or off-line. The hydrographs shown to the left illustrate the differences between on-line and off-line tank performance. With on-line tanks the maximum pass-forward flow is not achieved until towards the end of the event when the maximum water level in the tank has been reached. By contrast with off-line tanks the maximum pass-forward flows are achieved as soon as flows are spilled into the off-line tank from the main sewer because the water level reaches and remains at the weir level.

There is guidance given to designers on different types of flow control devices which can be used in respect of the pass-forward flow (eg Hydrobrakes, orifices, control pipes etc). There is also practical advice given to designers on providing bypass facilities (eg penstocks) for use if the main control becomes blocked.

However, there is very little guidance given to designers on what to provide to deal with exceedance events. There is very little discussion in most of the standard references on what should be provided in the way of overflow weirs etc or indeed whether tanks should normally be fitted with any overflow devices.

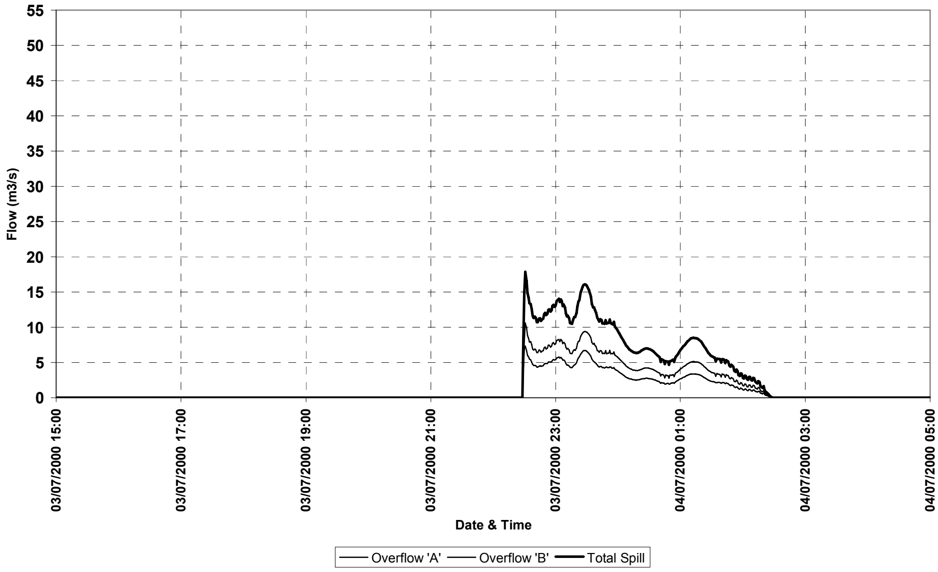
4. Rainfall

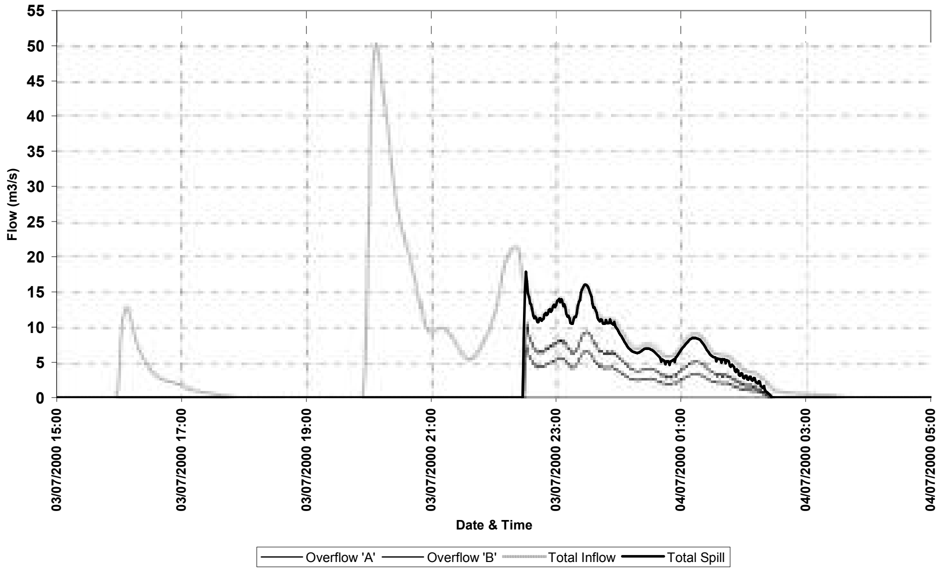
The changes which can be expected in the future in rainfall have been predicted by a number of organisations for a series of different scenarios depending on how quickly the earth is affected by global warming. Many of these predictions have appeared to gloss over the fact that we have been experiencing extreme rainfall for decades. The most well known recent example was the storm at Boscastle in Cornwall in August 2004.

