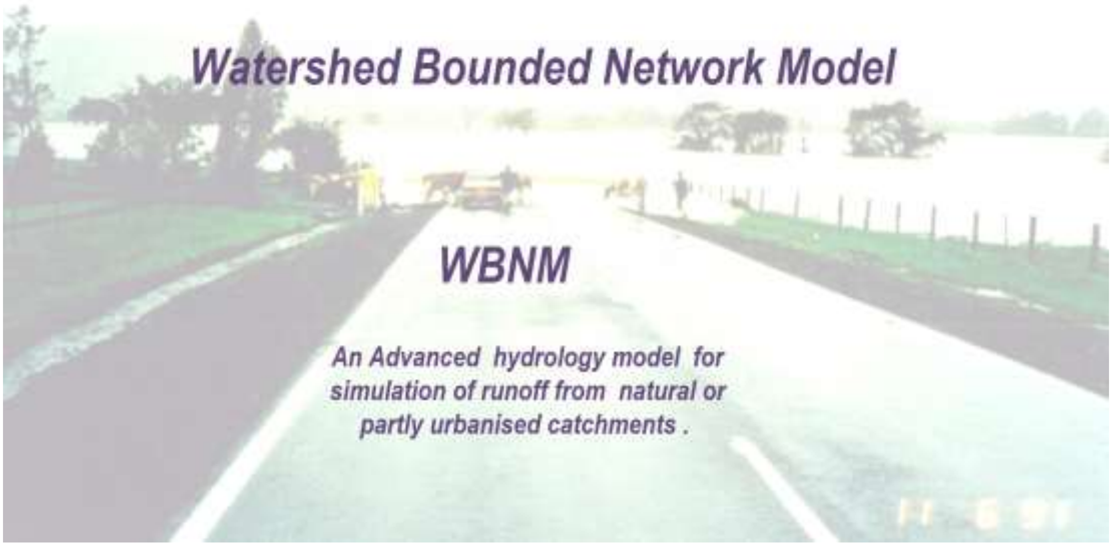


DETAILS of the THEORY used in WBNM

For all other details See:
WBNM_History.pdf
WBNM_References.pdf
WBNM_Runfile.pdf
WBNM_Validation.pdf
WBNM_Tutorial.pdf
WBNM_UserGuide.pdf

CONTENTS

1. SETTING UP WBNM to MODEL CATCHMENTS

A Basic WBNM Model for Natural Catchments

WBNM was originally developed for natural catchments, and is easiest to apply to this case. The catchment is divided into subareas by first identifying the main stream, then the major tributaries. The boundaries of the subarea draining to each tributary are then drawn, following the ridge or watershed line as defined by the surface contours. Each subarea will drain to the outlet of the tributary. These subareas (1, 2, 4 in figure 1) route excess rainfall using equations 1, 2 and 3 to produce a flood hydrograph at its outlet.

Next, draw the boundaries of the remaining subareas, again using the surface contours. Note that these subareas (3, 5 in figure 1) route excess rainfall to produce a flood hydrograph at the outlet, and in addition route runoff from upstream subareas through the stream channel using equations 1, 2 and 4. Subareas 3 and 5 therefore must have a stream channel with appropriate properties to convey the upstream runoff through them.

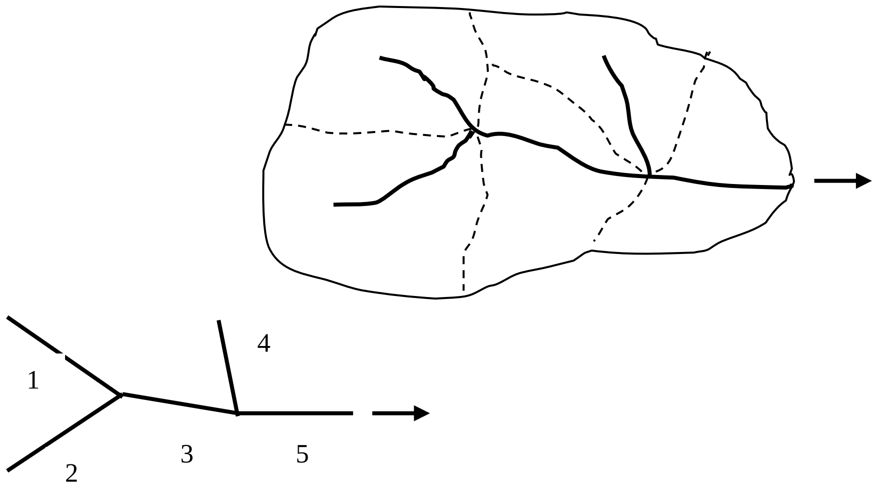


Figure 1. Dividing a Catchment into Subareas

Modelling More Complex Catchments with WBNM

The basic WBNM model is easy to set up, but somewhat limited in what it can do. WBNM has many options which give it considerable flexibility and allow complex flood studies to be carried out. We have aimed to introduce these options whilst retaining WBNM’s essential simplicity by associating many of the options with each subarea, and allowing them to be switched on or off as required.

What each Subarea Contains

Each subarea can contain the following components (figure 2) :

- a stream channel from top to bottom
- pervious surfaces
- impervious surfaces

- onsite detention storage for local runoff from the subarea (Local Structure)
- storage reservoir/ flood detention basin on the main stream channel (Outlet Structure)
- subarea outflows directed to top of nominated downstream subareas (as nominated in the topology block)
- outlet structure outflows directed to top or bottom of nominated downstream subareas (as nominated in the Outlet Structures block)

The top of the stream channel takes all hydrographs directed to this subarea from upstream subareas, and all external hydrographs imported to the top of this subarea. These hydrographs are summed and routed through the stream channel using equations 1, 2 and 4 to give a hydrograph at the bottom of the stream channel.

Pervious surfaces take the rainfall hyetograph for this subarea, subtract rainfall losses, and route the excess rainfall using equations 1, 2 and 3 to give a hydrograph from the pervious surfaces.

Impervious surfaces take the rainfall hyetograph for this subarea, subtract an initial loss for the impervious surface, and route the excess rainfall to give a hydrograph from the impervious surfaces. A reduced lag parameter is automatically calculated for impervious runoff.

The hydrographs from pervious and impervious surfaces are added to the hydrograph at the bottom of the stream channel, plus any hydrographs directed to the bottom of the subarea from the outlet structure of upstream subareas, plus any external hydrographs directed to the bottom of the subarea, to give the combined hydrograph at the bottom of the subarea. This flow can then be routed through an outlet structure at the bottom of the subarea.

Portions of the pervious hydrograph and impervious hydrograph can be directed into a local structure and routed through the storage. This routed hydrograph is added to other hydrographs at the bottom of the subarea**. Note that the local structure takes local runoff from the pervious and impervious surfaces in the subarea.**

An outlet structure (storage reservoir or flood detention basin) can be placed at the bottom of the subarea. If this storage is present, the summed hydrograph from the stream channel, plus any hydrographs directed to this point from outlet structures on upstream subareas, plus any imported hydrographs directed to the bottom of this subarea, plus hydrographs from the subarea pervious and impervious surfaces, and from the subarea Local Structure, is routed through the storage to give an outflow hydrograph**. Note that this outlet structure takes local runoff from the subarea plus flow from upstream subareas.**

Finally, the hydrograph at the outlet of the subarea is directed to a nominated downstream subarea. If you direct it to SINK, the hydrograph is directed out of the catchment.

Within the outlet structure routing, outflows from the various outlets eg culverts and weirs, can be directed to various nominated downstream subareas. Each of these outflows can be directed to the top or to the bottom of the nominated downstream subarea. This allows for surcharging flows, which may take different flow paths to the normal one, to be redirected within the catchment. Again, if you direct them to SINK, the hydrograph is directed out of the catchment.

You can direct flows from any subareas, or from any storage reservoirs to any number of locations outside the catchment. For example, you might have flows from several points going to the ocean as well as to the bottom of the catchment. You can direct to (say) OCEAN as well as to SINK. Normally all flows will exit the catchment at the bottom of the subarea, going to SINK. In this case, the calculated runoff depth will be equal to the excess rainfall depth. If however you direct flows from other points out of the catchment, the volume reaching the bottom subarea will be reduced, and the calculated runoff depth (runoff volume divided by the catchment area) will be less than the excess rainfall depth.

If hydrographs are directed to the top of a subarea, a stream channel must be provided to convey these flows to the bottom of the subarea. If there are no hydrographs at the top of the subarea (ie if the subarea is at the upper end of a stream tributary - subareas 1, 2, 4 in figure 1) then a stream channel need not be provided. In this case the hydrograph from the subarea comes from routing of its excess rainfall hyetograph.

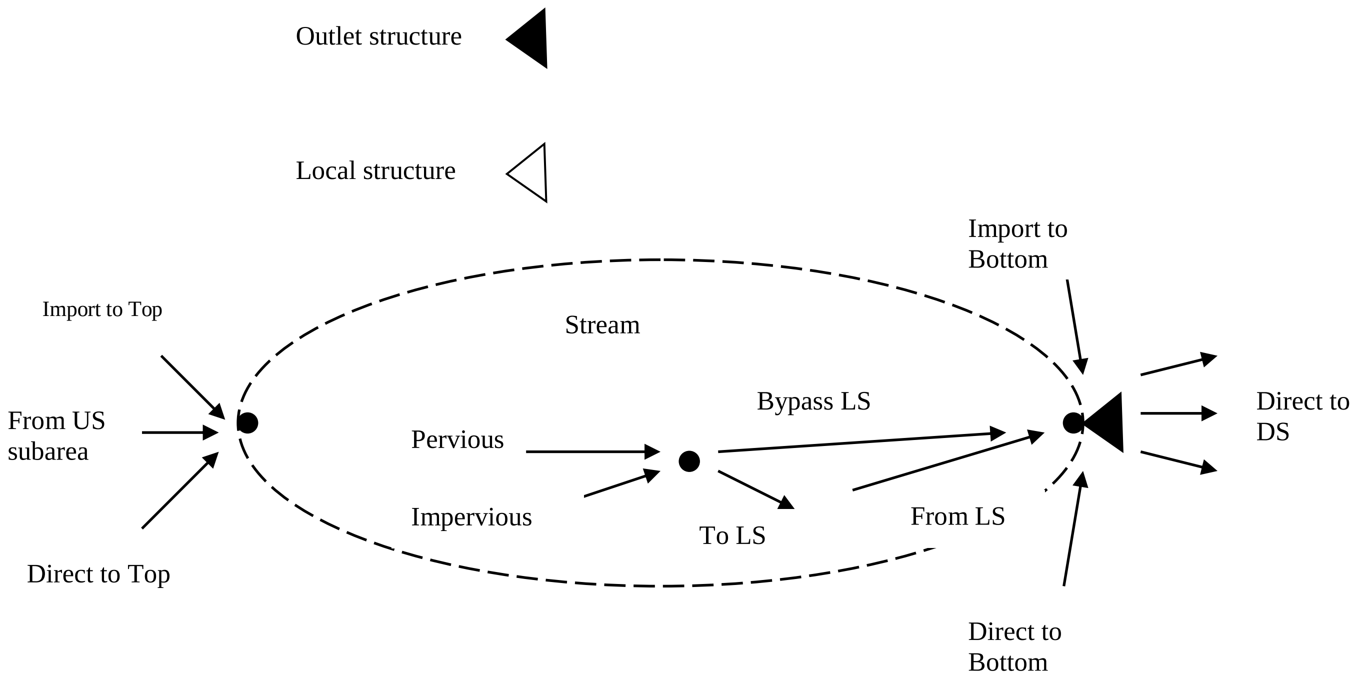
The foregoing paragraphs indicate that hydrographs coming from upstream subareas enter the subarea at the **top** of its stream channel. This applies to all hydrographs following the normal flowpaths on the catchment, as specified in the topology block of the runfile. However with outlet structures WBNM allows you to nominate whether the outflow goes to the top or bottom of the nominated downstream subarea.

This feature is particularly useful for surcharging flows when the stream channel capacity is exceeded. This will commonly occur when two adjacent tributaries pass under an embankment. If, say, a blocked culvert in one tributary causes surcharge, the surcharging flows will go along the embankment into the adjacent tributary. They will typically enter the **bottom** of the adjacent tributary. Hydrographs imported into the catchment can also be directed to the top or bottom of the subarea.

This structure of WBNM makes it extremely flexible, and allows you to model very complex flow situations. It is important to understand the processes occurring on each subarea to make full use of WBNM’s capabilities.

Further details of each of these operations are given later.

FLOWS in a SUBAREA



NOTES :

1) Every subarea can have some or all of these components
2) Subareas conveying flow from Top to Bottom must have a Stream Channel
3) Flows from any number of upstream Outlet Structures can go to the Top and/ or Bottom of this subarea
4) Flows from the Outlet Structure can be directed to up to 10 different downstream (DS) subareas, and can go to their Top or Bottom
5) The Local Structure takes runoff from the pervious and impervious surfaces of the subcatchment
6) The Outlet Structure takes runoff from the pervious and impervious surfaces of the subarea, plus runoff from Upstream subareas which has been conveyed from Top to Bottom in the Stream Channel, plus hydrographs directed into the subarea, plus hydrographs imported in to the subarea.

Figure 2. Components of a Subarea

Order of calculations in WBNM

WBNM calculates hydrographs subarea by subarea, starting at the uppermost one and moving downstream to the catchment outlet. If a subarea receives a hydrograph from an upstream subarea, this upstream subarea hydrograph must first have been calculated. Subareas are calculated in the order in which they occur in the topology block of the runfile.
Therefore the lowermost subarea in the catchment MUST be placed last in the topology block.

Within each subarea, calculations take place in the following sequence :

Flood routing from top to bottom of the stream channel

Rainfall hyetograph for the subarea calculated from surrounding rain gauges

Pervious surface local runoff hydrograph

Impervious surface local runoff hydrograph

Local structure storage routing of local runoff from the subarea

Summing of all hydrographs at the bottom of the subarea (stream channel hydrograph, pervious runoff, impervious runoff, outflow from local structure, any surcharging flows directed to the bottom of this subarea, external hydrograph directed to the bottom of the subarea).

