Accurate modelling of high-early-discharge onsite detention storages *

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SUMMARY: *High-Early-Discharge HED onsite detention storages can be used to reduce flood peaks in urban catchments. They consist firstly of a small chamber which rapidly fills, so that outflows from the storage rapidly rise to the permissible site discharge. As this occurs, the larger main chamber slowly fills. The design of HED storages, using two chambers with flow passing back and forth between them, makes analysis of their behaviour complex. A complete solution requires that volumes and water levels be monitored in both the first and main chambers at all times as the hydrograph passes through the storage. There may be advantages in replacing the complete solution by standard flood routing techniques, using a single chamber with a storage-discharge relation which approximates the behaviour of the two storage chambers. This paper first develops a complete solution for HED onsite detention storages, then uses it to evaluate three approximate flood routing methods.*

NOTATION

H_1	= elevation of main chamber in above ground HED storage (m)
Hw	= elevation of internal weir in below ground HED storage (m)
PSD	= permissible site discharge (m^3/s)
SSR	= site storage requirement (m^3)
I(t)	= inflow to storage at time t (m^3/s)
Q(t)	= outflow from storage at time t (m^3/s)
Hf(t)	= water level in first chamber at time t (m)
Hm(t)	= water level in main chamber at time t(m)
Sf(t)	= volume stored in first chamber at time t(m^3)
Sm(t)	= volume stored in main chamber at time t(m^3)
Sf(max)	= maximum volume stored in first chamber (m^3)
Sm(max)	= maximum volume stored in main chamber (m^3)
Qw (t)	= flow from first to main chamber over internal weir at time t (m^3/s)

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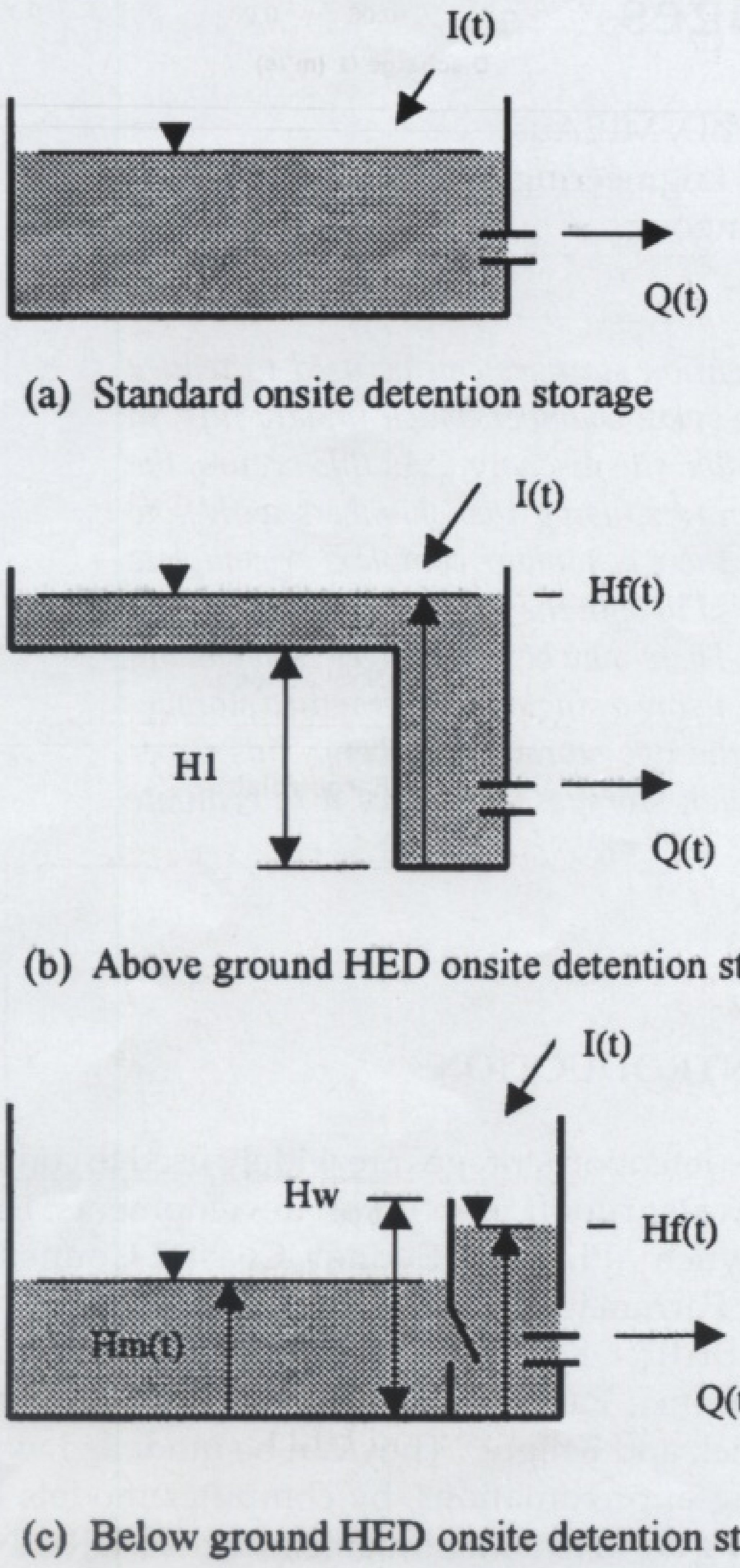
1 INTRODUCTION

Onsite detention storages are widely used to control stormwater runoff from urban developments: Lees and Lynch[1], Phillips[2], Sydney Coastal Councils[3], Upper Parramatta River Catchment Trust[4], Bewsher and Still[5], Kandasamy and O'Loughlin[6], Kandasamy, Patarapanich and Loncar[7], Ribbons, Warwick and Knight[8]. They can be modelled, with varying approximations, by computer models for flooding in urban catchments, such as DRAINS, MOUSE, XP-RAFTS and WBNM2002.

Onsite detention storages are basically of two types, standard and high early discharge (HED). Standard storages consist of a single chamber with one or more pipe or orifice outlets, and can be modelled by standard reservoir flood routing procedures (figure 1a). There are two types of HED storages, above ground and below ground. Above ground HED storages consist of a small pit which forms the first storage chamber, plus a larger main chamber (figure 1b). Runoff from the site rapidly fills the first chamber, so that discharges out of the storage rapidly rise to the permissible site discharge PSD. When the water level exceeds H_1 the main chamber starts to fill. The large volume of the main chamber causes the water level to remain relatively constant, and consequently the discharge remains near to the PSD as the main chamber fills. When the storm ends, water first drains from the main chamber. During this time the water level remains relatively constant and the discharge remains near to the PSD. When the water level falls below H_1, the small volume of the first chamber results in a rapid decrease in water level and a rapid fall in discharge out of the storage.



(a) Standard onsite detention storage

(b) Above ground HED onsite detention storage

(c) Below ground HED onsite detention storage

Figure 1: Types of onsite detention storages

In the above ground HED storage of figure 1b the water surface is continuous across the first and main chambers. While there is an abrupt discontinuity as $H_f > H_1$, a single valued storage-discharge relation applies to both the rising and falling parts of the hydrograph. The below ground HED storage of figure 1c is considerably more complicated because the first and main chambers are disconnected, with flow passing from the first to main chamber via an internal weir, and returning via a one-way valve. The two chambers have different storage characteristics, and the operation of each is conditional on the current state of the other chamber. This results in different characteristics for the storage depending on the water levels in the two chambers, and produces a looped storage-discharge relation.