Much of the subject of this paper stems from a project which Richard Allitt Associates were undertaking investigating the operation of overflows from a very large off-line detention tank. For this study historical tipping bucket rainfall data was obtained for the catchment covering a period of over 12 years. From this data it was possible to obtain 10 complete years of data. Within this 10 year period there was one storm with a return period of 1 in 126 years[†], one storm with a return period of 1 in 78 years and three storms with return periods in excess on 1 in 4 years. There were also some years with below average rainfall and without even a 1 in 1 year individual storm. Surprisingly the two largest storms both occurred within the same year.

5. Overflows in Extreme Events

Referring back to the study discussed at the start of this paper with the overflow which reached a discharge rate of over 7,000 l/s within 3 minutes the two hydrographs below put this in context. The detention tank in this study was a very large one providing over 150,000m^3 of storage. The upper hydrograph only shows the discharges from the two overflows on the tank and in the slightly heavier line shows the total spill from the two overflows.





The lower hydrograph also shows the total inflow to the tank which is actually from a number of points. The maximum inflow to the tank was nearly 50,000 l/s.

The lower hydrograph also shows very clearly that up until such time as the overflows start to spill the detention tank has stored a considerable volume (in this case 150,000m^3) but as soon as the tank is full any further inflows are very quickly routed through the tank and out of the overflow. It can be seen that once the overflows reached the total spill of nearly 18,000 l/s the spill flows match exactly the inflows (except for a small volume being pumped out of the tank).

To assess whether the modelling of this tank under these conditions can be relied upon the simulations were repeated with a variety of different simulation timesteps and a variety of different results timesteps. Normally, reducing simulation timesteps will overcome any model stability problems but changing the simulation timestep made virtually no change to the simulation results and it has therefore been possible to conclude that the sudden operation of the overflows is not a model stability phenomenon but would occur in reality. It was found that having results timesteps at 5 minute intervals could tend to reduce the potential severity of the problem and it was found that results timesteps of 1 minute or ½ minute was preferable.

The questions which arise from this are numerous but fundamental questions are:-

- Should detention tanks be provided with overflows ?

[†] Based on the Flood Estimation Handbook (FEH)

- If overflows are provided at detention tanks what design criteria should be used ?
- Is current modelling software capable of adequately simulating extreme events ?

The immediate conclusion which we reached when we considered these questions was that in most cases the answers depended on the particular circumstances and whilst overflows might be desirable in some locations they may not be in other locations. After some deliberation we concluded that the best approach was to consider and investigate how a detention tank would fail or in other words how it would perform during an exceedance event. An exceedance event being any event which exceeds the capacity of the tank.

6. Designing for Exceedance

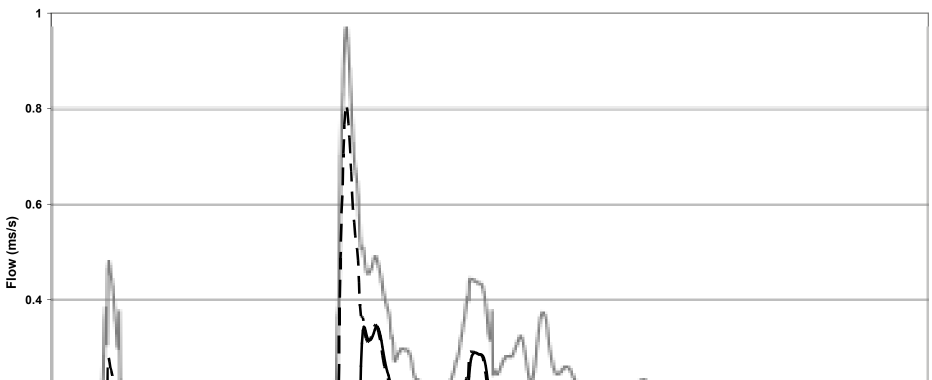
There is a CIRIA research project (Project Ref RP699) "Designing for exceedance in urban drainage systems – good practice" which is due to be published shortly. This project has investigated how during exceedance events when the minor flow pathways (typically the underground sewer network) become overloaded the major flow pathways (typically urban streets) come into operation and convey the floodwater. Much of the project investigated suitable design aspects and criteria for these major flow pathways.