Outlet structure storage routing of summed runoff at the bottom of the subarea

Directing flows to downstream subareas (if an outlet structure storage reservoir is present, you nominate whether flows go to the top or bottom of the subarea; if there is no storage reservoir, flows go to the top of the subarea).

Note that to direct the flows from a subarea to a nominated downstream subarea, you must name the downstream subarea in the TOPOLOGY block (to direct normal flows) and in the OUTLET STRUCTURES block (to direct outflows from the storage reservoir). WBNM searches for matches of these named subareas when directing flows. Therefore you **MUST** use **exactly** the same name in all blocks of the runfile. WBNM similarly searches for matches of the subarea names to identify those with local and outlet structures, for rainfall losses, stream channel details, and so on. Again you MUST use exactly the same names in all blocks of the runfile. For the lowermost subarea (where the catchment outlet is located) flows leave the catchment and are therefore sent to SINK. If you want the flows from any other subarea to leave the catchment (this will most likely be surcharging flows which are directed out of the catchment, you can send them to SINK or to another location such as OCEAN.

Note: you MUST direct flows from the lowermost subarea to SINK.

2. DETAILED INFORMATION on OPERATIONS IN WBNM

Units

Coordinates of maps, subareas and rain gauges	(usually metres, but any consistent set)
Subarea size	(hectares)
Impervious percentage	(%)
Time, Delay time, Muskingum K	(minutes)
Elevation in storages	(metres)
Discharge	(m^3/s)
Storage volume	(thousands m^3)
Storage factor	(decimal eg 0.9)
Culvert dimensions	(mm)
Weir length	(metres)
Weir coefficient	(SI units eg 1.70)
Discharge factor	(decimal eg 0.9)
Scour Factor	(m^3 water / m^3 soil eg 200)
Stream channel dimensions	(metres)
Stream channel bed slope	(% eg 0.5)
Stream channel side slope	(decimal ratio V:H eg 0.20)
Tailwater elevation	(metres)

Percentage of pervious, impervious runoff to onsite detention (%)

Recorded Rainfall hyetograph	(mm/hour or mm/time period)
Design Rainfall hyetograph	(mm/hour)
Rainfall period	(minutes)
Raingauge weighting factor	(decimal eg 0.9)
Area Reduction Factor	(decimal eg 0.95)
Terrain Roughness	(integer % eg 25)
Moisture Adjustment Factor	(decimal eg 0.63)
Initial loss	(mm)
Continuing loss rate	(mm/hour)
Horton infiltration rates	(mm/hour)
Horton k	(1/hour eg 2.0)
Runoff proportion	(decimal eg 0.75)

Note: all levels of water surfaces, culvert and weir levels, and tailwater levels are ELEVATIONS relative to your selected datum. Water DEPTHS are not used.

Basic Concept of WBNM

WBNM was originally developed to be a physically realistic representation of the catchment as it transforms storm rainfall into a flood hydrograph. It has built in lag relations, based on catchment geomorphology. The actual lag times in these relations were derived using recorded rainfall and flood hydrograph data. Because of this, it requires a minimum of parameters to be evaluated in its basic form, yet still gives good hydrograph reproduction.

A catchment is divided into smaller subareas, based on the stream network. Each subarea is bounded by its ridge line (or watershed) and forms a catchment within the larger catchment. This explains the origin of the model's name : **W**atershed **B**ounded **N**etwork **M**odel.

Each subarea is represented by a unit in the model which has the lag properties of the corresponding subarea, and which takes as input the rain falling on the subarea. WBNM calculates hydrographs at the outlets of all subareas, thus producing hydrographs at many points within the catchment. This allows detailed and complex flood studies to be carried out. Among the many features which can be modelled are: placing of storage reservoirs at various locations within the catchment, diversions of surcharging flows within the catchment, changes to stream channels, and runoff from pervious and impervious surfaces.

The equations in WBNBM are:

For each subarea, Conservation of Mass

$$I - Q = dS / dt \qquad (1)$$

where I = inflow rate at time t (m^3/s)
Q = outflow rate at time t (m^3/s)
S = volume of water stored on catchment surface at time t (m^3)
t = time (s)

The stored volume is related to the outflow discharge by

$$S = 60.\ K.\ Q \qquad (2)$$

where K = lag time between centroids of inflow and outflow hydrographs (minutes).

The lag time K will depend on the size of the subarea. If it remained constant for this subarea for all size floods, the model would be linear (similar to the Unit Hydrograph). However, based on recorded rainfall and flood hydrograph data (Askew, 1968, 1970), and also on hydraulic considerations, WBNM allows K to decrease as flood discharges increase, and is thus nonlinear.

Lag Relation for Overland Flow

Because the subareas are actually smaller catchments within the larger catchment, they are geomorphologically and hydrologically similar, and lag relations developed for catchments can be applied both to the larger catchment and to all of the smaller subareas within the larger catchment.

Lag times for runoff (strictly defined as the time difference between the centroids of the excess rainfall hyetograph and the surface runoff hydrograph) depend on the physical properties of the catchment. These include area A, shape (often defined as A/L^2) and slope for natural catchments. For urban catchments, the effects of impervious surfaces and modifications to stream channels should be included. Catchment flood response is often considered to be nonlinear, with lag times decreasing as the size of the flood increases.

For natural catchments, by far the dominant factor is the catchment area. Catchment shape and slope are strongly correlated with catchment area, indicating that they need not be considered in most cases. WBNM uses lag relations developed by Askew(1968, 1970):

$$\text{Overland Flow Lag Time} = \text{Lag Parameter}.\ A^{0.57}.\ Q^{-0.23} \qquad (3)$$

(A km^2 ; Q m^3/s ; Lag Time hours)

This equation contains a nonlinearity component (lag decreases as discharges increase), and an area component. Note that stream lengths L are commonly related to catchment area in relations close to $A^{0.57}$, so the equation indicates that lag times are approximately proportional to travel distances, as we would expect.

Lag time can be thought of as an average travel time for runoff from the catchment surface, and is related to flow velocity by L/V. This allows us to visualise the effects that changes to the catchment surface have on lag times, and hence on flood hydrographs. If, for example, a stream channel is modified so that flow velocities in it increase by 50% (ratio 1.50), travel times and lag times can be expected to decrease, being 0.67 of the lag time before modification. With these decreased lag times, the hydrograph occurs in a shorter time base, the time to rise and fall is less, and peak discharges are greater. The general relation is that a decrease in lag time results in an increases in flood peak discharges.

The lag relation in WBNM is physically realistic and allows lag times to be calculated automatically for all subareas depending on their size. Note that if a catchment is divided into a few large subareas, they will each have a large lag time, but in combination the few will not sum to an excessively large lag for the total catchment. Conversely, if the catchment is divided into many small subareas, they will each have a relatively small lag time, but in combination, summation of a large number of values means that the total lag time for the catchment as a whole will be maintained. Because of this, the number of subareas into which a catchment is divided is not critical.

The number of subareas used depends on the extent to which details of the catchment are to be modelled. Small catchments, particularly urban catchments, where a hydrograph is only required at the outlet, can be modelled as 1 subarea. Generally however, the catchment will be divided into several subareas. Guidelines for minimum and maximum numbers of subareas (Boyd, 1985) are :

Catchment area (km^2) :	0.1	1	10	100	1000	10000
Minimum	4	5	7	9	15	20
Maximum	20	26	35	45	60	80

Note however that values outside this range can be used without problems.

Lag Relation for Stream Channel Flow

Equation 3 allocates a lag time for each subarea, depending on its catchment size. This lag time applies to the transformation of excess rainfall into a surface runoff hydrograph, via overland flow and flow in small streams draining to the subarea outlet.

In some subareas, runoff from upstream subareas flows through the main stream channel. WBNM calculates separate hydrographs for the channel routing and for the overland flow routing on these subareas. This is a physically realistic approach, because flow velocities and lag times will be different for channel flow compared to overland flow. Also, this approach allows modification to catchment surfaces (eg imperviousness) separately from changing to stream channels (eg clearing, lining).

Since flow velocities are higher for flow in stream channels, lag times will be less than for overland flow. The lag time should be related to the length of the stream channel. However, the stream channel length is strongly correlated with the size of the subarea, allowing an equation similar to (3) to be used. Boyd et al. (1979, 1987) studied these relations between stream channel length and area, and also between the stream channel lag time and area, and used recorded rainfall and flood data, to show that a factor 0.6 should be applied to the lag time for hydrograph routing in the stream channel.

$$\text{Stream Channel Lag Time} = 0.6 \,.\, \text{Lag Parameter}.\, A^{0.57}.\, Q^{-0.23} \qquad (4)$$

Note that the Lag Parameter has the same value in equations 3 and 4, ie only one parameter is needed in WBNM. WBNM automatically adjusts for the smaller lag times in stream channels.

The relations between stream channel length and subarea size, and also the value 0.6 to adjust the stream lag have been confirmed by studies of Kemp(1995) and Jenkins(1997).

See also the section on Routing Hydrographs in Stream Channels.

Building equations 3 and 4 into WBNM has two major advantages for the user. Firstly, it is only necessary to measure the area of each subarea. Stream lengths and slopes are not needed, reducing data requirements. Secondly, because the nonlinearity is fixed at 0.77 (ie 1.0-0.23), and the factor 0.6 is applied to stream channel routing, it is only necessary to determine the value of one parameter to apply WBNM. This Lag Parameter is a scaling factor which increases or decreases the lag time allocated to all subareas.

Recommended Value of the Lag Parameter

Because equations 3 and 4 are general, and include the effect of catchment size and also the effect of flood size, a similar value of the Lag Parameter should apply to a wide range of catchment and flood sizes. This has been found to be the case. Figure 3 shows values of the Lag Parameter derived for 129 storms on 10 catchments in eastern NSW plotted against the size of the flood. Note that there is no trend for the Lag Parameter to either increase or decrease with increasing flood size, indicating that the built in nonlinearity is correct.

Figure 4 shows average values of the Lag Parameter for all events on each of the 10 catchments, plotted against catchment size. Again, the absence of a trend indicates that the term $A^{0.57}$ is quite satisfactory.

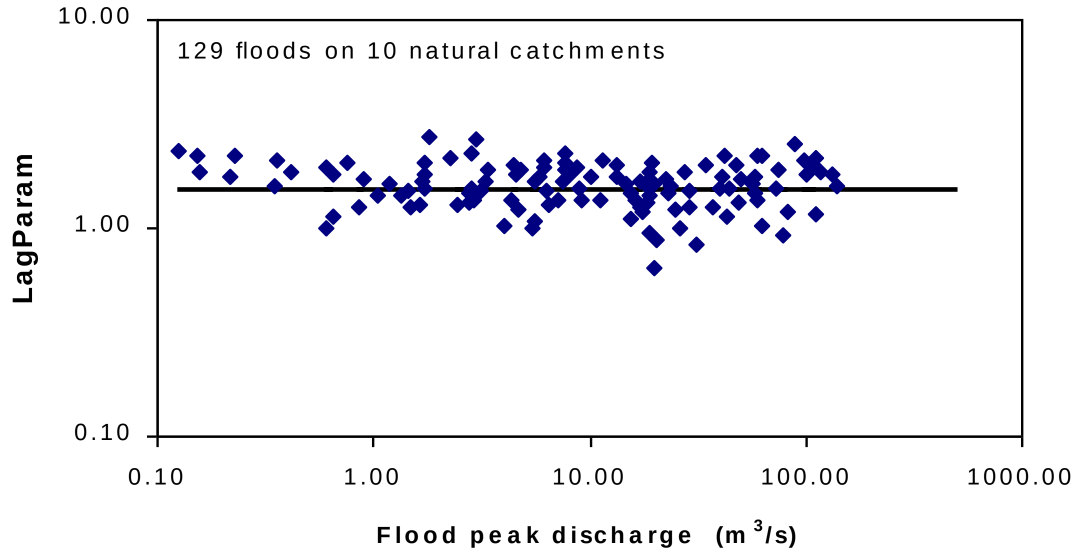


Figure 3. WBNM Lag Parameter versus Flood Peak Discharge

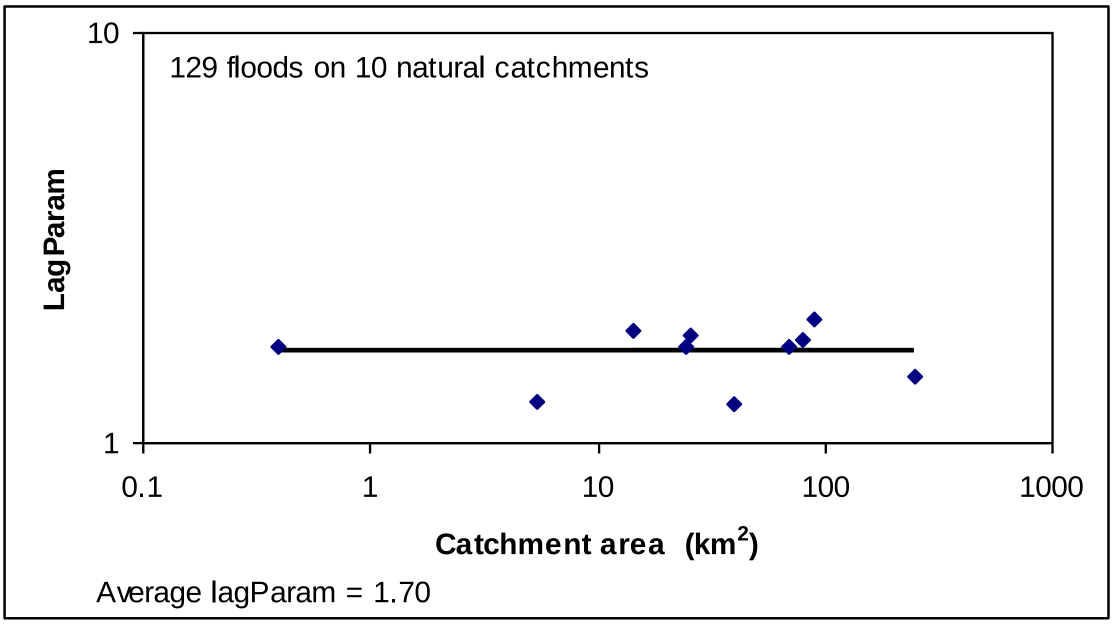


Figure 4. WBNM Lag Parameter versus Catchment Size

These results have been confirmed over a wide range of flood and catchment sizes (flood peaks from 0.3 to 1400 m^3/s and catchments from 0.04 to 9000 km^2). They have also been confirmed by the studies of Webb and O'Loughlin(1981) and Sobinoff et al(1983), and for design storms by Boyd and Cordery(1989).