In the below ground HED storage, the first chamber receives the runoff from the site. It has a small volume and rapidly fills to its design water level, at which time it releases water at the design PSD. When water in the first chamber reaches the internal weir crest level Hw, water flows from it into the main chamber. The water level in the first chamber, and the PSD, are maintained while the main chamber fills. The volume of the main chamber necessary to store this overflowing water is the site storage requirement (SSR).

Some advantages of HED over the standard onsite detention storage are:

- a lower SSR is needed to reduce runoff peaks to the specified PSD value
- because the first chamber of the HED always fills to its design water level, the same PSD is released for a wide range of storms
- the outlet orifice size is readily determined from the selected design water level in the first chamber and the design PSD, using standard orifice or pipe hydraulics
- the SSR is readily determined, being equal to the volume of the inflow hydrograph in excess of the PSD.

Offsetting this, they are more complicated to construct and have more components which could fail hydraulically. In addition, the operation of HED storages is more complex than standard storages, making it more difficult to model them. This paper has two objectives, to develop procedures for accurate modelling of HED storages, and to evaluate the feasibility of modelling them using several approximate methods.

2 ABOVE GROUND HED STORAGES

2.1 Operating details

Stage 1 – first chamber filling

After the storm starts, the first chamber fills rapidly and the outflow quickly increases to the PSD (which occurs when Hf (t) just exceeds H_1), figure 1b. At each time t the mass-balance equation is:

$$I(t) - Q(t) = \frac{dS_f(t)}{dt} \tag{1}$$

Whenever Hf (t) <= H_1, the small volume of the first chamber results in rapid rises and falls in Hf (t) and Q(t) as the inflow I(t) varies. During this time the outflow Q(t) is close to the inflow I(t).

Stage 2 – main chamber filling

When Hf (t) > H_1, the main chamber fills. During this time the water level remains relatively constant, at a little greater than H_1, and the outflow remains close to the PSD. The mass-balance is:

$$I(t) - Q(t) = \frac{d\left(S_f(t) + S_m(t)\right)}{dt} \qquad (2)$$

Stage 3 – main chamber emptying

As the inflow hydrograph decreases water drains from the main storage. The water level falls only slowly, remaining near to H_1, and the outflow also remains close to the PSD. The mass balance is given by eq 2.

Stage 4 – first chamber emptying

When Hf (t) < H_1, the water level and discharge Q(t) fall rapidly. The mass-balance is given by equation 1.

2.2 Modelling above ground HED storages

Because the water levels in the first and main chambers are continuous whenever Hf (t) exceeds H_1, a single valued storage-discharge relation applies to both the rising and falling hydrograph limbs. Figure 2 shows the relations for a PSD of 0.1 m^3/s and H_1 = 0.95 m. The elevation-discharge relation is dependent on the outlet dimensions, in this case two 150 mm orifices. The elevation-storage and storage-discharge relations show the abrupt change in storage volume as Hf (t) exceeds H_1. At point 1, Hf = H_1 and S = Sf(max). At point 2, S = Sf(max) + Sm(max) = SSR and Q = PSD.

Because a single valued function applies to both rising and falling limbs, standard reservoir flood routing procedures can be used to obtain accurate results. The only precaution is that a small time step may be needed to avoid oscillations in the hydrograph at the abrupt change in storage characteristics when Hf (t) is near to H_1.

As an example, the method was applied to a 1 hectare fully developed site. Runoff from 20 year, 90 minute and 2 year, 90 minute design storms (zone 1 temporal pattern) were used as the inflow hydrograph to the storage. Average intensities for these storms are 31 and 18 mm/hour respectively.

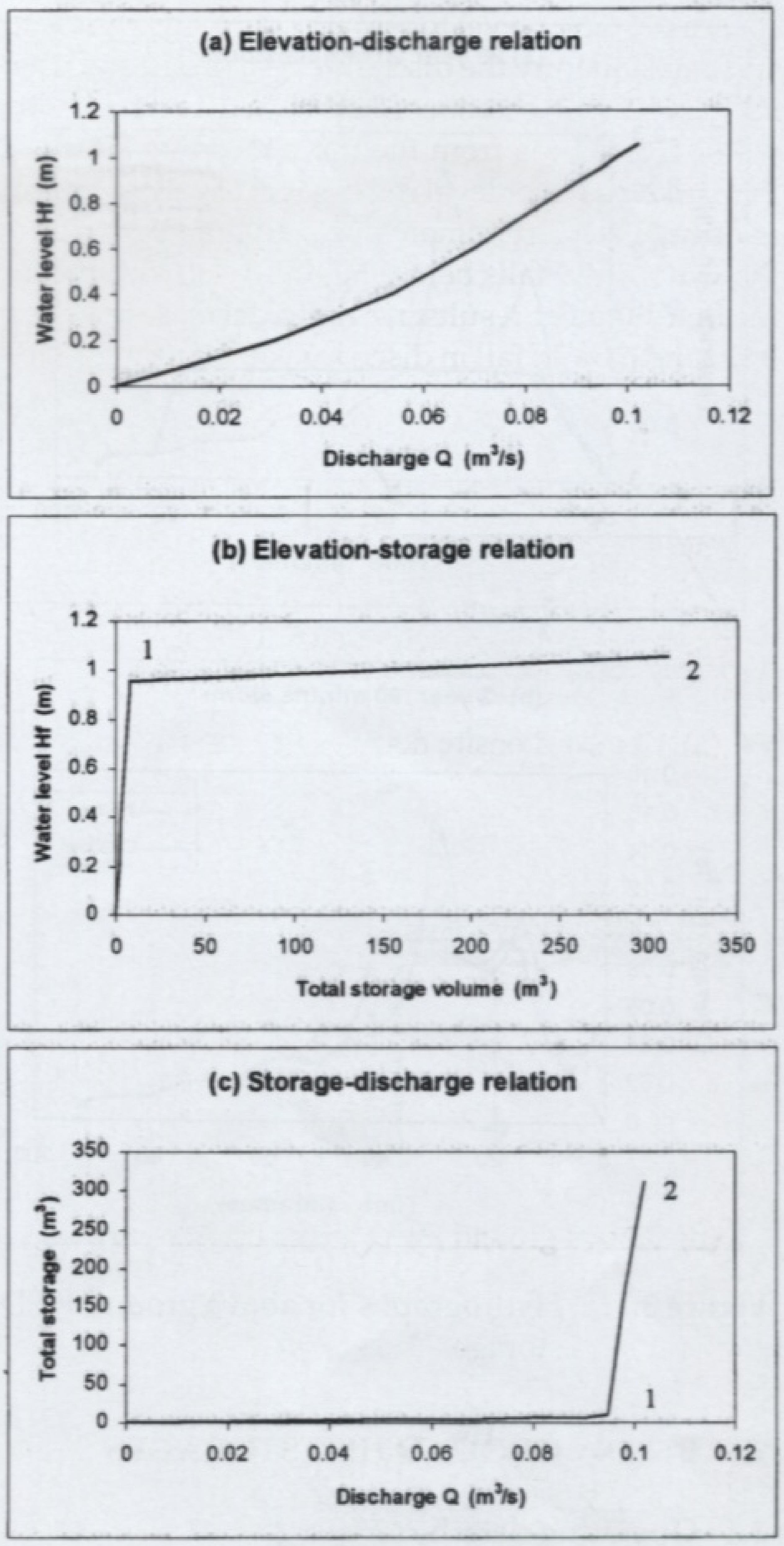
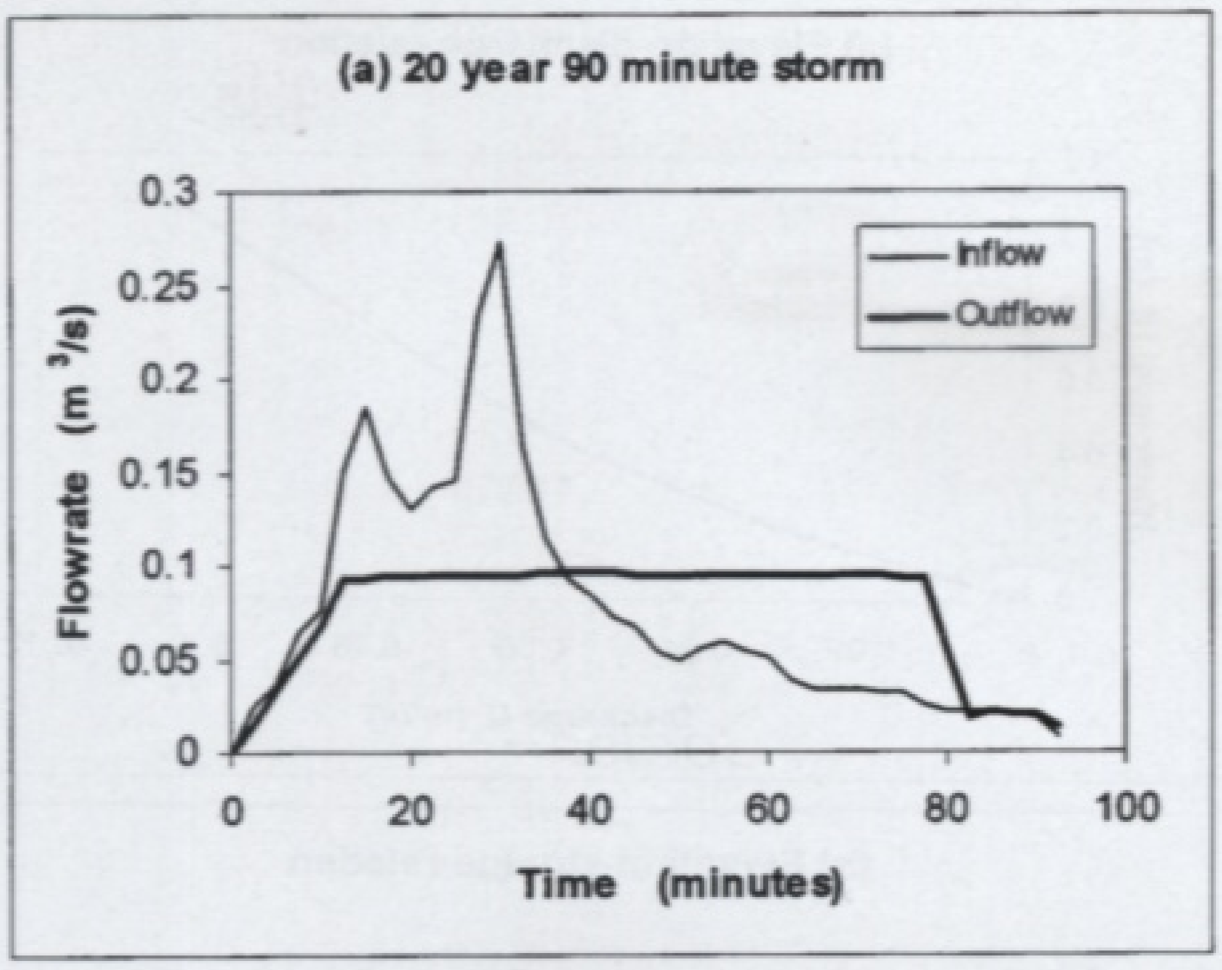