To assess how well the standard hydraulic modelling software can be used to simulate exceedance events we carried out investigations on two detention tanks; one on-line and one off-line. For simplicity the same 1 in 126 year recorded rainfall event described earlier was used.

Case Study 1 : Off-Line Tank with Weir Overflow

The detention tank used in this example is an off-line tank which was constructed at a CSO location. Pass forward flow is controlled by a Hydrobrake and there is a single high side weir overflow which spills into the off-line tank. At the far end of the tank there is another weir which spills to the watercourse. The geometry of both weirs is identical in terms of crest level and length.

Simulations with minor storms demonstrate that spills to the watercourse have been eliminated by the detention tank. For the exceedance event it was found that the model satisfactorily simulated the flow over the weir into the tank, the filling of the tank and then the spill from the tank once the tank was full. This is shown in the hydrograph to the left. The heavy dashed line shows the spill from the sewer into the detention tank and this peaks at about 800 l/s but it is not until nearly 20 minutes later that the overflow from the tank into the watercourse starts to spill. These hydrographs also show that once the tank has filled the spill flow out of the tank exactly matches the inflow. In



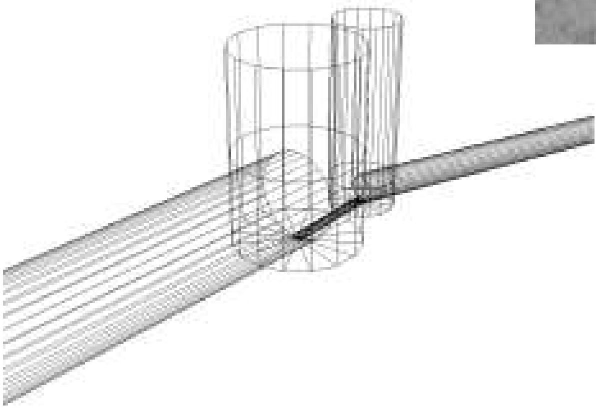
this case the spill out of the tank peaks at about 340 l/s which is considerably lower than the 800 l/s spill into the tank.

This has shown that detention tanks where a large weir is installed can be satisfactorily modelled for exceedance events simply by modelling the weirs as they exist (or as designed when checking designs). Where the weirs are of insufficient size to pass the required flows and there is resultant flooding from the sewers this does not necessarily hold good for reasons which are explained below.

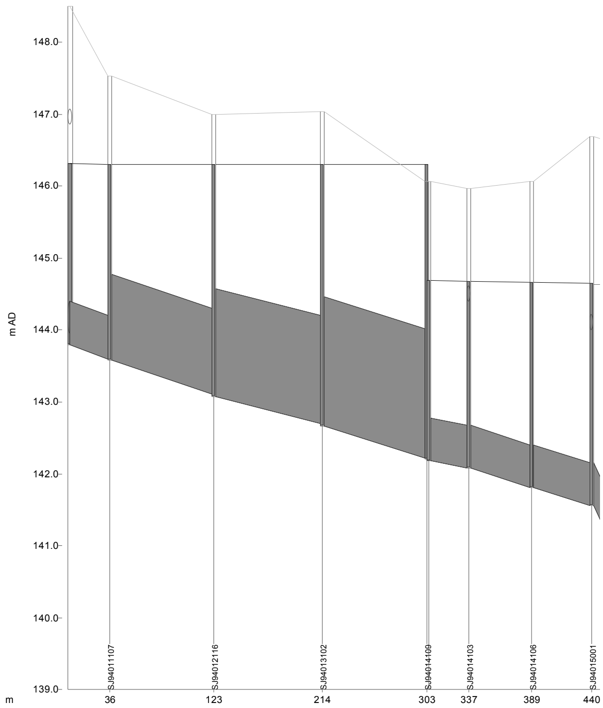
Case Study 2 : On-Line Tank without overflow

This example is an 1800mm dia on-line tank sewer on a partially combined sewer system. There is a 600mm dia incoming sewer and a 600mm dia outgoing sewer but with a 175mm dia orifice plate at the outlet from the tank. There is no overflow provided at the tank and there is no overflow out of the sewer system at this location. The tank was provided to attenuate flows and to alleviate flooding at a downstream location.