Figure 5 shows results for a larger data set of 54 catchments in Queensland, NSW, Victoria and South Australia (Boyd and Bodhinayake, Australian Journal of Water Resources, 2006, 10(1):35-48). Average lag parameters for these 4 states were 1.47, 1.74, 1.74 and 1.64 respectively. The lag parameter values were independent of catchment area, stream slope and storm characteristics. An average parameter value of 1.60 was found to apply to all 4 states.

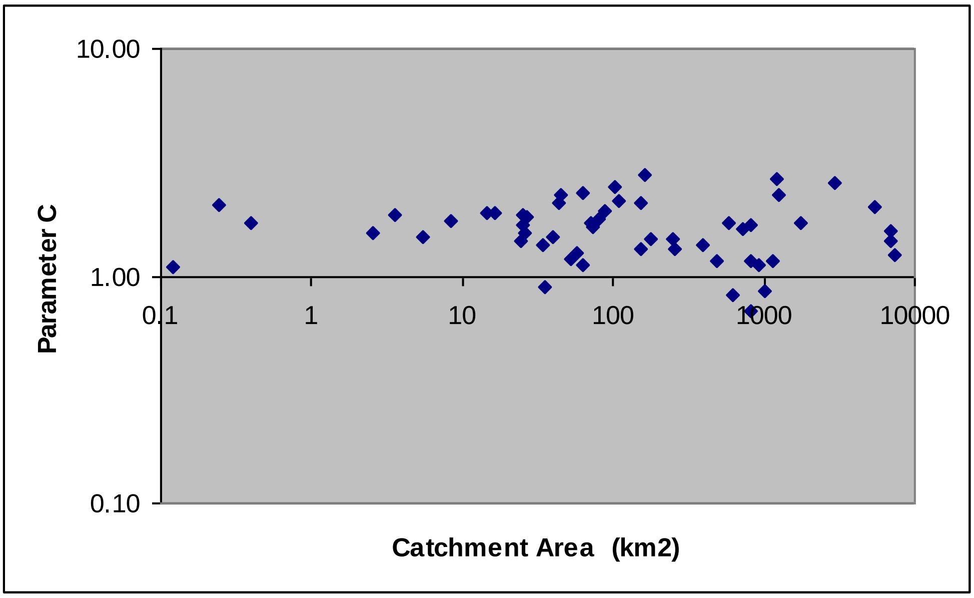


Figure 5. WBNM Lag Parameter versus Catchment Size – 584 storms on 54 catchments in Queensland, NSW, Victoria and South Australia.

The average value of Lag Parameter for the catchments of figures 3 and 4 is 1.68, while for those of figure 5 it is 1.60. The study of design flood estimation by Boyd and Cordery (1989) used 36 catchments in eastern and inland NSW (size range 0.04 - 1140 km^2). The study calculated design storms following standard procedures in Australian Rainfall and Runoff(1987), subtracted design losses as recommended in ARR1987, then used WBNM to calculate the corresponding design floods for a range of average recurrence intervals. The peak discharges of these design floods were then compared with peaks from a flood frequency analysis performed on the recorded data for each catchment. The Lag Parameter was then adjusted to make the WBNM peak discharge agree with the flood frequency peak discharge. From this study, the average Lag Parameter was found to be near to 1.80.

Considering all of these results, a value of the **Lag Parameter near to 1.6 is recommended**. It is, of course, desirable to check the results of any model against recorded flood data. This will preferably be a recorded hydrograph, but peak flood levels could also be used.

Because WBNM allows adjustments for impervious surfaces, and for modifications to stream channels, this base value of the Lag Parameter should apply to natural, part urban and fully urban catchments. **The same value of the Lag Parameter should be used for all subareas (global value), unless there is good evidence for varying it.**

Nonlinear and Linear Flood Response

Nonlinearity in WBNM is based on studies of floods in natural catchments by Askew(1968, 1970). Figures 6 and 7 show typical results. Note that the nonlinearity power for this catchment is –0.24. Askew found that this value did not vary for different catchments and adopted an average of –0.23.

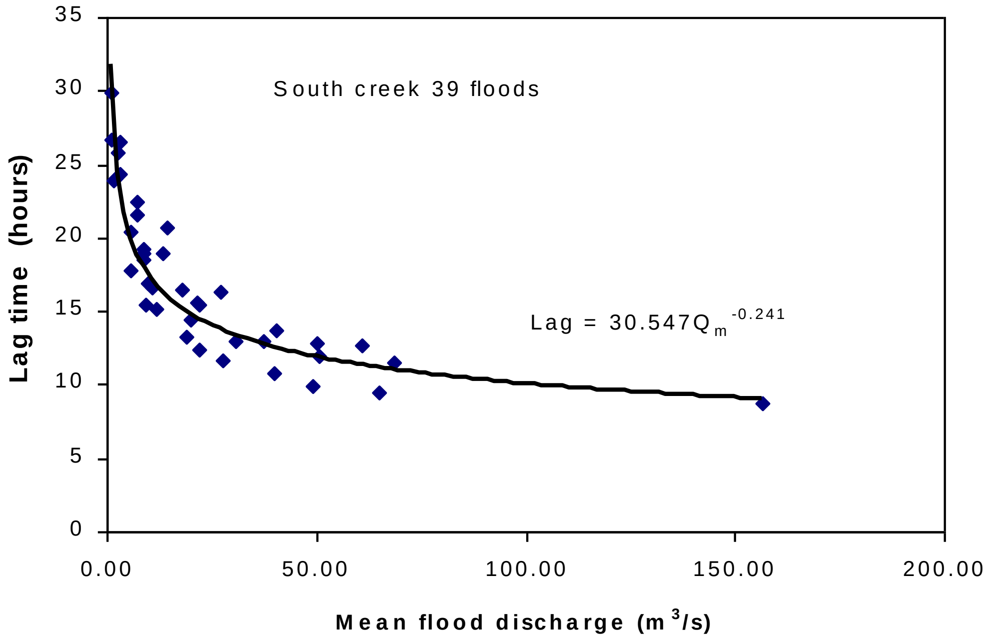


Figure 6. Lag Time versus Flood Discharge

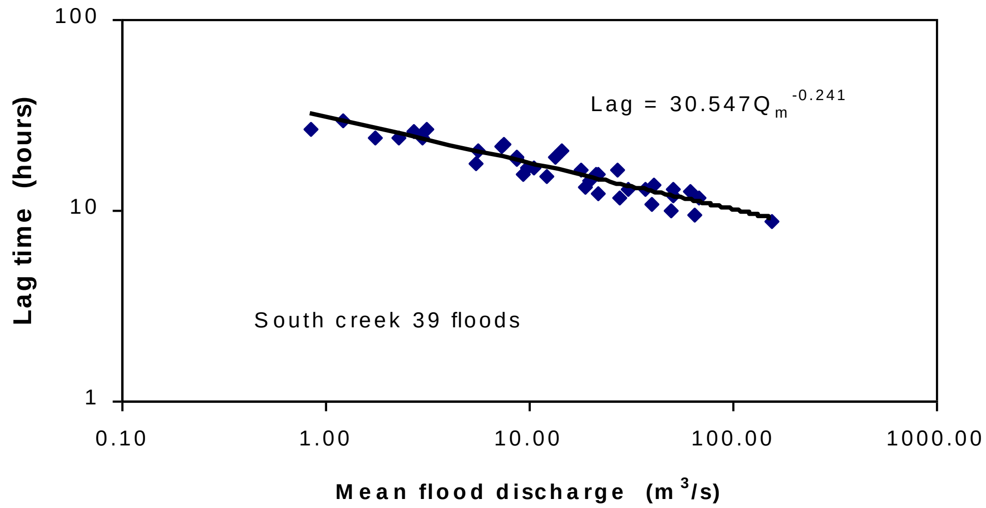


Figure 7. Lag Time versus flood discharge (log scales)

Equations 3 and 4 are of type $K = k.Q^{m-1}$. Nonlinearity is sometimes expressed by combining this with equation 2

$S = K.\ Q = k.Q^{m}$

WBNM uses m-1 as its measure of nonlinearity. The default nonlinearity exponent is therefore m-1 = -0.23 (ie m = 0.77). This default nonlinearity causes the lag time to decrease as the flood discharge and flow velocities increase.

WBNM allows this nonlinearity exponent to be varied. If m-1 is less than -0.23 (eg –0.3, the nonlinearity is greater, and lag times decrease even more as discharges increase. If m-1 = 0, the flood response is linear and lag times remain constant over the range of discharges.

We strongly recommend that you adopt the default value of m-1 = -0.23 unless there is strong evidence that it is different than this value. The nonlinearity exponent should not be varied to better calibrate a single flood event. Nonlinearity controls the change in model lag over a range of flood discharges, and if it is varied, it should be to reflect these changes over a range of flood events of different sizes.

WBNM also allows modification to the form of the nonlinearity. It is sometimes considered that whereas small to medium floods behave nonlinearly, larger to extreme floods may behave linearly (Bates and Pilgrim, 1983, 1986; Wong and Laurenson, 1983). WBNM models this by allowing you to specify a discharge at the catchment outlet above which the response changes from nonlinear to linear. As discharges are calculated during a flood, WBNM automatically decreases the lag time at each time step, according to equations 3 and 4. When the discharge exceeds the specified value, WBNM retains the last calculated lag time for all subsequent calculations (and so remains linear), until the discharge falls back below the specified value, when nonlinearity resumes. In WBNM, the specified nonlinear/ linear breakpoint discharge at the catchment outlet is automatically adjusted and applied to all points within the catchment. This is done by factoring this discharge by the square root of the ratio of area contributing to the point, over the total catchment area. For subareas 1, 2 and 4 in figure 1, this area is for the subarea itself. For stream channels (subares 3 and 5), it is the total area upstream of this point. The SUM_SUBAREAS table gives the area and running (ie cumulative) area at these points, plus these adjusted breakpoint discharges. Note that this table only shows the breakpoint discharges for runoff from the subarea itself (and are thus based on the subarea size alone), and not for the stream channel using the total area upstream of this point. The breakpoint discharges calculated for the stream channels are not shown.

If you specify a positive nonlinear/ linear breakpoint discharge, the nonlinear/ linear routing option is invoked. If you specify a negative value (-99.9), the standard nonlinear routing is invoked. WARNING, if you specify a zero nonlinear/ linear discharge, the flood routing is COMPLETELY linear.

We strongly recommend that you run WBNM as fully nonlinear (with m-1 = -0.23) unless you have good evidence that a change from nonlinear to linear response occurs.

Effect of Catchment Slope on the Lag Parameter

The lag relations (equations 3 and 4) include the effect of subarea size A and discharge Q, but do not include the catchment slope S. This is because the studies of lag times on natural catchments by Askew(1968, 1970) found that catchment slope was not a significant factor. The catchment area is certainly the dominant influence on lag time, with other factors such as slope or catchment shape being secondary, or not significant at all. It is worth noting that some other studies and other models include S in the lag relation, but always as a secondary variable, after area A.

There are several reasons for catchment slope not having a strong effect on catchment lag time. Firstly, the area A can vary over about 5 orders of magnitude when different catchments are considered, whereas S will have a much reduced range, typically 1 order. Thus the range of S used to develop any relation with lag time is lower. Secondly, S is not as accurately measured as A and depends on the map scale used, so there is less certainty in the adopted S value. Thirdly, many studies have shown that slope S and area A are themselves correlated, since smaller subareas will often be in the steeper part of the larger catchment containing them. This correlation is quite strong when the subareas are within a larger catchment, but it also applies when different catchments are considered. Thus a relation between lag time and area will to some extent include the effect of slope.

It might be noted that studies of travel times and flow velocities in streams (Leopold et al 1964, Pilgrim 1977, 1982) show that velocities remain relatively constant along a stream, both in the steeper headwaters and in the flatter lower reaches. An explanation for this is that when moving downstream along a stream, the decrease in slope is compensated by an increase in depth and hence in hydraulic radius. These therefore indicate that slope is not a dominant factor in determining lag times. Further discussion of the effects of slope on lag parameters is contained in Boyd and Bodhinayake (Australian Journal of Water Resources, 2006).

By requiring only the area A in the lag relations, WBNM greatly simplifies data requirements, and experience has shown that WBNM gives good results using only the area A.