Figure 2: Elevation-storage-discharge loops for above ground HED

Other storms and other size sites could be used, with similar results. It is worth noting that the design storms are double peaked, which is common for durations greater than 30 minutes. The PSD was set at 0.10 m^3/s. Figure 3 shows outflow hydrographs calculated for this above ground HED storage. The larger volume in the 20 year storm takes longer to drain from the main chamber, resulting in the outflow remaining near to the PSD for some time after the inflow peak. When the main chamber does empty (at time 77.5 minutes) the water level in the first chamber falls rapidly. The double peaked design storm produces a double peaked inflow hydrograph which, for the 2 year storm, dips below the PSD. At this time the water level Hf and outflow discharge fall slightly, before rising again with the second peak.



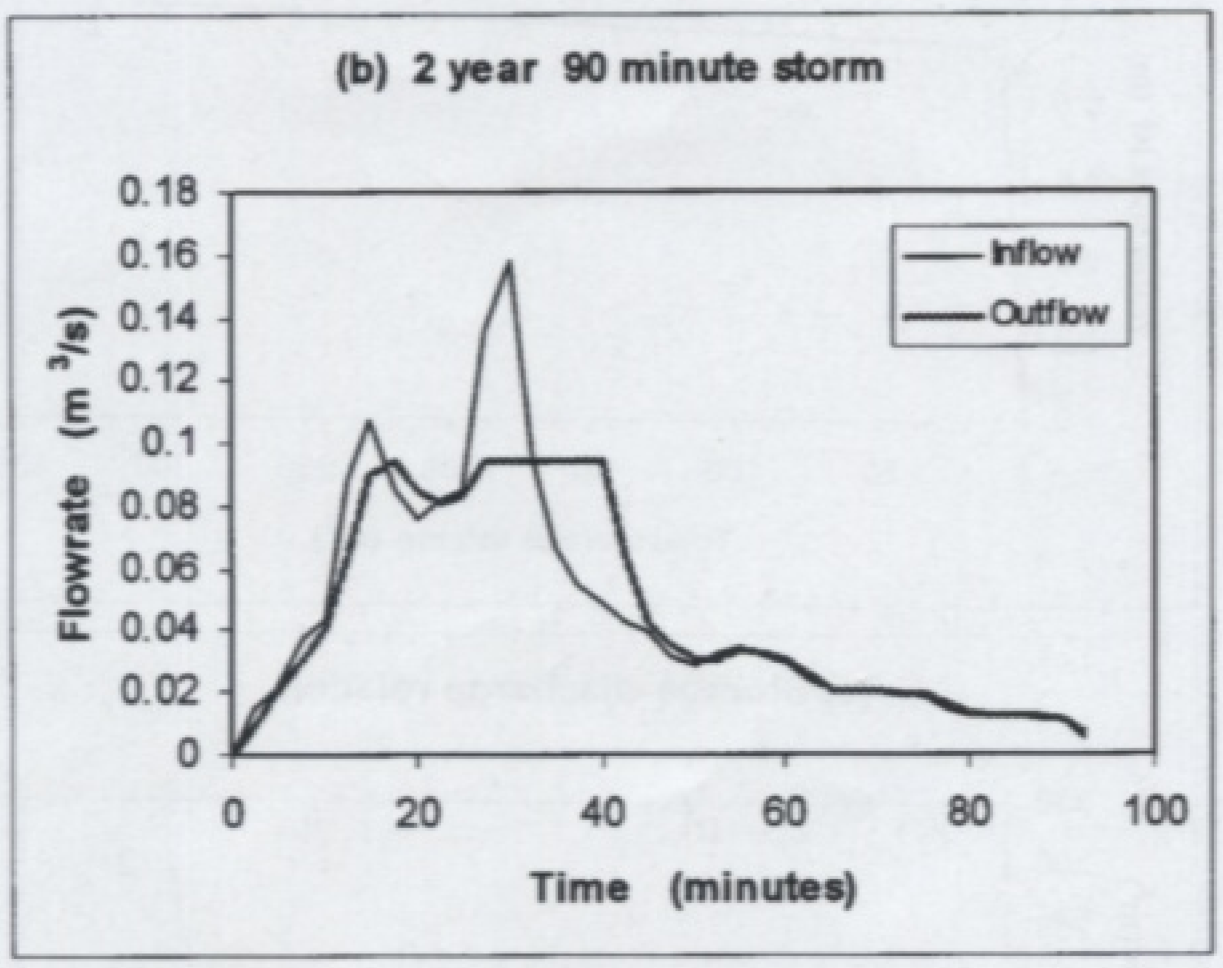


Figure 3: Hydrographs for above ground HED storage

3 BELOW GROUND HED STORAGES

3.1 Operating details

Stage 1 – first chamber filling (figure 4a)