The arrangement of the tank as modelled is shown diagrammatically below.

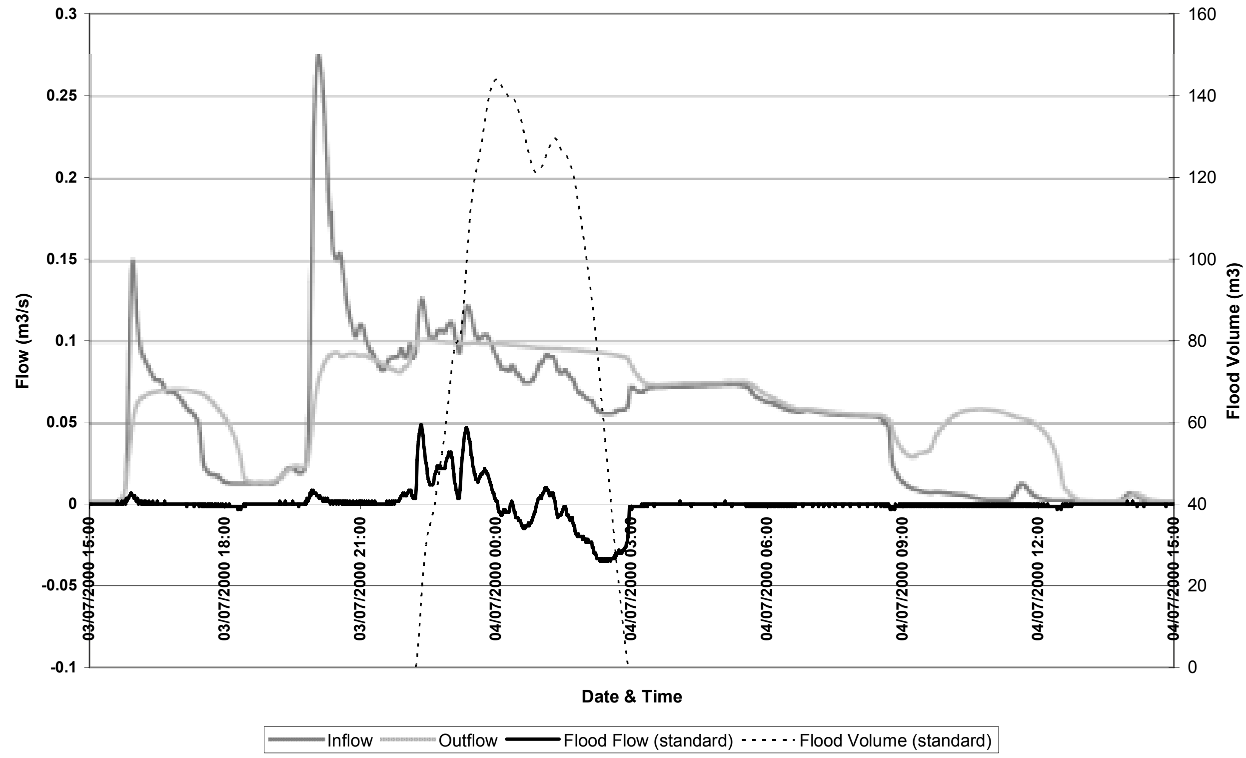


With standard design storms up to the limit of the design criteria (1 in 50 year storms in this case) all the flow is contained within the system and there is no flooding simulated. However, with the exceedance storm (1 in 126 year return period) there was flooding simulated from the manhole at the downstream end of the tank sewer as shown in the longitudinal section below.