Routing Hydrographs in Stream Channels

This operation is invoked if the subarea is included in the FLOWPATHS block of the runfile. The hydrograph at the top of a stream channel is routed to the bottom of the stream channel. Three options are available (figure 8):

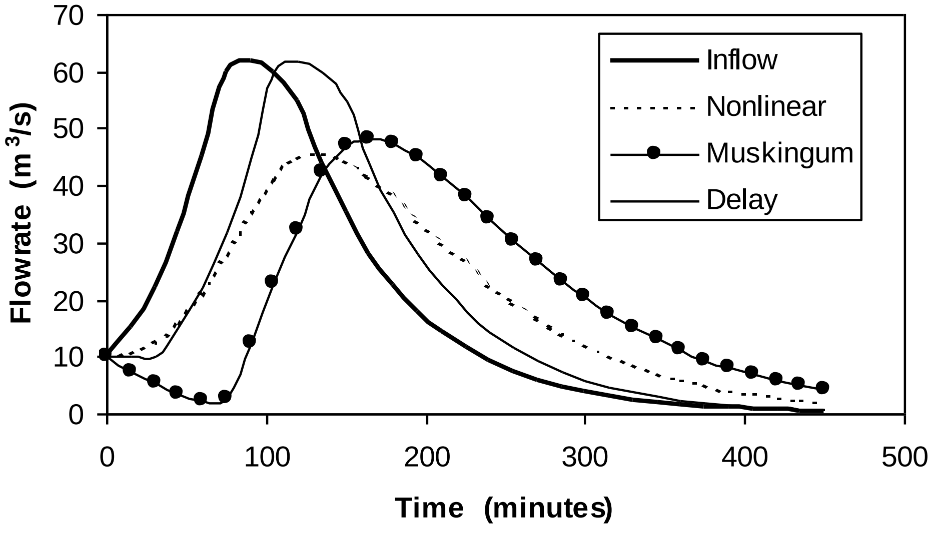


Figure 8. Routing Hydrographs in Stream Channels.

Nonlinear Routing

This was used in the original development of WBNM and is the default option for natural catchments. To give the user more flexibility, equation 4 is modified to

Stream Channel Lag = Stream Lag Factor . 0.6 . Lag Parameter. $A^{0.57}$. $Q^{-0.23}$ (5)

For natural catchments and streams, the Stream Lag Factor is set at 1.0. If however the stream is modified so that flow velocities and lag times change, then the Stream Lag Factor can be adjusted accordingly. For example, if the stream is lined so that velocities increase by 50%, then lag times through the stream channel will be 0.67 of the original value, so the Stream Lag Factor would be adjusted to 0.67. Guidelines for the Stream Lag Factor are :

Natural channel	1.0
Gravel bed with rip-rap	0.67
Excavated earth	0.5
Concrete lined	0.33

Note: In previous versions of WBNM the value 0.6 was entered in the runfile. In recent versions of WBNM, this factor is built in to the model. For natural streams, you should use a Stream Lag Factor of 1.0. The computer program will then automatically factor the lag times for the stream by 0.6

Time Delay

The hydrograph can be simply delayed in time as it passes through the stream channel, without any other modification. Simply specify the time delay in minutes.

Muskingum Routing

Muskingum routing can be applied to the hydrograph. This gives both attenuation and translation of the hydrograph at the bottom of the stream channel. Specify Muskingum K (minutes) and X (range 0 to 0.5) in the runfile.

A knowledge of how Muskingum K and X affect the hydrograph is useful in assigning values to them. If X=0, the hydrograph peak at the bottom of the stream channel will always intersect the recession limb of the hydrograph at the top of the channel. For X=0, as K increases, the lag time increases and the hydrograph peak at the bottom of the channel moves further to the right and down the recession (ie the peak discharge gets smaller and occurs later in time, while still intersecting the recession). For any of these values of K, as X increases from 0 towards 0.5, the hydrograph peak at the bottom of the channel remains at (or close to) the same point in time as for X=0, but the peak discharge increases. When X=0.5, the peak discharge at the bottom of the channel is about the same as the peak discharge at the top of the channel.

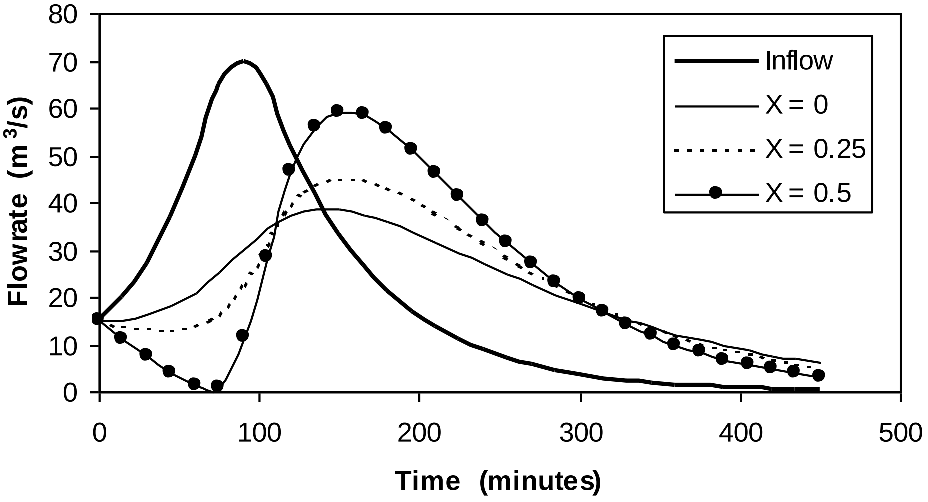


Figure 9. Muskingum Flood Routing – effect of parameter X.

Muskingum-Cunge Routing

Muskingum-Cunge uses the same routing algorithms as standard Muskingum routing, and so will produce the same result for a given set of K and X. The difference is that M-C bases values of K and X on hydrograph and stream channel properties.

$$K = \frac{L}{C}$$

$$X = 0.5 - \frac{Q_p}{2.S.B.C.L}$$

where
L = channel reach length
C = flood wave celerity
Q_p= flood peak discharge
B = channel surface width
S = channel bed slope

Q_p and B are average values along the reach.

Note: When routing hydrographs in stream channels for natural catchments and streams, nonlinear routing with Stream Lag Factor of 1.0 is the default option.

Note: If the Lag Parameter, the Stream Lag Factor, or the subarea area is set to zero, no routing occurs and the hydrograph at the bottom of the stream channel is identical to the hydrograph at the top.

Calculating the Rainfall Hyetograph for each Subarea

Rainfall for recorded storms can be entered as mm/period, mm/hour or percentage. After input, values are converted to mm/hour, and all subsequent references to rainfall are in mm/hour.

The rainfall hyetograph for each subarea is calculated from the rainfall hyetographs at the various rain gauges, as specified. Two options are available:

#####START_INPUT_RAINGAUGE_WEIGHTS - Thiessen weighting factors can be specified for each rain gauge contributing to the subarea. The hyetograph for the subarea is then the weighted sum of all raingauge hyetographs. The Thiessen weights for each subarea will normally sum to 1.0, however you can make them sum to more or to less than 1.0 if desired. You would normally select the nearest rain gauge and give it a weight of 1.0 (or possibly a value greater or less than 1.0 if you want to factor up or down), with the weights for all other gauges set to 0.0.

#####START_CALC_RAINGAUGE_WEIGHTS - uses the coordinates of the raingauges and of the centres of area of all subareas to weight the different raingauges. WBNM finds the nearest rain gauge to the subarea and adopts its temporal pattern. The total depth in the storm for this subarea is a weighted depth of all surrounding rain gauges. The weighting uses the inverse square distance of each raingauge to the subarea centre of area (ie a rain gauge which is far away will have a small weight). Users should use this approach with caution as a poor (non bracketing) set of gauge locations can create unexpected anomalies in the spatial distribution of rainfall depths.

The rainfall can be entered in the runfile at a time period which is different to the time period used for flood routing calculations. In this case, after the rainfall hyetograph for the subarea is calculated from the various raingauges, it is converted to have a time step equal to the calculation period. All subsequent calculations use this calculation period.

Calculating Hydrographs from Pervious Surfaces

This operation is invoked for all subareas whose area is greater than zero.

Pervious surface rainfall losses are first subtracted from the subarea's rainfall hyetograph. Four options are available for pervious surface rainfall losses:

Initial loss-continuing loss rate - a global value of initial loss (mm) and continuing loss rate (mm/hour) can be specified for all subareas, or individual values can be specified for different subareas.

Initial loss - runoff proportion - as before, global values or subarea specific values of initial loss (mm) and runoff proportion (range 0 - 1.0) can be specified.

Horton infiltration - global or subarea specific values of F_0 (mm/hour), Fc (mm/hour) and k (1/hours) can be specified.

Time varying rainfall losses - a continually time varying loss rate can be specified for every period in the rainfall hyetograph. This is useful for recorded storms where the rainfall occurs in distinct bursts and the losses are likely to be different in each burst. The same time varying loss rates apply to all subareas (global values).

The pervious surface runoff hydrograph is calculated from the using excess rainfall hyetograph using equation 3 for the lag time.

If the Lag Parameter is set to zero, no routing occurs and the pervious hydrograph is equal to the pervious surface excess rainfall hyetograph (converted from mm/hr to m^3/s). If the subarea area is set to zero, the pervious hydrograph consists of zero values.

The source area concept of runoff generation, where runoff comes only from those subareas which are saturated, can be modelled with WBNM. Subareas which do not generate runoff are given a runoff proportion of 0.0 (or alternatively, given an initial loss or continuing loss sufficiently high to prevent any excess rainfall generation). Subareas which generate runoff are given appropriate values of initial loss, runoff proportion, or continuing loss.

Calculating Hydrographs from Impervious Surfaces

This operation is invoked if the subarea area is greater than zero and the percentage impervious is greater than zero.

Impervious surface rainfall losses are first subtracted from the subarea's rainfall hyetograph. This consists of an initial loss (mm) only. Continuing losses are assumed to be zero.

Routing of the excess rainfall hyetograph to give an impervious surface runoff hydrograph uses a modified lag relation. Flow velocities on paved or impervious surfaces are known to be higher than on grassed or pervious surfaces, so the lag time for impervious surfaces will be lower. Studies by Rao et al(1972), Aitken(1975) and NERC(1975) give lag times for urban catchments, however these values are based on whole catchments rather than just the impervious surfaces, and consider the urban percentage rather than the impervious percentage. Studies by Schaake et al(1967) and Espey et al(1977), and also the "kinematic wave" equation in Australian Rainfall and Runoff give estimates of lag times for impervious surfaces.

Considering all these studies, lag times on paved or impervious surfaces are often taken to be approximately one tenth of those on pervious surfaces.

Some of these studies indicate that linear routing is appropriate on impervious surfaces, and as a result of this that the exponent of area A will be less than 0.57.

More detailed discussion of these factors is given in Boyd et al(1996, 1999 and 2001).

Based on analysis of 144 floods on 9 urban catchments in Australia (Bufill and Boyd, 1992 and Boyd and Milevski, 1996), WBNM adopts linear routing for impervious surfaces and uses a reduced lag time. The lag relation is

$$\text{Impervious Surface Lag} = \text{Impervious Lag Factor} \,.\, \text{Lag Parameter} \,.\, (A.IMP/100)^{0.25} \qquad (6)$$

The impervious area on the subarea is A.(IMP/100) where IMP is the impervious percentage. The same Lag Parameter as for pervious surfaces (ie a value between 1.30 and 1.80) is used, but is reduced by the Impervious Lag Factor which has a recommended value of 0.10.

If the impervious surfaces are quite small and therefore have a small lag time, the impervious surface runoff hydrograph will be very similar to the excess rainfall hyetograph.

WBNM splits each subarea into pervious surfaces and impervious surfaces. The impervious surfaces respond quickly to rainfall and represent the surfaces directly connected to the drainage system. This "effective" impervious area can be identified from plots of rainfall and runoff depths for a range of storms recorded on the catchment (figure 10). The diamonds represent events with runoff from impervious surfaces only. The squares are events with both pervious and impervious runoff. The effective impervious fraction is given by the slope of the line through the diamonds.

Boyd et al. (1993) in a study of 38 urban catchments, found that on average the effective impervious area was 75% of the total impervious area estimated from maps (figure 11). The same study found that the effective impervious area was 37% of the urban area (figure 12).

Catchments which are fully urbanised (URB = 1) can have a wide range of impervious fractions, because the fully urbanised catchment can have low, medium or high density development. Figure 12 shows that the average impervious area is 37% of the urbanised area, and this value fits well with the trend for URB in the range 0 to 1.0.

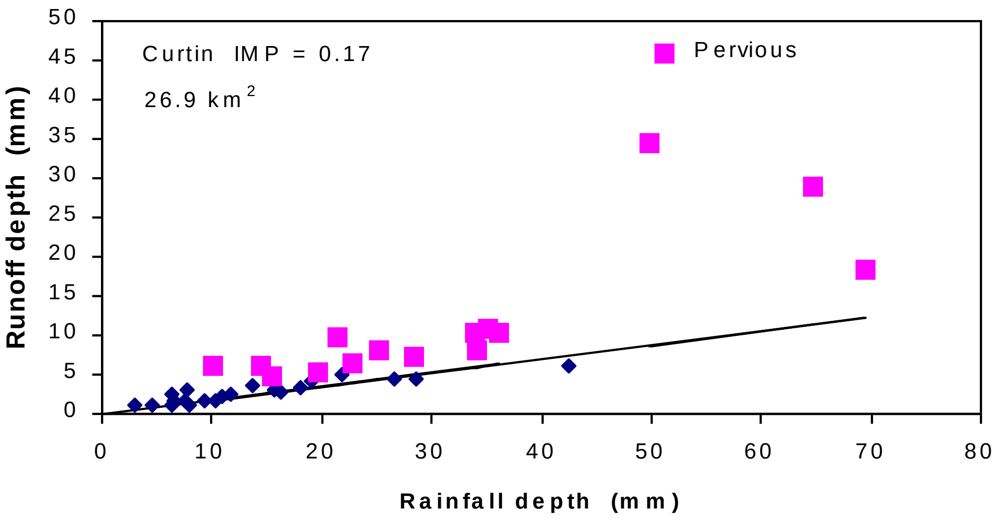


Figure 10. Runoff and Rainfall Depths – Curtin Urban Catchment.