At the start of the storm the water level $H_f(t)$ in the first chamber rises rapidly, as does the discharge from the storage Q(t). Since $H_f(t) > H_m(t)$, the one-way valve between the first and main chambers is closed, preventing water entering the main chamber. The mass-balance is given by equation 1. During this time $H_f(t)$ can rise or fall rapidly, depending on the time variation of the inflow I(t). Because $S_f(t)$ is small, Q(t) is close to I(t), and the outflow hydrograph closely resembles the inflow hydrograph.

In this, and all other cases, the outflow from the storage Q(t) is a function of the outflow orifice size and the water level in the first chamber $H_f(t)$. Additionally, in all cases the one-way valve prevents flow between the chambers whenever Hf (t) >= Hm (t). When emptying, this valve has a greater discharge capacity than the outflow orifice, therefore the water level in the first chamber cannot fall below the water level in the main chamber under any conditions:

$$H_f(t) \geq H_m(t) \tag{3}$$

Stage 2 – main chamber filling (figure 4b)

When Hf (t) > Hw, flow passes over the internal weir and starts to fill the main chamber. During this time, the outflow Q(t) remains close to the PSD. The mass-balance is:

$$I(t) - Q(t) - Qw(t) = \frac{dS_f(t)}{dt} \tag{4}$$

Flow over the internal weir is a function of the water level and weir dimensions. The volume of water entering the main chamber is given by:

$$S_m(t) = \int Qw(t).dt \tag{5}$$

and the water level in the main chamber Hm (t) is a function of its volume Sm (t). Note that the main chamber may or may not completely fill, depending on the size of the inflow hydrograph.

During this time, if the inflow I(t) decreases, the water level in the first chamber will rapidly fall, with a corresponding rapid decrease in Q(t). Similarly, an increase in inflow I(t) will result in a rapid increase in Hf (t) and Q(t). If Hf (t) falls to the level of Hm (t), then the flood routing is controlled by stage 4 (described later).

Stage 3 – first chamber emptying down to the level in the main chamber (figure 4c)

As the inflow hydrograph starts to decrease, the water level in the first chamber falls until it reaches the water level in the main chamber. The mass-balance is given by equation 1.

Stage 4 – first and main chambers emptying (figure 4d)

The inflow hydrograph is falling and the first and main chambers are emptying. The mass-balance is given by equation 2. The one-way valve prevents the water level in the first chamber falling below the water level in the main chamber, so that Hf (t) = Hm (t) and both chambers empty together. During this time, an increase in the inflow hydrograph can cause the water level in the first chamber to increase rapidly, with a rapid increase in discharge Q(t), according to eq (1).

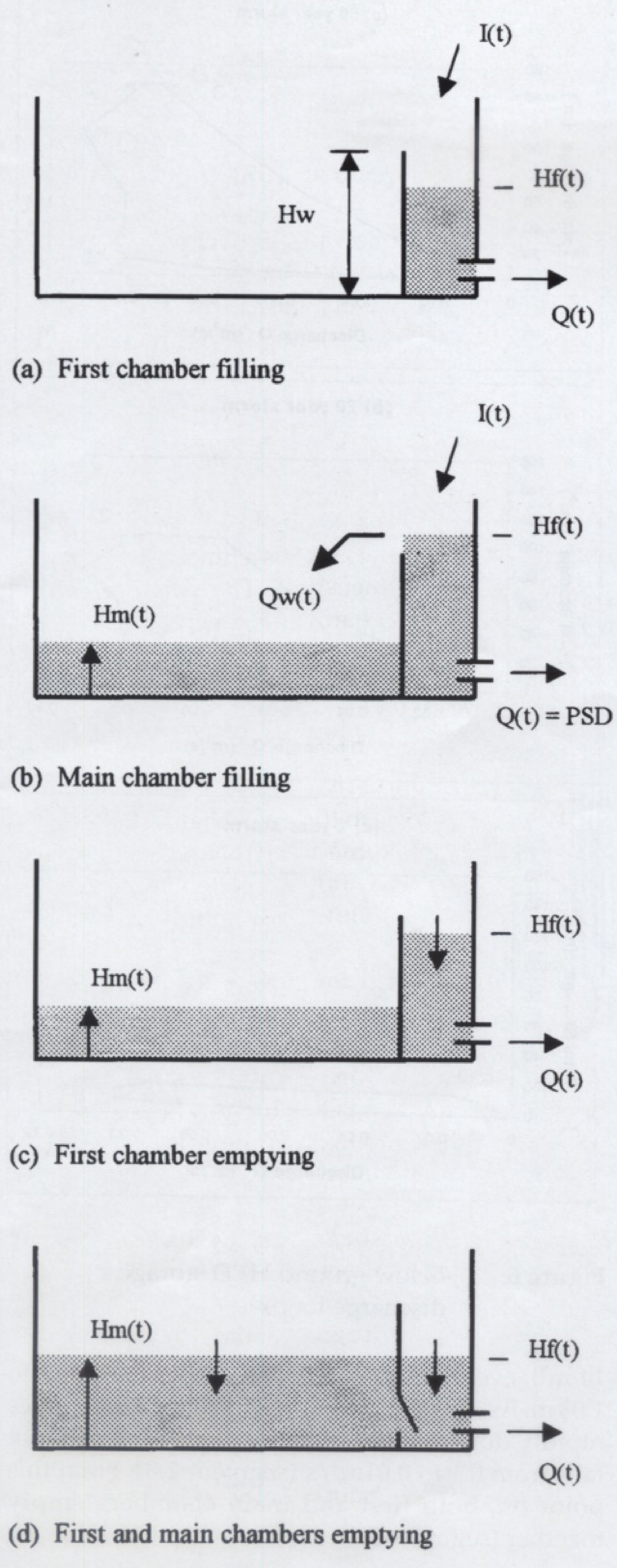


Figure 4: Operation of below ground HED storages

Accurate modelling of the HED storage requires that each of the controlling conditions be tested at each time step, followed by storage routing according to the appropriate mass-balance equation, and subject to the appropriate constraints on water levels. Because the operation of the chambers depends on their water levels, it is necessary to track Hf (t) and Hm (t) at all times. A computer program was written to solve this set of equations, and to produce a complete time-history of the inflow and outflow hydrographs, water levels in the two chambers, volumes stored in the chambers, and flows between the two chambers. This was then used to evaluate several approximate solutions.

3.2. Complete modelling of below ground storages

Figure 5 shows the performance of below ground HED storages for a range of storms, using the procedures of section 3.1. Both the first and main chambers were rectangular prisms, holding 8 and 152 m^3 respectively when full to 1 m depth. The internal weir height was Hw=0.95 m and the weir length was 2.0 m. Two 150 mm orifices were used to control outflows.