This simulation showed that there was about 140m^3 of flooding from this manhole over a period of about 100 minutes. The model was set with the flood type as 'stored' so that the floodwater returned back into the sewer when there was capacity available. The hydrograph overleaf shows the inflow, the continuation flow and the flow into flood storage. Also shown on this hydrograph is the volume of storage (using the right hand axis) with the dashed line. The rate of flow into and out of storage is shown with the heavy solid line and it can be seen that the peak flooding flow rate is about 50 l/s. Whilst this flow rate appears quite modest there was concern that maybe such a flow rate was unrealistic in terms of actual flooding mechanisms.

The photograph overleaf shows a flooding location where the surcharge pressure in the sewer is sufficient to force some water out through gaps around the manhole cover it is insufficient to lift the cover and allow large flow volumes to exit the sewer.

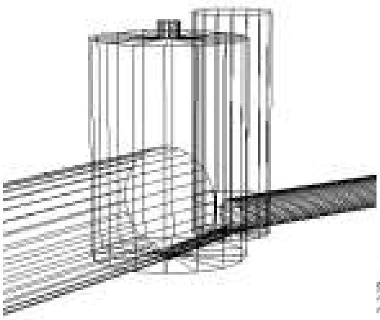
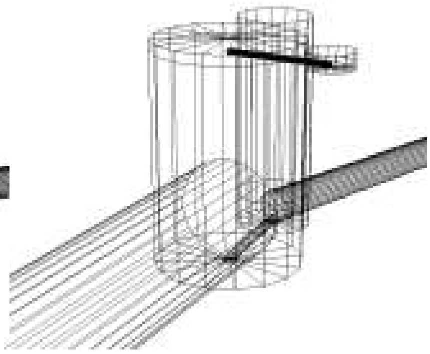
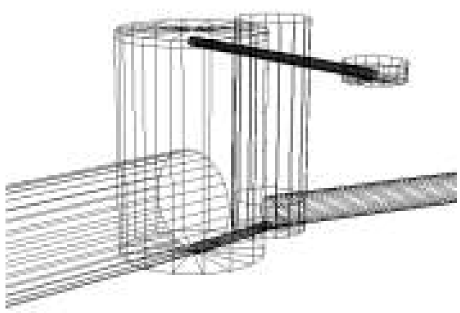


Whilst this particular case is not extreme it was concluded that maybe the current normal modelling techniques were allowing floodwater out of the sewers too quickly and allowing it to return at too high a rate (especially when it might be only two road gullies draining the floodwater back into the sewer).

Three alternative methods were experimented with to attempt to reduce the flooding flow rate out of the system and the rate at which floodwater can re-enter the system. These were:-

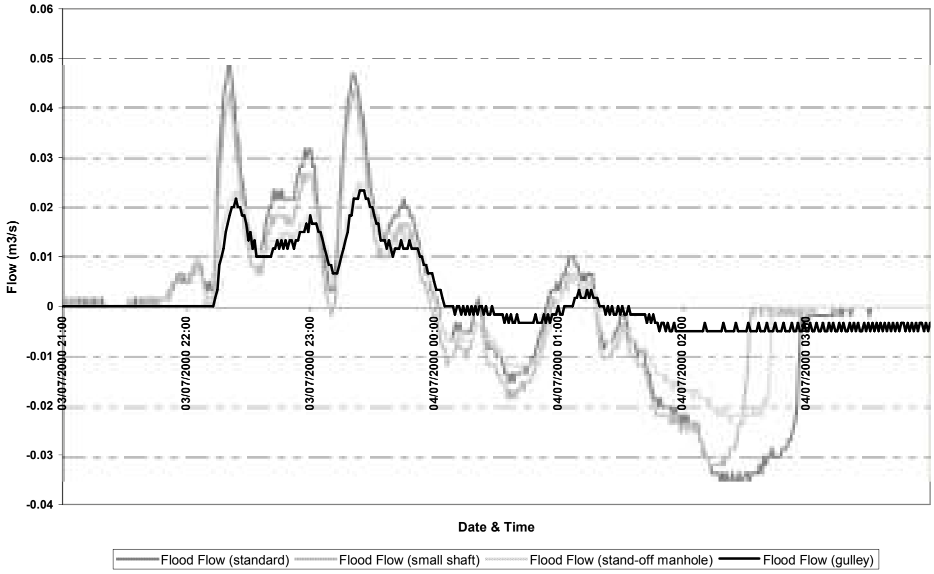
- Using a small shaft at the manhole;
- Using a stand-off manhole with a small linking conduit between;
- Using a stand-off gulley with a head-discharge relationship .