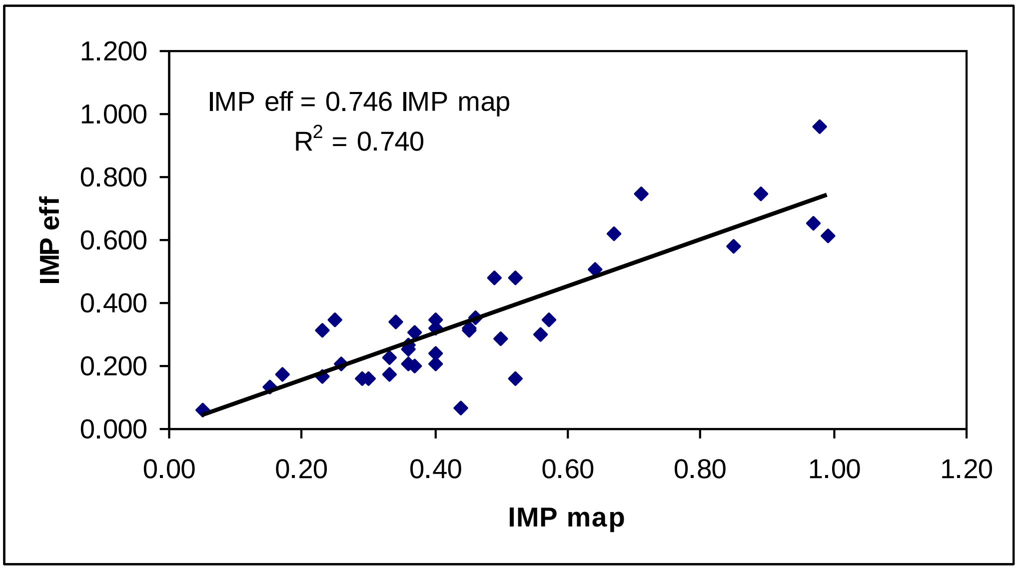


Figure 11. Effective Impervious Fraction versus Map Impervious Fraction.

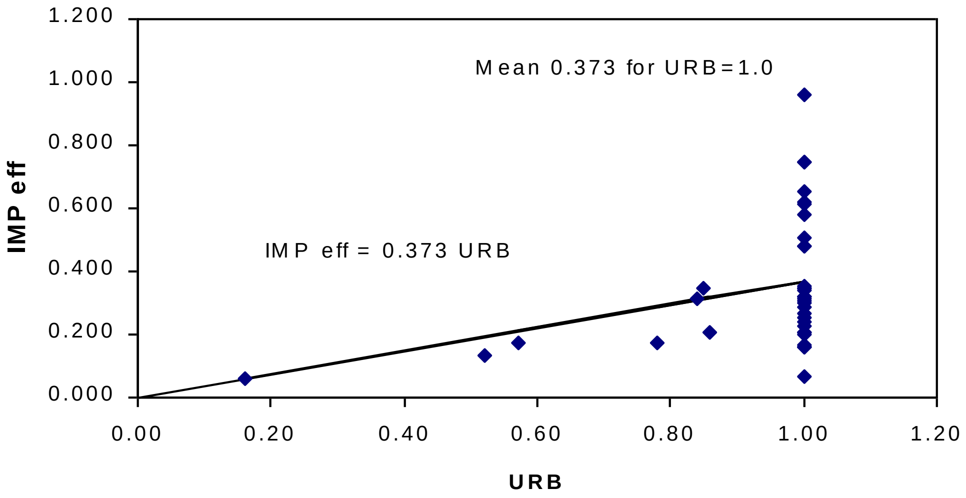


Figure 12. Effective Impervious Fraction versus Urban fraction.

Some urban models, such as DRAINS, split the subarea into impervious, supplementary impervious and pervious surfaces. Runoff from the impervious surfaces has reduced rainfall losses and a reduced lag time. Runoff from the supplementary impervious surfaces has reduced rainfall losses but the same lag time as the pervious surfaces, because it is assumed to flow across the pervious surfaces before reaching the drainage system. This effect can be modelled in WBNM by considering the pervious and supplementary impervious surfaces together and applying an average rainfall loss to them. They will then be routed using the lag time appropriate to pervious surface.

If the Lag Parameter or the Impervious Lag Factor is set to zero, no routing occurs and the impervious hydrograph is equal to the impervious excess rainfall hyetograph (converted from mm/hr to m^3/s). If the subarea area is zero, the impervious hydrograph consists of zero values.

By calculating separate hydrographs from pervious and impervious surfaces, WBNM allows considerable flexibility in modelling. For example, the impervious runoff can be directed to an onsite detention storage, or some combination of the impervious and pervious hydrographs can be directed to the storage. Also, different pollutant loads could be assigned to the pervious and impervious runoff.

A Summary of Parameter Values

Flood routing for conversion of rainfall to runoff on pervious and on impervious surfaces, as well as flood routing in streams, all use particular values of the lag **time**. This lag time depends on the size of the subarea (ie larger subareas will have larger lag times), and WBNM has built-in relations which calculate the lag time for each of these processes. WBNM therefore uses a single value of a lag **parameter** which applies to all subareas. This basic lag parameter is adjusted by lag **factors** to calculate runoff from impervious surfaces, and for flood routing in streams. Thus WBNM essentially has three parameters:

Lag Parameter for conversion of rainfall to runoff on pervious surfaces

Impervious Lag Factor to reduce the Lag Parameter to a value appropriate to impervious surfaces

Stream Lag Factor to reduce the Lag Parameter to a value appropriate to flood routing in streams

For conversion of rainfall to flood runoff on natural catchments, and on pervious surfaces of urban catchments, a Lag Parameter value near to 1.6 will generally be used. Note, the Lag **Time** for each subarea is calculated as the product of the Lag **Parameter** and the subarea size, so that the same value of lag parameter should apply over a wide range of catchment sizes.

Routing in stream channels uses a similar lag relation, since the stream length is related to the subarea size, but one which calculates lower lag times because of the higher flow velocities in the stream. WBNM automatically does this. A Stream Lag Factor is used to adjust for changes to the stream channel. For undisturbed streams such as would be found on natural catchments, a Lag Factor of 1.0 is used. If the stream is modified by clearing, straightening, or concrete lining, flow velocities will increase and lag times will decrease. If, for example, the velocity increased by a factor of 2, the Stream Lag Factor would be 0.5.
Note, the lag **time** for routing in streams is the product of the **Lag Parameter** and subarea size (see previous paragraph) and the **Stream Lag Factor**. Normally, you will have a Lag Parameter which applies to all subareas, and a Stream Lag factor which applies to the stream in the particular subarea. Modifications to stream travel times are modelled by adjusting the Stream Lag Factor. If you reduce the **Lag parameter** for a particular subarea, you will also reduce the Stream Lag time, since this is a product of the lag parameter and the Stream Lag Factor.

Conversion of rainfall to runoff on impervious surfaces uses an **Impervious Lag Factor** to allow for the faster flow velocities on these surfaces compared to pervious surfaces. This will generally be near to 0.10.
Note, as for routing in streams, the **Impervious Lag time** is the product of the **Lag Parameter** and subarea size, and the **Impervious Lag Factor**.

Generally, we recommend that you set a single value of the Lag Parameter for all subareas in the catchment, (usually near to 1.6). Changes to streams can be modelled by adjusting the Stream Lag Factor (or leaving it at 1.0), and runoff from impervious surfaces can be modelled by setting the Impervious Lag Factor to 0.10. See reference:

Boyd, M.J. and Bodhinayake, N.D. (2006). WBNM runoff routing parameters for south and eastern Australia, Australian Journal of Water Resources, 10(1), pp. 35-48.

Effects of Setting Parameters to Zero

It is worth noting the consequences of setting the various parameters and other values to zero. You may be able use this knowledge to model unusual cases.

1) Size of the subarea

If the AREA of a subarea is set to zero, local hydrographs from the pervious and impervious surfaces will not be generated.

Because the Lag Time for stream channel routing depends on the size of its subarea (the Stream Lag Time is a product of the overall model Lag Parameter, the Stream Lag Factor and the subarea size) the stream lag time will be zero and no stream channel routing will occur. The hydrograph at the bottom of the stream channel will be identical to that at the top of the stream channel*.

2) Stream Lag Factor

If the stream lag factor is zero, the lag time in the stream will be zero and no channel routing will occur in that subarea. The hydrograph at the bottom of the stream channel will be identical to that at the top of the stream channel*.

* This only applies to the default nonlinear channel routing option.

3) Impervious Lag Factor

If the Impervious Lag Factor is set to zero, no routing of excess rainfall on the impervious surfaces will occur. The runoff hydrograph from the impervious surfaces will be equal to the excess rainfall hyetograph (adjusted from mm/hour to m^3/s).

4) Overall Model Lag Parameter

Because the Stream Channel Lag Time* and the Impervious Lag Time are products of the overall catchment Lag Parameter and the appropriate Lag Factors, setting the overall Lag Parameter to zero results in no routing of runoff from pervious or from impervious surfaces. These hydrographs will be equal to the corresponding excess rainfall hyetographs (adjusted from mm/hour to m^3/s). Similarly, there will be no routing in the stream channel and the hydrograph at the bottom of the channel will be identical to that at the top of the stream channel.

5) Storage Volumes in Outlet Structures and Local Structures

If you set all values of storage volumes in the Elevation-Storage tables to zero, no storage routing occurs and the outflow hydrograph is identical to the inflow hydrograph. This can be simply achieved by setting the Storage Factor at the end of the BASIN_DETAILS block to zero. This gives a simple way of temporarily switching off a structure such as a flood detention basin.

6) Discharges through Culverts and Weirs

You can quickly adjust the discharge through culverts and weirs by setting the Discharge Factor in the OUTLET_DETAILS block of LOCAL and OUTLET structures. A Discharge factor of 0.9 for example, reduces the discharge for all heads to 90% of the full discharge. This can be used to model partial blockage of culverts and weirs by debris. Setting the Discharge factor to 0 models a completely blocked outlet.

Calculation Time Step

The time step used for flood routing calculations will generally be equal to the time step of the rainfall hyetograph, but it can be smaller, and in some cases this will be necessary.

During routing calculations, if the lag time for a subarea (which changes from time step to time step) becomes smaller than the calculation time step, solution of the nonlinear routing equations is no longer possible. If this occurs, execution ends and a message is sent to the screen telling you to **reduce the calculation time step**. Simply change this value in the runfile, we suggest to half the original value. This is most likely to occur on small subareas when the discharges are large - for example a stream channel in a small subarea near the bottom of the catchment. In extreme cases, when the subarea size is very small, to avoid using an extremely small time step, you may want to replace nonlinear stream channel routing by a time delay.

Flood routing through outlet structures a
nd local structures **may also require a small calculation time step**. This may be necessary if culverts and weirs are used. When the weir becomes active, discharges increase rapidly for a small increase in water level, giving an abrupt change in the elevation-storage-discharge relation, and requiring a smaller calculation step. In extreme cases, a time step which is too large may cause outflow discharges to exceed the inflow peak. If this happens, WBNM will give a WARNING message.

Calibration using Recorded Rainfall and Flood Data

As with any model, it is desirable to calibrate WBNM using recorded flood data wherever possible. In the absence of full hydrograph data, it may be possible to use observed maximum water levels to infer flood peak discharges with which to check results.

If a full set of rainfall hyetograph and flood hydrograph data is available, WBNM should be calibrated on this.

AR&R 1987 Design Storms (DES87)

Design storms are calculated using procedures in Australian Rainfall and Runoff. Storm durations from 5 minutes to 72 hours, and average recurrence intervals from 1 to 500 years are calculated. Durations allowed are: 5, 6, 10, 15, 20, 25, 30, 45, 60, 90, 120, 180, 270, 360, 540, 720, 1080, 1440, 1800, 2160, 2880, 4320 minutes. ARIs allowed are: 1, 2, 5, 10, 20, 50, 100, 200 and 500 years. Probable Maximum Precipitation storms are also allowed.

WBNM takes the 9 basic IFD coefficients and first calculates the average rainfall intensity. This is then distributed into the appropriate design storm temporal pattern, depending on the zone.

If the selected calculation period is greater than, or is not an even divisor of the design storm temporal pattern time period, WBNM writes a message to the screen and adjusts the calculation period appropriately.

The basic IFD coefficients can be entered directly into the runfile used for the run, or they can be contained in a separate IFD file as part of a data bank of IFD coefficients. In this case, the name of the IFD file and the name of the rainfall station is entered in the runfile. A sample external IFD file, WBNM.IFD is included with this software.

AR&R 1987 Embedded Design Storms (EMB87)

The critical duration burst in a storm often occurs after a considerable amount of prior rainfall. In this case, streams and storage reservoirs on the catchment may be running part full at the start of the design burst. WBNM models this by having an embedded design storm option (Rigby and Bannigan, 1996). You can specify the average recurrence interval ARI and duration of the critical burst, and the ARI and duration of the longer storm event within which the burst is embedded.

WBNM first calculates the design rainfall hyetograph for the longer event, then replaces the most intense portion of the event by the burst design rainfall hyetograph. The burst hyetograph is inserted into the event hyetograph by centering its most intense rainfall ordinate over the most intense ordinate of the event. The remaining ordinates in the event (outside of the burst duration) are then adjusted to give the correct depth for the total event.

The resulting embedded design storm consists of a short duration burst which is equal to the standard design storm of this duration. This is embedded within a longer event which has the correct total depth for a design storm of this duration, but with some ordinates (those outside the burst) which have been adjusted slightly.

Note that this method is not included in AR&R and users should use their judgement to determine whether the resulting design flood is appropriate.

AR&R 1987 Probable Maximum Precipitation

Probable Maximum Precipitation storms are calculated by WBNM using procedures in Australian Bureau of Meteorology Bulletin 53 (1994). This applies for storm durations up to 6 hours and catchment areas up to 1000 km^2. Durations allowed are: 10, 15, 20, 25, 30, 45, 60, 90, 120, 150, 180, 240, 270, 300, 360 minutes. The average rainfall intensity is distributed into the Bureau's generalised temporal pattern.

In some parts of Australia, Bulletin 53 is limited to less than 1000 km^2 and less than 6 hours, and seasonal variations are possible. Bulletin 53 also allows spatial variation of PMP. WBNM does not calculate this spatial variability. See Bulletin 53 for more details. Note that Bulletin 53 has undergone several revisions, see their website www.bom.gov.au/hydro/has/bull53

To calculate a PMP storm, specify 9999 in the Average Recurrence Interval field in the runfile.