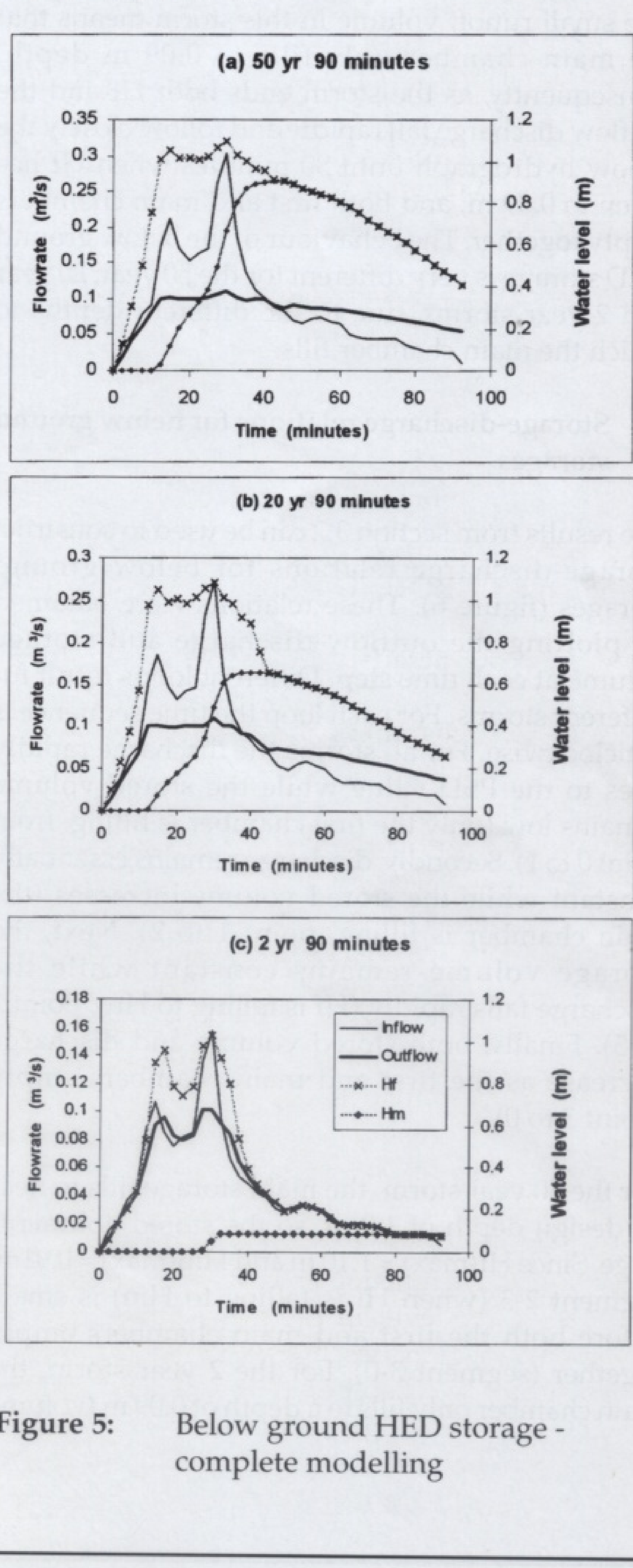


Figure 5: Below ground HED storage - complete modelling

For a 50 year storm (figure 5a) the water level in the first chamber Hf rapidly rises to the design level of 1 m. The outflow discharge, which is a function of Hf, also rises rapidly to the PSD of 0.1 m^3/s. After 10 minutes Hf exceeds the internal weir level (0.95 m), and the main chamber starts to fill. The large volume of runoff in this storm fills the main chamber to 0.91 m (at time 42.5 minutes). During all of this time Hf > Hm and Hf rises and falls as the inflow hydrograph rises and falls. At 42.5 minutes Hf has fallen to the level in the main chamber. From then on both Hf and Hm decrease together as the main and first chambers empty.

For the 2 year storm (figure 5c), Hf rises rapidly to 0.95 m then drops as the inflow dips, before rising to 1.03 m. Since Hf only exceeds the internal weir level on this second peak, it is only then that the main chamber starts to fill (at time 27.5 minutes). The small runoff volume in this storm means that the main chamber only fills to 0.09 m depth. Consequently, as the storm ends both Hf and the outflow discharge fall rapidly and follow closely the inflow hydrograph until 80 minutes when Hf has fallen to 0.09 m, and both first and main chambers empty together. The behaviour of the below ground HED storage is very different for the 50 year, 20 year and 2 year storms, due to the different depths to which the main chamber fills.

3.3 Storage-discharge relations for below ground storages

The results from section 3.2 can be used to construct storage-discharge relations for below ground storages (figure 6). These relations were obtained by plotting the outflow discharge and storage volume at each time step. Different loops result for different storms. For each loop the time sequence is anticlockwise. For all storms the discharge rapidly rises to the PSD value while the stored volume remains low (only the first chamber is filling, from point 0 to 1). Secondly, discharge remains essentially constant while the stored volume increases (the main chamber is filling, point 1 to 2). Next, the storage volume remains constant while the discharge falls rapidly (Hf is falling to Hm, point 2 to 3). Finally, both stored volume and discharge decrease as the first and main chambers empty (point 3 to 0).

For the 50 year storm, the main storage fills to near its design depth of 1.0 m, so the stored volume is large. Since Hf(max) = 1.10 m and Hm(max) = 0.91m, segment 2-3 (when Hf is falling to Hm) is small, before both the first and main chambers empty together (segment 3-0). For the 2 year storm, the main chamber only fills to a depth of 0.09 m (volume