Modelling of these 3 options are shown diagrammatically below from left to right respectively.

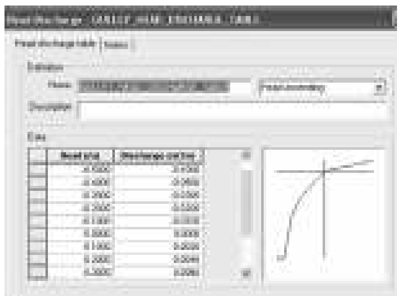




The hydrograph below shows the rate of flow of the flooding for these methods. The standard arrangement (as in the previous hydrograph) had a flow rate of nearly 50 l/s both from the sewer into storage and also back from storage into the sewer. The arrangement with the small shaft had the shaft part of the manhole reduced from 5.2m^2 to 0.1m^2 but this approach only reduced the peak flow rate to about 44 l/s.

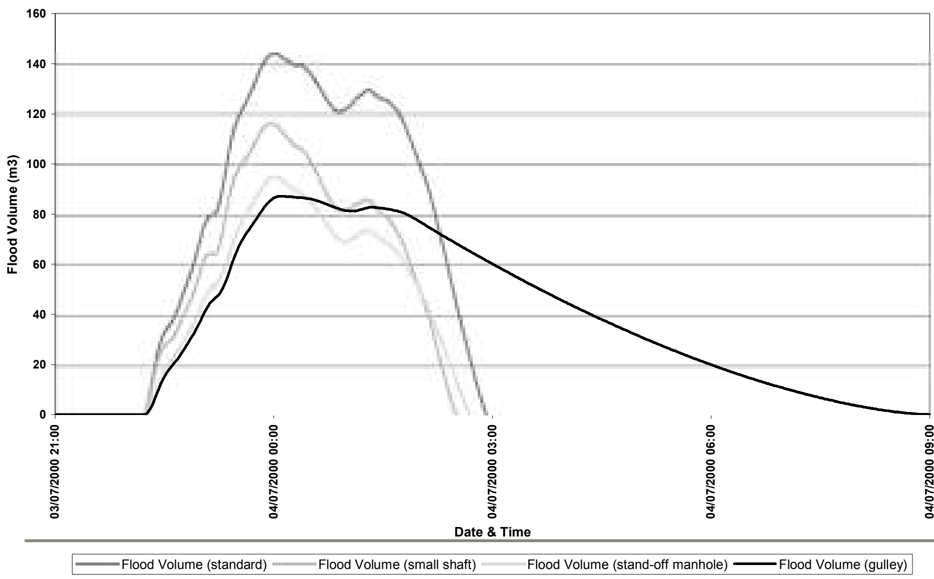
The stand-off manhole was connected to the main manhole with a 100mm dia pipe and it was this pipe which controlled the flow rates. The main manhole was changed to a ‘sealed’ flood type so that all of the flooding had to be via the stand-off manhole.



The approach using the stand-off gully utilised a feature of the latest version of Infoworks CS (v6.0) which allows for an additional flooding type. This allows for user defined head / discharge relationships and user defined floodable area relationships. By setting the flood type of the main manhole to ‘sealed’ all of the flooding is routed through the stand-off gully. Various different head / discharge relationships were attempted and it was found by having a negative head and negative flow rate this could control the flooding phase whilst with positive heads and positive flow rates the flood drain down rates could be controlled (see head-discharge table to the right). Attempts were made such that at smaller internal pressures where the flows are forced out of small gaps and holes in the manhole cover (as the photograph) whilst at greater internal pressures the manhole cover is lifted and the flooding flow rate is larger. The drain down rate was set quite low to illustrate the differences which could be achieved. In the hydrograph above the flow rate for this approach is shown with the solid heavy line. The maximum flow rate for the flooding was brought down to about 23 l/s whilst the drain down rate only achieved about 4 l/s.