AR&R 1987 ARF Calculation

The Areal Reduction Curves in Book 2-1 of Australian Rainfall and Runoff (1987, 1998) can be applied to reduce rainfall intensities depending on storm duration and catchment area. These are automatically calculated by WBNM (set to –99.9), however you can override this and use your own factor (eg 0.95).

AR&R 2016 Design Storms (DES16)

ARR 2016 introduced the concept of an ensemble of storm temporal patterns for design flood estimation (DFE) rather than the single location specific pattern provided in AR&R 1987. It also provided different temporal patterns for catchments > 75km2 in area (areal patterns) than for catchments <= 75 km2 in area (point temporal patterns). The Bureau of Meteorology (BOM also substantially updated the IFD data for Australia.

To establish the DFE at the catchment outlet for a particular AEP, WBNM calculates the average peak discharge from the storm ensemble for each duration, with the DFE for that AEP being the maximum of the average peak discharges for the spectrum of discharges.

It is important to note that while this DFE discharge is associated with a particular duration it is not necessarily associated with any of the ensemble temporal patterns and therefore is not necessarily associated with any particular hydrograph produced by the ensemble. If a peak discharge is the only objective this is not an issue, however if a hydrograph is required (for example to design a basin or quantify the time a road is overtopped) then this procedure creates a problem. Until this is formally resolved, WBNM adopts the closest ensemble hydrograph that has a peak discharge equal to or greater than the DFE, as the adopted design flood hydrograph. As this procedure typically leads to two different peak discharges, both the true DFE peak discharge and the peak of the adopted hydrograph are included in the storm summary, As the time to peak and volume are design hydrograph parameters, the values included in the storm summary are those of the adopted hydrograph.

WBNM outputs the calculated DFE for each duration for each subarea if the crit_des file output is enabled in the ini file. It is important to note that this table only provides the correct DFE at the catchment outlet, as partial area effects may be affecting the calculated peak discharge at locations internal to the catchment. WBNM provides an option to nominate an internal location (subarea) at which the correct DFE is calculated, reflecting the potentially different ARF and temporal pattern type applicable, if that location was to be considered the catchment outlet. In this way the magnitude of the partial area effect can be quantified and a decision made as to how far this potential change to peak internal discharges needs to be pursued.

At present there are still questions as to whether the DFE procedure should be mean/average or median based and whether the ‘adopted’ design hydrograph should be simply the hydrograph with a peak closest to the DFE. In addition, there are possible benefits in using an uneven number of temporal patterns (not 10 as at present) so that the pattern with the median discharge could be used both for the DFE discharge and associated DFE hydrograph. Resolution of these matters may ultimately lead to change in WBNM’s calculation of design discharges.

As the present BOM IFD data for events rarer than the 1% AEP event is restricted to durations greater than or equal to 24hours. As this prevents simulation of critical duration design storms for all but very large catchments, WBNM does not accommodate theses rarer AEPs at present. It is understood that the BOM is working on extending the duration range for these rarer events and will be extending their dataset on the web to cover these additional shorter durations sometime in the new year (2018).

AR&R 2016 Probable Maximum Precipitation

At the time of preparing this guide, It is not clear whether the PMP will be revised by the BOM or not. Until this is clarified, WBNM does not include PMP based DES16 storms.

AR&R 2016 ARF Calculation

ARR 2016 includes an updated procedure for calculation of the Areal Reduction Factor applicable to a catchment. WBNM adopts this procedure if the user enters a -1 for ARF in their ARR 2016 storm block in their runfile.

User Defined Storm Temporal Patterns

You can apply your own design temporal patterns to a range of storms, by using the #####START_RECORDED_RAIN option and specifying PERCENT as the Rainfall Units rather than MM/HOUR or MM/PERIOD. Simply enter the temporal pattern as percentages of the total depth in the storm, then add the total storm depth (mm). Sample runfile Multistorm.wbn shows an example.

Storm & Burst Losses

ARR1987 was and ARR 2016 remains a ‘storm burst’ based simulation procedure.

Only a REC event can be considered a ‘storm’ event.
As such, losses for a REC storm event are typically full storm losses as derived from the resulting runoff hydrographs. The assessment of appropriate DES87 and DES16 event losses is however complicated by the presence of initial and potentially continuing losses (pre burst losses) that would occur prior to a burst of particular duration. In a DES87 event it has been common practice for this to be recognised by reducing the IL in a short burst simulation in recognition that these losses would already have occurred in pre burst rainfall, with the full IL applying for longer duration burst events. ARR 2016 has refined this process, providing statistically derived ‘storm’ losses across Australia with typical pre-burst losses for different duration events across Australia. If losses is set to ARR2016, WBNM uses these data to establish design losses appropriate to the event being simulated, in line with the recommendation of ARR 2016. User supplied subareal specific losses remain an option for ARR 2016 design storms as foe ARR 187 design storms.

Recorded Hydrographs

Up to ten recorded hydrographs can be used. They can occur at the TOP or the BOTTOM of the subarea. The time step can be different to the calculation period and different to the time step of the rainfall. WBNM converts the recorded hydrograph to have a time step equal to the calculation period.

Recorded hydrographs can be entered as Discharge hydrographs, or as Stage hydrographs. If a rating curve is supplied in the runfile, Stages are converted to Discharges, and both are written to the output file.

For a recorded hydrograph at the BOTTOM of the catchment outlet, WBNM calculates the average depth of recorded runoff by dividing its volume by the total catchment area..

Imported Hydrographs

Up to ten imported hydrographs can be used. They can enter at the TOP or BOTTOM of the subarea. The time step can be different to the calculation period and different to the time step of the rainfall. WBNM converts the imported hydrograph to have a time step equal to the calculation period.

A hydrograph imported to the TOP of a subarea is added to other hydrographs at this point and routed down the stream channel. A hydrograph imported to the BOTTOM of a subarea is added to the other hydrographs at that point.

Imported hydrographs can be entered as Discharge hydrographs, or as Stage hydrographs. If a rating curve is supplied in the runfile, Stages are converted to Discharges, and both are written to the output file.

Baseflow Hydrographs

WBNM is an event model and calculates surface runoff hydrographs from storm events. It does not model baseflow. However, if you want to include a baseflow hydrograph in your output, you can do this by *importing* a baseflow hydrograph.

Baseflow hydrographs can be imported to any point in the model. If you import it to the BOTTOM of the lowest subarea, the hydrograph at the catchment outlet will consist of the surface runoff hydrograph at this point, sitting on top of the baseflow hydrograph. The imported baseflow hydrograph can have as many ordinates as you want, listed at any time step. If you want a straight line baseflow hydrograph, you can simply specify 2 ordinates, with time step equal to the time between these two ordinates.

If you import baseflow hydrographs to points further upstream in the model, they will be added to the surface runoff at these points, and the *total hydrograph will be routed* through successive stream segments to the outlet.

If baseflow hydrographs are imported into WBNM, runoff volumes and depths will include the baseflow. The runoff volumes therefore will exceed the excess rainfall volumes and depths, by the volume contained in the baseflow hydrograph. Similarly, peak discharges will increase, being the sum of the surface runoff and the baseflow discharges.

Flood Routing in Storage Reservoirs/ Flood Detention Basins

This operation is invoked if the subarea is included in the OUTLET STRUCTURES block or in the LOCAL STRUCTURES block.
The same flood routing procedures are used for Outlet Structures as for Local Structures, except for the following:

Local Structures take local runoff from the pervious and impervious surfaces in the subarea. You specify the proportion of each runoff hydrograph which goes into the storage, with the remainder bypassing it. For example, the local structure can take 90% of the impervious surface runoff and 20% of the pervious surface runoff. All outflow from the local structure goes to the bottom of the subarea where it is added to the bypassing flow. This flow can be delayed by a specified time. Local Structures will generally be used for onsite detention flood routing (see later section).

Outlet Structures are located at the outlet of the subarea. They take all runoff at this point: flow in the stream channel routed from the top of the subarea, local pervious and impervious runoff which bypasses the local structure, and local pervious and impervious runoff which has been routed through the local structure. The outlet structure also takes any surcharging flows from subareas further up the catchment which have been directed to the bottom of this subarea. Note, any surcharging flows which have been directed to the top of this subarea have already been included in the hydrograph routed from the top to bottom of the stream channel. Outflow from each of the various outlets of the outlet structure (ie culverts and weirs) can be directed to different downstream subareas.

Note that a local structure only takes runoff from the subarea in which it is located, whereas an outlet structure can take runoff from all upstream subareas.

The following points refer to both local structures and to outlet structures:

Inflow to the storage reservoir is routed through it using a level pool (Puls) routing procedure. Flood routing through the storage requires a table of elevation-storage volume-discharge values (HSQ). The elevation-discharge values can be entered directly, or can be calculated

The H-S and H-Q relation uses linear interpolation between the elevation H values specified in the runfile. For good definition of the curvature of these relations, the H values should be entered at relatively small increments. However you do not need hundreds of these values. WBNM currently allows up to 50 values in the H-S table, and this will be sufficient for most cases.

In the HSQ table, the bottom-most elevation does not have to be zero (ie it can be AHD), and the bottom-most storage volume does not have to be zero (ie there can be water stored below this elevation). However, the bottom-most **discharge must be zero.** For example:

Elevation H (m)	Storage Volume S (10^3 m3)	Discharge Q (m^3/s)
100.0	85.0	0.0
100.5	92.3	4.6
101.0	97.8	9.3

If the elevation-discharge relation is calculated, you can enter up to ten blocks of culverts and/or weirs, each block having different size and elevation. Within each block, you can have any number of identical culverts. If, for example, you have three identical culverts within a block, the total discharge through this block of culverts is three times the discharge through one of the culverts.

Because the elevation-discharge relation changes at the invert of each block of culverts, for good definition of the relation it is desirable that one of the points in the elevation-storage-discharge relation coincides with the invert of each block. When designing a storage, you may want to adjust the elevation of the culvert or weir inlets by a trial and error procedure. To avoid you having to add or subtract additional points, WBNM automatically checks and adds them at each culvert/weir elevation.

If storage volumes are set to zero in the storage, no flood routing occurs, and the outflow hydrograph is identical to the inflow hydrograph. The maximum water level however is noted. This could apply, for example, to culverts under roads where the ponded storage volume is negligible. The maximum water level will depend on the culvert size, and this information can be used to determine whether surcharging of the road occurs.

You can temporarily set these storage volumes to zero, while retaining the original elevation-storage relation, by setting the Storage Factor to zero. The Storage Factor can be used for other purposes. If you are designing a flood detention basin and want to try larger or smaller storage volumes, simply change the value of the Storage Factor. You can also apply Discharge Factors to the discharges from the various outlets of the Structure (see Culvert and Weir Hydraulics section).

NOTE: If you set both the Discharge Factor and Storage Factor to zero, you have created a meaningless structure. By setting Discharge Factor = 0, no outflow occurs and the inflow volume accumulates in the structure. However, since Storage Factor = 0, there is no volume in the storage to accommodate the inflow. The result is that the water level in the structure goes to infinity. WBNM will give you an error message in this case.

The surface area of the storage can be specified, and WBNM adds any rain falling on this surface area directly into the inflow hydrograph. If you allocate a part of the subarea's total area to this surface area, you may wish to deduct the value from the total area of the subarea in the SURFACES block of the runfile.

If you want the storage reservoir to be part full at the start of the storm, you can nominate an initial water level from which flood routing commences. The outflow volume will be greater than the inflow volume, by an amount equal to the initial volume in the storage. The initial discharge will be greater than zero, equal to the initial water level in the basin.

If the lowest block of culverts/ weirs is set at an elevation above the bottom of the storage, the resulting dead storage volume will be filled before outflow commences. The outflow volume will be less than the inflow volume, by an amount equal to the volume of dead storage. The initial discharge will be zero until the dead storage is filled.

If, during the storage routing calculations, the maximum water level in the HSQ table is exceeded, WBNM extends the Elevation-Storage-Discharge table by linear extrapolation of the top 2 points in the table. If this happens, you should extend the HSQ table in the runfile to cover these higher values.

Outflow hydrographs from outlet structures and local structures are written to 3 decimal places in the output metafile (hydrographs are written to 2 decimal places everywhere else). You can make use of this when modelling very small catchments, where you might want discharges to more than 2 decimal places. If this is the case, use a dummy outlet or local structure which has zero storage volumes. An example of a dummy storage in the runfile is:

```
#####START_OUTLET_STRUCTURES_BLOCK#|###########|###########|########
         1
#####START_OUTLET_STRUCTURE#1
DUMMY STORAGE WITH ZERO VOLUME
SUB3
#####H_S_Q
#####START_OUTLET_DETAILS
         1
#####HSQ
      1.00
SUB5
TOP
      0.00
#####END_OUTLET_DETAILS
#####START_BASIN_DETAILS
         2
       0.00       0.00       0.00
       1.00       0.00       1.00
       0.00
       0.00
#####END_BASIN_DETAILS
#####END_OUTLET_STRUCTURE#1
#####END_OUTLET_STRUCTURES_BLOCK###|###########|###########|#########
```

The following points refer to Outlet Structures:

After flood routing through the storage reservoir, different portions of the outflow hydrograph can be directed to nominated downstream subareas. WBNM does this by allowing the outflow hydrograph through each of the ten blocks of culverts/ weirs to be directed as nominated. Each directed hydrograph can be delayed by a specified time. Normally, the hydrographs from all blocks of culverts/ weirs will be directed to the immediately downstream subarea (as specified in the TOPOLOGY block), and they will enter at the **top** of this subarea. In the case of surcharging flow however, the outflow from a block may be directed to some point further down the catchment, or even completely out of the catchment. These surcharging flows may go to the **bottom** of the destination subarea. An example of this is a storage reservoir at the outlet of a subarea where the culvert capacity is exceeded, the raised water levels will overtop the stream bank which acts as a weir, directing flow to some downstream point. WBNM allows you to nominate whether each outflow goes to the top or bottom of the nominated subarea. If there is an outlet structure at the bottom of the destination subarea, flows directed to the bottom of this subarea go into the outlet structure.