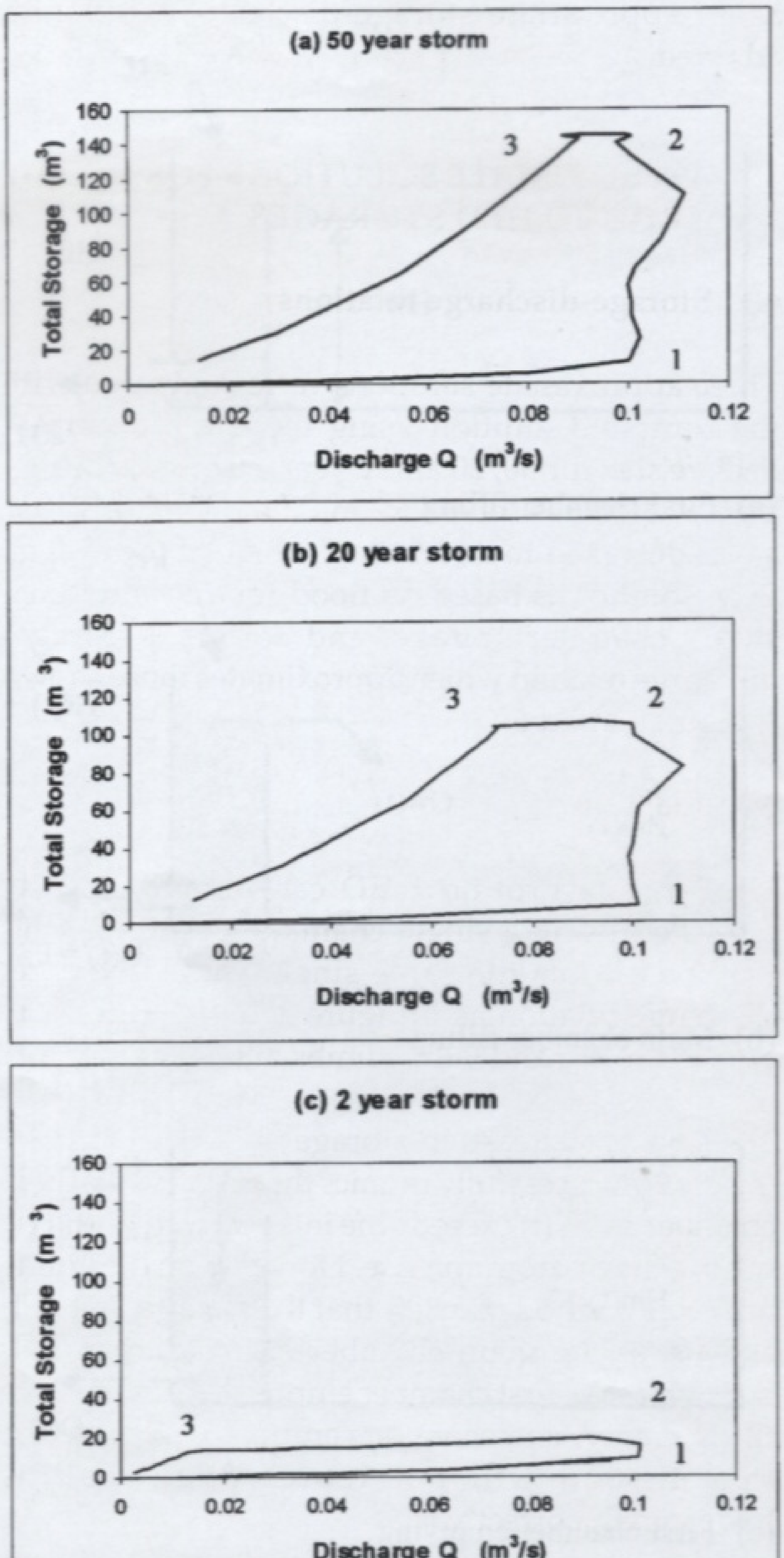


Figure 6: Below ground HED storage - discharge loops

14 m^3), even though the first chamber has filled to 1.03 m. As the storm ends, the first chamber empties rapidly down to 0.09 m and the discharge rapidly falls from 0.1 to 0.01m^3/s (segment 2-3). From this point on, both first and main chambers empty together (segment 3-0).

From these results it is clear that complete modelling of below ground HED storages needs to keep track of the water level Hf in the first chamber, since it controls the outlet discharge, and also of the water level in the main chamber, since it controls Hf when the storage is emptying. However, it may be possible to obtain a simpler solution using standard flood routing through a single chamber storage, as long as an appropriate storage-discharge relation is adopted.

4 APPROXIMATE SOLUTIONS FOR BELOW GROUND HED STORAGES

4.1 Storage-discharge relations

Three approximate solutions were compared with the complete solution using the runoff from a 1 hectare site, for 50, 20 and 2 year, 90 minute design storms. Hw was set at 0.95 m. The PSD of 0.10 m^3/s was designed to be released when Hf (t) = 1.0 m. Each solution is based on flood routing through a single chamber storage, and adopts a storage-discharge relation which approximates those shown in figures 2 and 6.

Method 1

Since above ground HED can be successfully modelled using a single chamber (section 2), this method adopts the same single valued storage-discharge relation as in figure 2, and applies it to both the rising and falling limbs of the hydrograph.

The abrupt increase in storage volume at point 1 (figure 2c) successfully mimics the filling of the main chamber as Hf (t) exceeds the internal weir level Hw during the hydrograph rise. However on the falling limb this method predicts that the storage first falls slowly (as the main chamber empties) then falls rapidly (as the first chamber empties). In reality, both the first and main chambers empty together and so this method will not represent the falling hydrograph correctly.

Method 2

The reason that method 1 first falls too slowly, then falls too rapidly on the falling limb, is that it adopts the same storage-discharge relation for both rising and falling limbs. In reality, once the main chamber is full and inflow starts decreasing, both the first and main chambers empty together. If the main chamber fills to the SSR value, then the storage-discharge relation falls from this maximum point (point 2 on figure 7a) directly to the origin. Method 2 therefore adopts figure 7a. Flood routing proceeds around the loop, from 0 to 1 to 2 and back to point 0. As will be seen in section 4.2, this method is satisfactory as long as the main chamber is full or near to full (see figure 6a), but is incorrect for smaller storms.

Method 3

For smaller storms, when the main chamber does not fill, it is necessary to use a storage-discharge loop specific to that particular storm. Figure 7b shows a set of loops, depending on the maximum volume stored in the main chamber. In method 3, the maximum volume stored as the water level in the main chamber rises to a peak is noted, and used to select one of the loops from figure 7b. These loops approximate those shown in figure 6, which were derived from the complete routing procedures.