The hydrograph below shows the flood volumes for the different approaches taken. It can be seen more clearly what a difference the stand-off ‘gully’ approach can make. Not only is the total flood volume the least but also the rate of flooding is the lowest of all the techniques evaluated but what is most striking is the far more gradual drain down of the flooding back into the sewer system. The flexibility of the ‘gully’ approach will enable the particular circumstances of each case to be modelled in detail. It would however, be useful to have some standard or typical head / discharge relationship for the more common situations.



From the modelling work done on this case study it can be concluded that the modelling produces a realistic simulation of what might actually occur though there is concern that the flow rates of the flooding from the manholes into the flood storage maybe unrealistically high. This could perhaps create the situation where all the flooding is simulated at one location rather than from several manholes.

7. Can the onset of flooding be softened ?

One of the undoubted but little known features of detention tanks fitted with overflows is that once the tank is full the spill rates match almost exactly the inflow rates but they come into operation very quickly. At the start of this study we considered two overflows which achieved discharge rates of 7,000 l/s and 10,000 l/s within a period of 3 minutes. By studying the inflow hydrograph in relation to the spill hydrograph it can be appreciated why this situation arises. In some situations there could be safety implications in overflows coming into operation so rapidly. It would be worthwhile assessing whether there are any measures which could be taken to ‘soften’ the onset of overflows discharging so that people would receive some form of warning about the imminent discharges.

A technique of using stepped weirs rather than just a single weir was investigated. This has implications as the storage volume required is measured up to the lowest point of the overflow weir and there may also be maximum water level criteria to meet to avoid flooding. In practice there may therefore only be a limited range within which the weir could be stepped.

The example given at the start of this paper was re-examined with stepped weirs at both overflows with the lowest weir set 100mm below the original setting, a shorter length at the original level and then two further levels 200mm and 400mm higher than the original level. Using the same 1 in 126 year return period storm gave the results showed virtually no change. Because the main storage in this example is in the form of a deep tunnel there is only the storage provided in the access shafts between the lowest weir and the highest weir level – this is a very small volume compared with the size of the tank.

Also the example used in Case Study 1 was used with the final overflow from the tank set with a stepped weir. It was expected that some differences would be found with this example as the levels of the weirs were still within the storage depth of the tank (ie there was a reasonably large difference in stored volume between the lowest weir and the highest weir). Surprisingly, there was very little change in the discharge from the overflow which is probably accounted for by the fact that whilst there was some storage available between the lowest and highest weirs this volume was insignificant compared with the inflows into the tank during this part of the event.

8. Conclusions and Recommendations

The conclusion from this investigation is that in most aspects the currently available modelling programs are adequate to simulate the performance of detention tanks under exceedance events.

It has also been concluded from this investigation that if detention tanks are fitted with overflow weirs the discharges over these weirs arising from exceedance events will rise very rapidly and reach very high discharge rates within a very short time. These can have significant safety implications in some locations which will need to be considered at the design stage of detention tanks.

There is some concern that maybe the rates at which flooding is simulated as occurring from manholes could be unrealistically high. A number of techniques have been evaluated to overcome these concerns and it has been found that recently introduced techniques in Infoworks are sufficiently adaptable to enable the flooding rates to be modelled at whatever rates the modellers considers appropriate.

Arising from this study the author considers that the following recommendations are appropriate:-

- Where detention tanks are installed or are being designed, their performance under exceedance events should be modelled to assess how they perform;

- The exceedance events used to assess detention tank performance should be ‘real’ storms with several peaks and not just synthetic, single peak design storms;

- The decision on whether detention tanks should be provided with overflow weirs should remain with designers but if they are provided, the weirs and spillways should be designed to safely discharge the maximum inflow rate which the sewer system could deliver to the tank. If overflow weirs are provided then consideration should be given to what effects there might be if the weir suddenly comes into operation. Where necessary measures to ‘soften’ the onset of discharge should be evaluated and modelled;

- Where overflow facilities are not provided from detention tanks then the flooding which would result from exceedance events should be modelled. Where necessary advanced modelling techniques should be used if the flooding rate is unrealistically high;

- Providing the modelling is undertaken carefully, to a high standard and following the latest advice the simulated results can be treated with a high degree of confidence.