Note that when you direct flows from all 10 blocks of outlets to the top of the immediately downstream subarea, all of these flows contribute to the tailwater level in the downstream channel, and you can use the #####H_S(TWF), #####H_S(TWR) or #####H_S(TWC) options to use outlet control in the culvert hydraulics. If some of these blocks of outlets go to other subareas, and these outlets are **weirs**, you can also use outlet control for the remaining culverts. However, if you direct flows to other subareas through culverts, then they will have different tailwater conditions than in the downstream channel, and you cannot use the outlet control option. See the following Culvert and Weir Hydraulics section for more details.

You can also direct different portions of the Outlet Structure outflow to different nominated downstream subareas when using the #####H_S_Q option. In this case the table consists of a column of elevations, followed by a column of storage volumes, followed by a series of columns of discharges, each being the part of the total outflow which is to be directed to the nominated downstream subarea. For any elevation, the sum of the discharges in all columns is the total discharge from the storage at that elevation. This total discharge is used, together with the elevation and storage volume, for reservoir routing calculations. The routed hydrograph is then split into components, with each component going to its nominated downstream subarea. Consider for example the following HSQ table:

Elevation (m)	Storage Volume (th. m^3)	Discharge to subarea 1 (m^3/s)	Discharge to subarea 2 (m^3/s)	Discharge to subarea 3 (m^3/s)
10.00	0	0	0	0
10.25	6.0	1.4	2.5	0
10.50	13.0	2.6	4.8	3.6
10.75	20.0	3.9	7.3	7.5

For flood routing calculations, the outflow discharges from the storage for the successive elevations would be 0, 3.9, 11.0, 18.7. If, after flood routing, the total outflow discharge at a particular time is 14.85, then the water elevation in the storage is 10.625 (linear interpolation between 10.50 and 10.75) and the discharges going to subareas 1, 2 and 3 at this time are 3.25, 6.05, and 5.55 respectively.

Culvert and Weir Hydraulics

The methods of the US Federal Highway Administration(1965), commonly known as the Concrete Pipe Association of Australia charts, are used to determine culvert hydraulics for both local structures and for outlet structures. WBNM uses the equations of Boyd(1987a,b) fitted to these nomographs.

Weir hydraulics use SI units (metres, m^3/s) with the appropriate discharge coefficient for these units. For sharp crested weir this is close to 1.80 and for a broad crested weir 1.70. Currently, we do not apply tailwater elevations to calculate weir flows (ie downstream submergence is not included).

If you specify #####H_S in the runfile, culvert hydraulics are calculated assuning INLET control applies. You need to enter the number, type (box or pipe), entrance type, and invert elevation in the runfile. WBNM then calculates the elevation-discharge relation.

If you specify #####H_S(TWF), #####H_S(TWR), or #####H_S(TWC), culvert calculations check both INLET control and OUTLET control, and adopt the overall controlling case. WBNM first calculates the discharge corresponding to each water elevation in the elevation-storage table of the runfile, assuming that inlet control applies. Next, WBNM takes each of these discharges and calculates the corresponding headwater depth assuming that outlet control applies. These two sets of headwater depths are compared, and the larger (and therefore controlling) value adopted. Finally, the adopted headwater-discharge relation is interpolated to give a table of elevation-discharge values, at the original elevations specified in the runfile. To improve the interpolation accuracy, WBNM uses intermediate elevations in the calculation, in addition to those specified in the runfile.

Outlet control calculations first assume that the culvert flows full. WBNM then checks the calculated headwater depths, and if the ratio HW/D is less than 0.75, and if flow in the culvert barrel is subcritical, the headwater is recalculated for part full flow in the barrel.

For outlet control, friction losses in the barrel use the appropriate velocity in the barrel, either for full flow or for part full flow (for part full flow the flow depth in the barrel is calculated from Manning). Exit losses use the appropriate exit velocity. If TW<Dc, the exit velocity is based on Dc. If Dc<TW<D, the exit velocity is based on TW. If D<TW, the exit velocity is based on full culvert flow.

The effective tailwater depth is set equal to the larger of TW and (Dc+D)/2.

All of these calculations are in accordance with the US Dept. of Transport and Concrete Pipe Association methods.

Note: TW is tailwater level, D is culvert depth, Dc is critical depth. Strictly, HW is the level of the upstream energy line above the culvert invert. If water is ponded so that approach velocities are low, HW is close to the headwater depth.

For outlet control, tailwater elevations in the downstream channel can be specified in 3 ways:

#####H_S(TWF) The tailwater elevation is FIXED for all discharges, in which case only one tailwater elevation is required.

#####H_S(TWR) The tailwater elevation varies with discharge, depending on a RATING table supplied in the runfile.

#####H_S(TWC) The tailwater elevation varies with discharge depending on the cross section of the trapezoidal downstream CHANNEL. In this last case, the runfile must contain the channel dimensions and Manning coefficient.

For the rating and channel options, the tailwater elevation-discharge relation is calculated as follows. For each storage reservoir water elevation specified in the runfile table, the discharge in the downstream channel is calculated assuming inlet control applies to all blocks of culverts. These flows are summed to give the total flow in the channel for each storage reservoir water elevation. The corresponding tailwater elevations are then calculated. This procedure is not quite correct, in that outlet rather than inlet control may apply, so that the discharges used to set the tailwater may be slightly high. However this difference in discharge should be small in most cases, and the difference in tailwater depth will be even smaller. This procedure should give quite acceptable tailwater elevations.

If **all** blocks of culvert outlets go to the top of the immediately downstream subarea, you can use outlet control (by specifying #####H_S(TWF), #####H_S(TWR) or #####H_S(TWC). However if some of these blocks of **culverts** go to different subareas, then the tailwater conditions are different for these culverts and outlet control **cannot** be applied.

If all blocks of outlets which are **culverts** go to the top of the immediately downstream subarea, and some of the blocks which are **weirs** go to different subareas, then since the weir flows are unaffected by tailwater, you can specify ##### H_S(TWF), #####H_S(TWR) or

#####H_S(TWC). In this case the flows from the culverts are used to calculate the tailwater elevation in the downstream channel.

If inlet control applies (#####H_S) then the tailwater is not relevant and you can direct flows from any culverts and/ or weirs to any subareas.

If you want to direct flows from the **culverts** to **different** subareas, and you want to apply outlet control, you can do this by separately calculating the elevation-discharge relation for each block of culverts (using its relevant tailwater condition) then specifying the #####H_S_Q option.

Note that scourable spillway discharges are calculated separately and are not included in the tailwater calculations.

You can trace the various steps as WBNM calculates the headwater-discharge relation for the culvert by setting the OUT_CULVERTS flag to TRUE. Detailed results are then written to an output file RUNFILE_CULV.OUT. Since outlet control calculations are complex, you should check this output file to confirm results are correct.

Note, WBNM uses **elevations** for water level in the basin, for culvert invert levels, for downstream channel bed levels, and for tailwater levels. You must enter elevations, not depths in the runfile.

You can apply a Discharge Factor to each block of culverts or weirs. This can be used to model full (factor equals 0) or partial (factor between 0 and 1) **blockage by debris**. All discharges in the elevation-discharge table are multiplied by this factor. If the factor for all blocks of culverts is zero, no flow leaves the structure, and the water level in the structure rises until the entire volume of water in the inflow hydrograph is stored. You can also use this Discharge Factor when designing flood detention basins, to allow a simple increase or decrease of discharges. You can apply this discharge factor to culverts and weirs, but it is fixed at 1.0 for scourable weirs.

Debris can block a culvert from the top down (floating debris) or from the bottom up (heavier debris). WBNM currently does not consider these specifically, but allows the user to select a discharge factor appropriate to the particular case.

Note that when you apply a Discharge Factor to culverts, the discharges are first calculated assuming no blockage. These results are written to the culvert details file RUNFILE_CULV.OUT. It is only when flood routing calculations are carried out in the storage (Outlet Structure or Local Structure) that the discharges are adjusted using the discharge factor. These adjusted discharges can be viewed in the output metafile (in the #####START_HSQ_OUTLET_STRUCTURE and #####START_HSQ_LOCAL_STRUCTURE blocks). For culverts with outlet control, and tailwater controlled by a rating table or by the downstream channel dimensions, this assumption will not quite hold. This is because the culvert blockage may reduce the discharge in the downstream channel, and hence its tailwater level and so the corresponding culvert elevation-discharge relation. This effect is likely to be small.

Blockage of Culverts by Debris

The Discharge factor discussed in the previous section can be used to model blockage of culverts and weirs by flood mobilised debris. If the culvert/ weir is fully blocked, discharges are zero and the Discharge Factor is 0.0. If the culvert/ weir is part blocked, and say 60% of the all clear flow passes through under a given headwater, then Discharge Factor is 0.60, etc.

You can specify a time at which the blockage occurs, and the culverts/ weirs will remain open until that time, thereafter being blocked according to the specified Discharge Factor. Blockage Time is entered in minutes. If a negative time is entered (ie –99.0) then blockage occurs at the end of the most intense rainfall period in the storm.

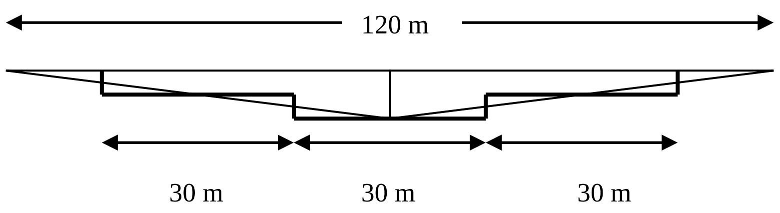
The Blockage Time is entered on the same line as the Discharge Factor. If this field is left blank, blockage occurs at time zero; if it is a positive number, blockage occurs at that time; if it is a negative number, blockage occurs at the end of the most intense rainfall period.

Blockage of culverts/ weirs affects flood routing through Local Structures and Outlet Structures. Sample runfile Storage_Blockage.wbn gives an example. When the structure has several outlets, eg box culverts, pipe culverts and weirs, then the *blockage time entered for the first of these outlets in the wbn runfile is adopted as the blockage time for all outlets*. Each of these outlets will block according to the entered Discharge Factor.

Culverts and weirs can be blocked, but scourable embankments cannot be.

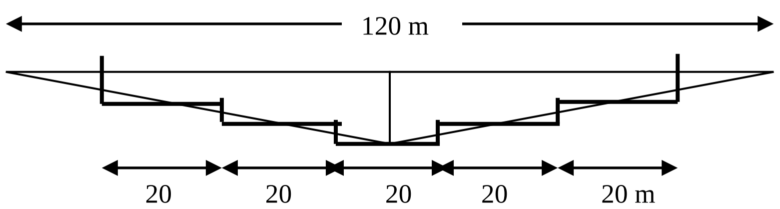
Flow Over Road Embankments

Flow over road embankments will typically occur at a low point in the road surface elevation. This can be modelled using a set of horizontal broad crested weirs, each at a specified elevation. As an example, a stretch of road 120 m long with a 0.9 m dip could be modelled with 3 horizontal weirs. These could be a 30 m long weir at elevation 0 m (the low point of the dip) and two 30 m long weirs at elevation 0.45 m:



Dip in Road Modelled with 3 Horizontal Crested Weirs

As an alternative, the 120 m stretch of road could be modelled with 5 horizontal weirs. These could be a 20 m long weir at elevation 0 m, two 20 m long weirs at elevation 0.30 m, and two 20 m long weirs at elevation 0.60 m:



Dip in Road Modelled with 5 Horizontal Crested Weirs

Hydraulic calculations show that discharges calculated using sets of 3 and 5 weirs generally agree within 10%. These discharges are also similar to those calculated using the triangular weir cross section, depending on the weir coefficients chosen.

Stage Hydrographs

WBNM can calculate a Stage hydrograph as well as a Discharge hydrograph at the outlet of every subarea. To do this, place an Outlet Structure in the subarea, and give it an Elevation-Discharge relation. This can be done using #####H_S_Q (see section on Flood Routing in Storage Reservoirs). If you set all values of the storage volume to zero in the Elevation-Storage-Discharge table, there is no flood routing, and therefore no effect on the hydrograph, but the Stage corresponding to every discharge value in the hydrograph is calculated. This feature is useful for placing rating tables at selected points to obtain stage hydrographs. If the stage is controlled by a structure of culverts and/or weirs, you can use the #####H_S, #####H_S(TWF), etc options, and the Elevation-Discharge relation is automatically calculated.

Preliminary Flood Detention Basin Design

The following guidelines allow rapid preliminary design of onsite detention storages and flood detention basins. They are based on the paper by Boyd(1980), and appear in Australian Rainfall and Runoff Book 5, section 1.5.6, and Book 8 section 1.6.6.

The objective is to reduce the inflow peak discharge I_p to some lower value Q_p. The storage volume S_{max} which must be provided to do this is a function of the volume of water in the flood V. A simple but approximate relation is:

$$S_{max} = V.\,(1 - Q_p / I_p) \qquad (7)$$

By setting the SUM_OUTLET_STRUCTURES flag and the SUM_LOCAL_STRUCTURES flags to TRUE, values of V, I_p and Q_p are written to the screen and can be used in equation 7.

Once the required storage volume S_{max} has been determined, the known elevation elevation-storage relation for the basin can be used to note the corresponding maximum water level H_{max} reached in the storage. A culvert can then be selected to pass the required discharge Q_p under this head H_{max}.

This procedure thus allows rapid determination of the required storage volume, and of the required outlet size. The preliminary design should then be checked by running WBNM with these culvert dimensions as an outlet structure or local structure, adding a high weir for high flows if necessary.

Scourable Spillway

Scourable or fuseplug spillways are one method of enlarging spillway capacity during extreme floods (Vermeyen and Mares, 1992).

WBNM models scourable spillways where the spillway is designed to scour as water discharges over it. The scourable portion is incorporated into a standard spillway with specified culvert and spillway dimensions. Only one of the outlets can be a scourable spillway, the remaining can be blocks of pipe or box culverts or weirs.