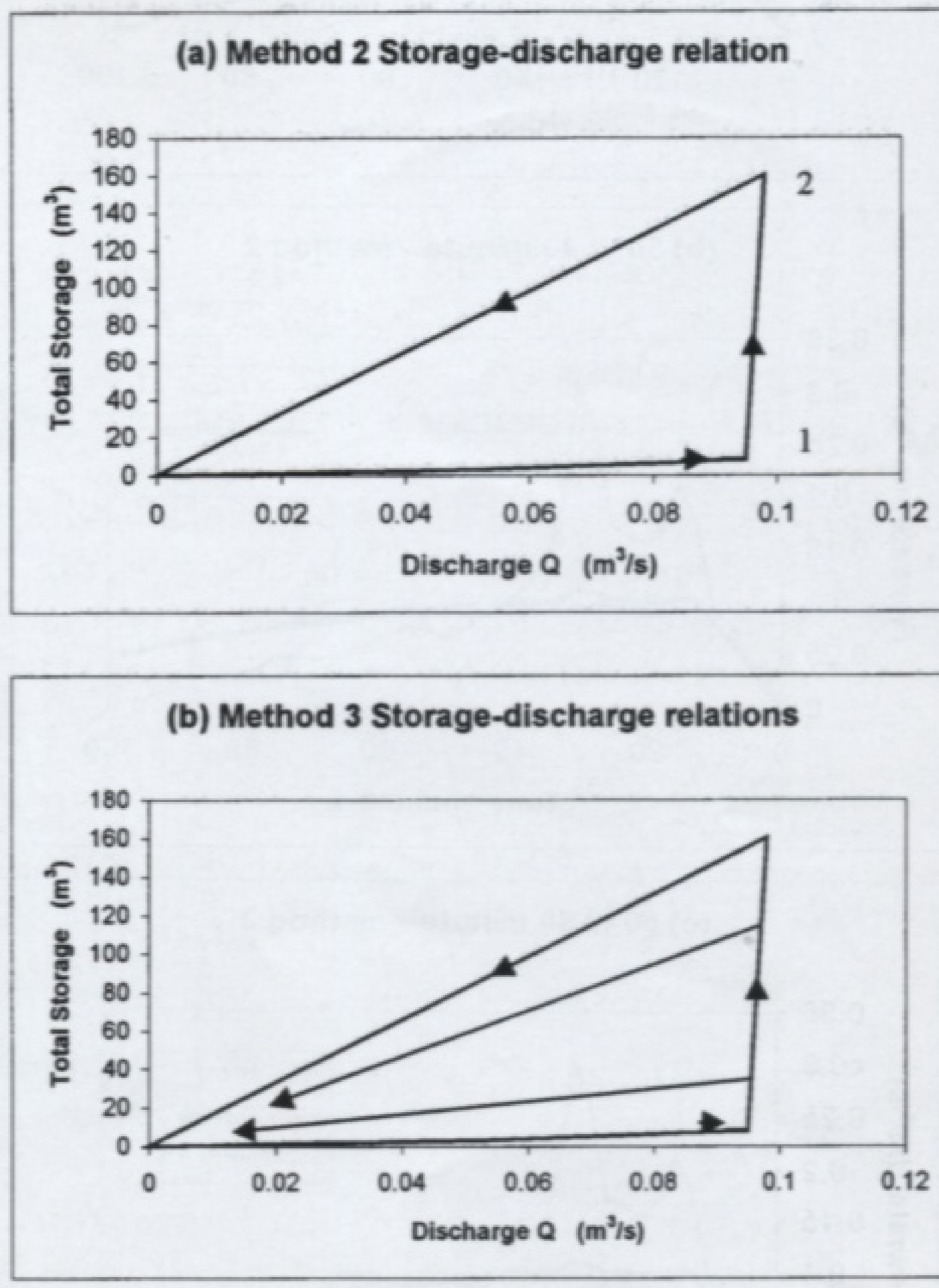


Figure 7: Below ground HED storage - approximate methods

4.2. Results using approximate solutions

Figure 8 shows hydrographs calculated for the 50 year storm, using approximate methods 1, 2 and 3. Results for the complete routing procedures described in section 3 are plotted as the *Outflow* line. Results for the approximate methods are shown as solid lines with points. The large runoff volume in this storm means that the discharge for method 1 remains at 0.1 m^3/s for a long time, and the hydrograph only begins to fall at time 90 minutes, overestimating the falling limb.

Because the main storage is nearly full at 0.91m and 138 m^3, the loops used in method 2 (figure 7a) and method 3 (loop 0-1-4-0 in figure 7b respectively) are similar to the actual loop (figure 6a) and so both methods 2 and 3 give good approximations to the correct solution.

Figure 9 shows hydrographs for the 2 year storm. In this case the small runoff volume only fills the

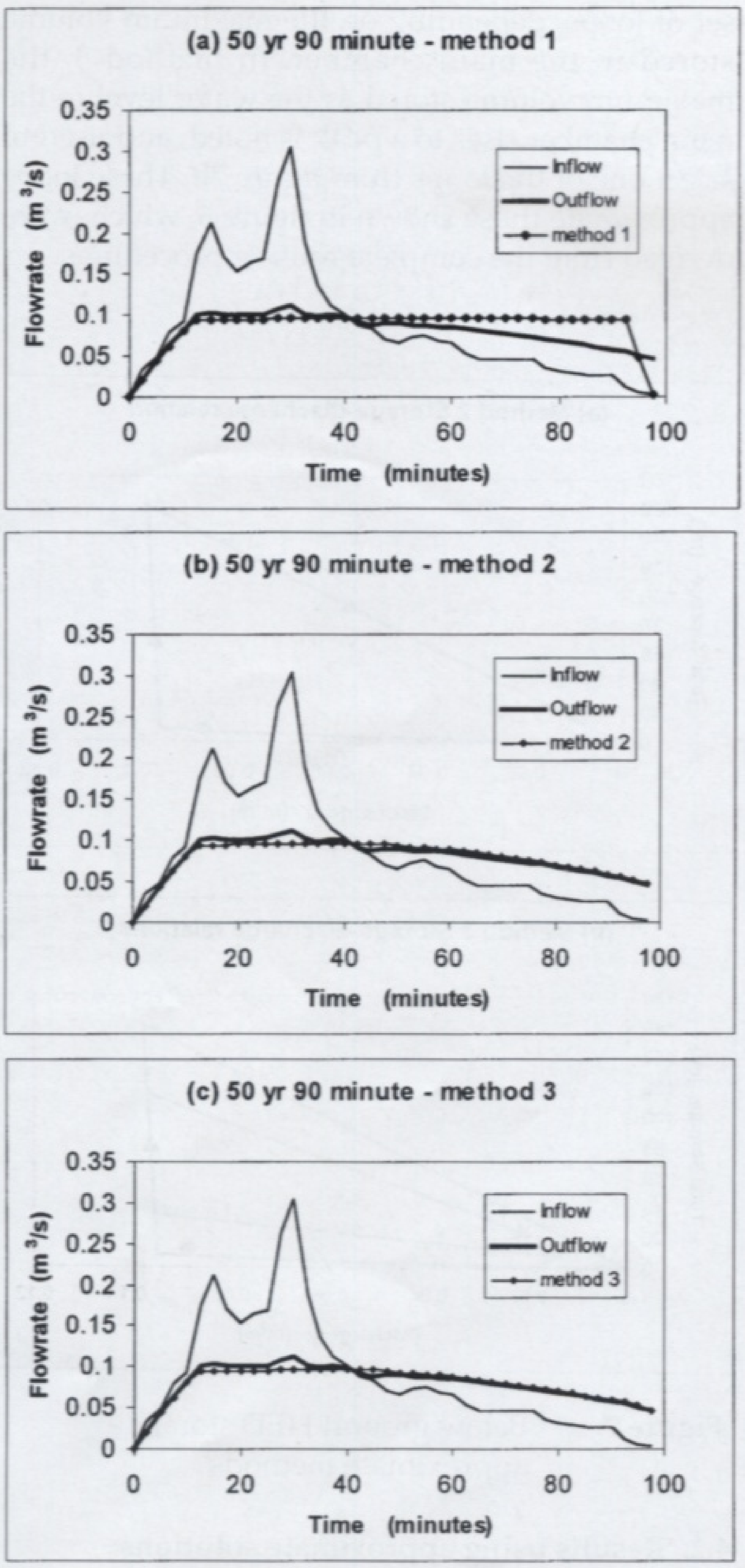


Figure 8: Approximate methods for below ground HED - 50 year storm

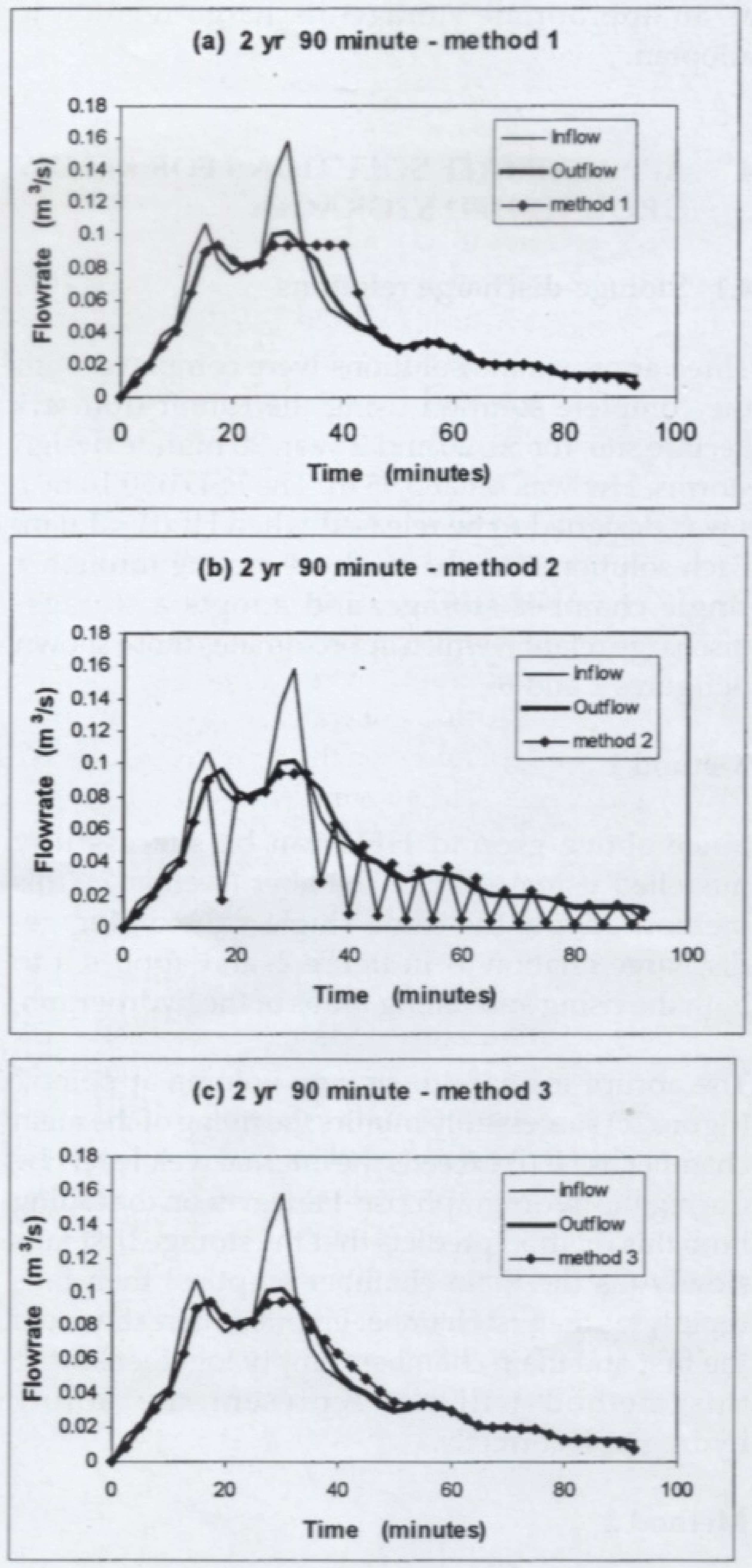


Figure 9: Approximate methods for below ground HED - 2 year storm

main storage to 0.09 m, with a stored volume of 14 m^3. Method 1 again overestimates the falling limb. Because the stored volume is only 14 m^3, method 3 adopts the storage-discharge loop 0-1-2-0 in figure 7b. Since this is similar to the actual storage-discharge loop for this case (figure 6c) this method produces quite reasonable results.

Method 2 however produces very poor results for the 2 year storm (figure 9b). The reason is that it adopts the storage-discharge loop of figure 7a, which is very different from the actual one (figure 6c).

As flood routing proceeds in this case, there is a sudden drop at time 17.5 minutes, and again at 35 minutes, as the inflow drops below the outflow discharge. This causes a sudden jump from point (0.095 m^3/s, 14 m^3) on segment 1-2 of figure 7a to point (0.01,14) on segment 2-0, and back again, resulting in oscillations in the calculated hydrograph. Because of the approximation in the adopted storage-discharge loop, the calculated outflow drops below the inflow on the recession limb from time 35 minutes onwards. As a result, mass balance is not conserved and the outflow volume is underestimated by 25%.

5 CONCLUSIONS

Two types of high early discharge (HED) onsite detention storages have been analysed, above ground and below ground. In above ground storages the water surface is continuous between the first and main chambers, and they can be accurately analysed using standard flood routing procedures. A single valued storage-discharge relation for both the rising and falling limbs of the hydrograph (figure 2) has been shown to give correct results (figure 3).

Below ground storages are considerably more complex. A complete flood routing procedure has been developed for this case. This procedure requires that water levels be calculated in the first and main chambers at every time step. This is necessary because flows of water between these two chambers are dependent on the relative water levels in the chambers. The procedure gives a complete time history of flows into and out of the storage, and flows between the two chambers of the storage. It also gives a time history of water levels and volumes stored in the two chambers (figure 5) .

Three approximate methods for flood routing in below ground storages were evaluated by comparing their results with the complete procedure, using a range of storms. The methods all use standard flood routing in a single chamber, but different storage-discharge relations. All three methods give good estimates of the rising limb, and of the maximum outflow or PSD. They also give good estimates of the required storage volume SSR, since this is the volume of inflow exceeding the PSD. The methods do, however, give different results for the falling limb of the outflow hydrograph.

Method 3 was found to be the most accurate of the approximate methods, because it selects a particular storage-discharge loop, depending on the maximum volume stored (figure 7b). Hydrographs estimated using this method were generally quite close the correct result, with only slight overestimation in some cases (figures 8c, 9c). A drawback of this method is that it requires additional steps to determine the storage-discharge loop which is unique to each storm. This being so, it may be more effective to move to the complete solution given in section 3.1.

Method 2 gave poor results for smaller storms, when the storage did not completely fill, resulting in severe oscillations and underestimation of the falling hydrograph limb (figure 9b). While it is possible to avoid this by specifying an additional constraint on the falling limb, this complicates the routing procedure, and again it may be more effective to move to the complete solution.

Method 1 is the simplest approximation, using a single valued storage-discharge relation for both the rising and falling limbs of the hydrograph, rather than a loop. Because of this, the outflow remains near to the PSD for some time before falling rapidly, and so overestimates the falling limb. Despite this, it does gives reasonable estimates of the outflow hydrograph for both large and small storms (figures 8a, 9a), it is robust, and it ensures that mass balance is always obeyed. If standard flood routing in a single chamber is to be used, this may be the most effective of the three approximate methods.

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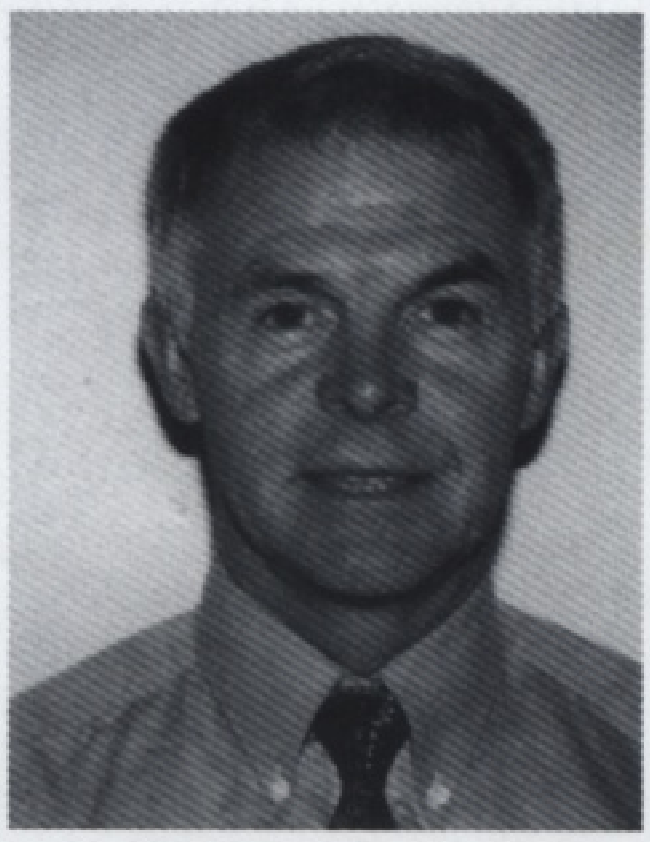

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Michael Boyd is a developer of the flood hydrograph model WBNM2002 for natural and urban catchments, including onsite detention storage. Details of this are available at www.uow.edu.au/eng/research/wbnm.html.