The scourable spillway is built into a standard spillway, with its crest elevation at the same level as the standard spillway. The smaller pilot channel forms a depression in the spillway crest and starts the scouring process.

In addition to the standard spillway crest elevation and crest length, the scourable spillway needs the following eight values :

1. Scourable crest length (perpendicular to the flow direction)
2. Scourable bottom elevation

These define the horizontal and vertical limits of the scoured section.

The scourable spillway cross section is trapezoidal. You will have to specify the :

3. Crest width at the standard spillway elevation
4. Width at the scourable spillway bottom elevation

5. Pilot channel crest length
6. Pilot channel crest elevation

As water begins to discharge over the scourable spillway, it first flows over the smaller pilot channel. As this happens, the volume scoured over each time step acts to enlarge the flow section area. Thus the scourable flow section area is initially in the pilot channel, and gradually enlarges to occupy the total area defined by the scourable crest length and bottom elevation.

You will also specify the Scour Factor (7), which is the volume of water passing over the scourable spillway section per unit volume of earth scoured. A survey of embankment collapse by MacDonald and Langridge-Monopolis (1992) indicates that the scour factor depends on the head of water behind the scourable embankment. Greater heads of water produce higher flow velocities and require smaller outflow volumes to produce the same volume of scoured material. For heads less than 5 m the Scour Factor can be 500 m^3/m^3 or more, reducing to 300 m^3/m^3 or less for heads greater than 10 m.

Finally, you will specify the minimum time from the start of the event at which scour commences (8). Scour will not start until this time or later. You can use this to model a recorded flood event where it is known that the spillway scoured out at a known time.

As the scourable section scours, it enlarges from the pilot channel section to occupy the section defined by the scourable crest length and scourable bottom elevation. The scoured length and depth enlarge in the same proportion, ie when the scoured elevation is half way between the pilot channel crest elevation and the scourable bottom elevation, the scoured length is halfway between pilot channel crest length and the scourable crest length.

WBNM adopts a rectangular scoured section, looking in the downstream direction, which together with the upstream and downstream embankment slopes, gives a trapezoidal prism as the scored volume. MacDonald and Langridge-Monopolis report that the side slopes of the scoured section (looking in the downstream direction) may be 2V:1H. Adoption of a rectangular section ie vertical side slopes as opposed to 2:1 side slopes, should not have a large effect. For example, with a scourable length of 50 m and scoured depth of 10 m, 2:1 side slopes gives a top scoured length of 60 m and bottom scoured length of 50 m. Adoption of vertical side slopes in WBNM gives a rectangular scoured section 54 m long for the same scoured volume.

Note that the elevation-discharge relation is recalculated after every time step and is thus continually changing. For good definition of the relation, it is desirable that the elevation-storage-discharge relation between the scourable bottom elevation and the maximum expected water level be defined using reasonably small increments of elevation. For the same reason, if the scouring causes dramatic changes to the spillway flow section, rapid changes to the elevation-discharge relation result, and **it may be necessary to use a smaller time step for calculations**.

When the scourable spillway scours out, the rapid increase in flow section area allows a rapid outflow, and it is possible for the outflow peak discharge in practice to exceed the inflow peak discharge. WBNM models this effect.

If the dbg_run flag is set to TRUE, you can track the performance of the scourable spillway as the flood passes. Values of inflow, outflow, flow over the scoured section, water elevation, current scoured crest elevation and length, and volume scoured are given versus time.

If you set the OUT_SCOURABLE flag to true, these values are written to an output file RUNFILE_SCOUR.OUT. This file contains a complete history of the enlarging spillway section, water level, outflow discharge, and scoured volume.

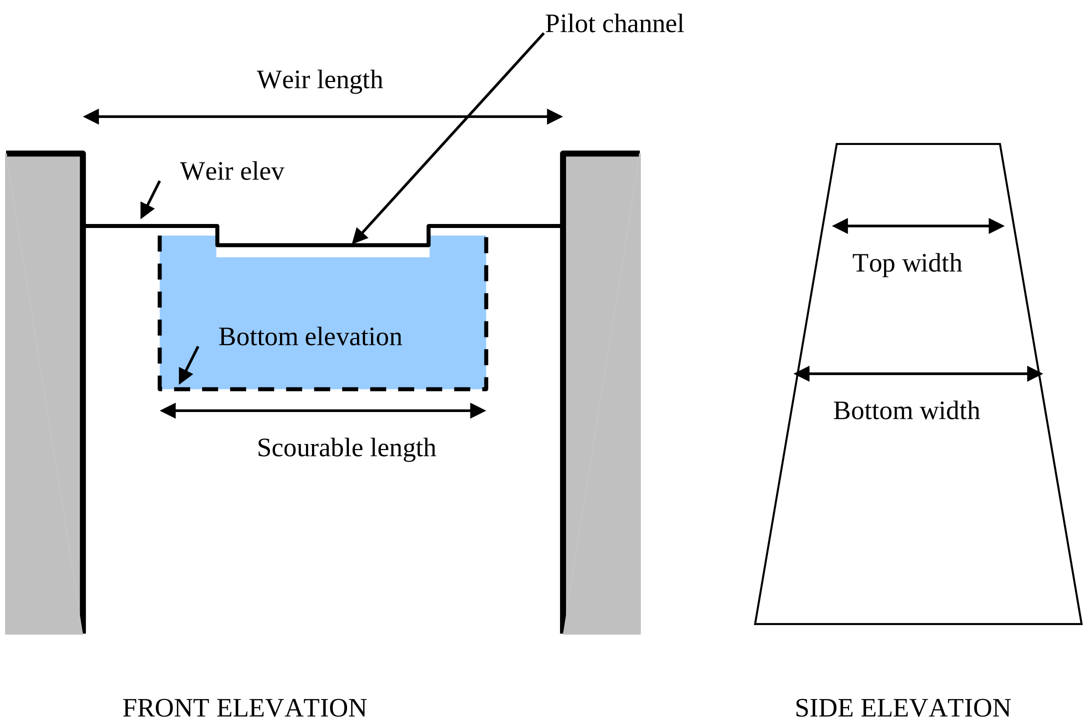


Figure 13. Scourable Spillway

Onsite Detention Storage

This operation is invoked if the subarea is included in the LOCAL STRUCTURES block.

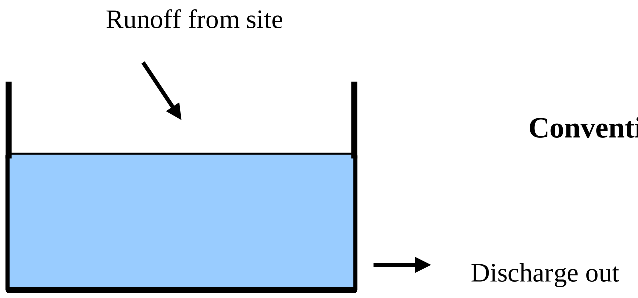
A portion of the local runoff from a subarea can be directed to an onsite detention storage before it joins the flow at the outlet of the subarea. The inflow to the onsite detention can be a percentage of the pervious surface runoff hydrograph, or a percentage of the impervious surface hydrograph, or any combination of the two.

Inflow to the onsite detention storage is routed through the storage using a level pool (Puls) routing procedure. Outflow from the onsite detention is added to the hydrograph at the bottom of the stream channel, together with the portions of the pervious and impervious hydrographs which bypass the onsite storage, to give the total hydrograph at the bottom of the subarea.

Culvert hydraulics and flood routing procedures in this Local Structures block are identical to those in the Outlet Structures block. You can also use a scourable spillway in the Local Structure.

The outflow from the onsite detention storage can be delayed by a specified time before it reaches the bottom of the subarea. When the storage has multiple outlets (box or pipe culverts, weirs etc), the flow from all outlets has the same delay time since they all go to the same point (the bottom of the subarea). This is in contrast with Outlet Structures where the flow from each outlet can go to a different downstream subarea, and consequently have a different delay time. Note that in the runfile, for consistency with the format of Outlet Structures, a time delay is entered for each outlet. However only the first of these values is adopted for the time delay.

A conventional OSD storage consists of a single storage. The maximum discharge released is often called the Permissible Site Discharge PSD, while the maximum volume of water stored is called the Site Storage Requirement SSR.



Conventional OSD Storage

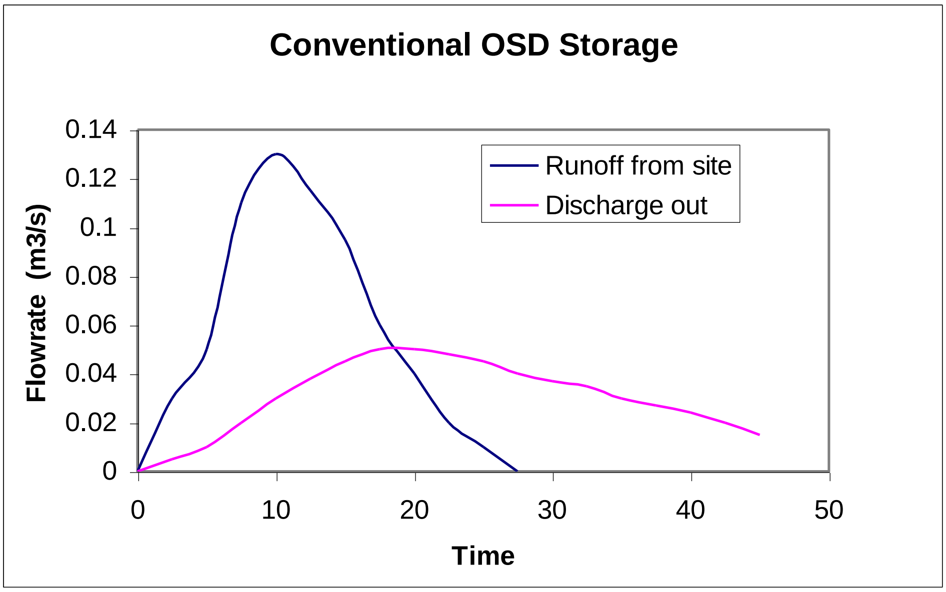


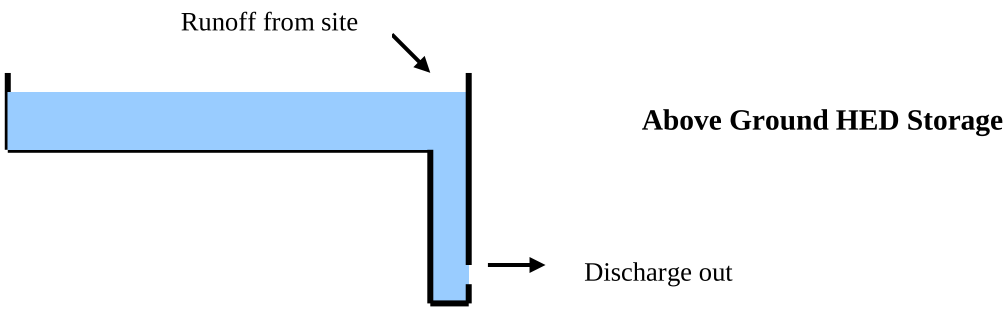
Figure 14. Conventional OSD Storage

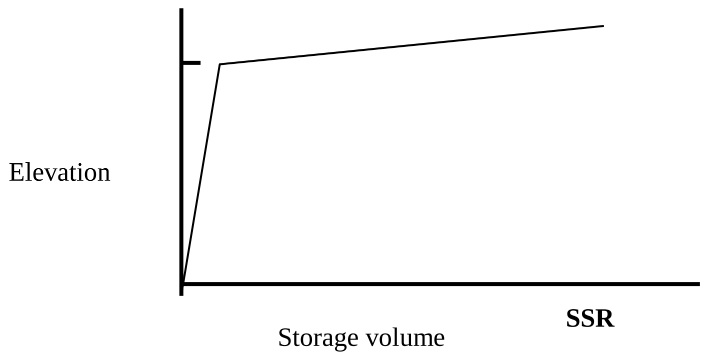
Modelling High Early Discharge Storages

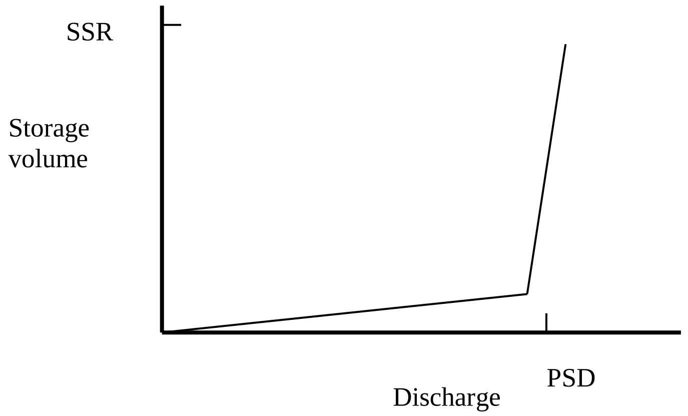
Both Local Structures (Onsite Detention Storages OSD) and Outlet Structures can model High Early Discharge storages. The advantages of HED storages are that the discharge out of the storage can be better controlled at the permissible site discharge PSD value, and the site storage requirement SSR is smaller than in a conventional storage (Boyd, 2001).

HED storages can be of two kinds, above ground and below ground. In both cases, the aim is to allow the outflow discharge to quickly rise to the PSD and to remain at this relatively constant value. This is done by providing a two stage storage. The first storage chamber has a small volume and quickly fills. When it is full, the discharge out of the storage is equal to the design PSD. As this occurs, and the discharge remains at the PSD, the main storage chamber fills. The required volume of the main chamber is equal to the SSR.

For the above ground storage, the first storage rapidly fills and the outflow discharge rises to the PSD. As the main chamber starts to fill, the water level increases only slightly so the discharge remains near to the PSD. At the end of the storm, the main chamber empties slowly, followed by a rapid emptying of the first chamber. Schematics of the structure, elevation-storage relation, and hydrographs are shown in figure 15.







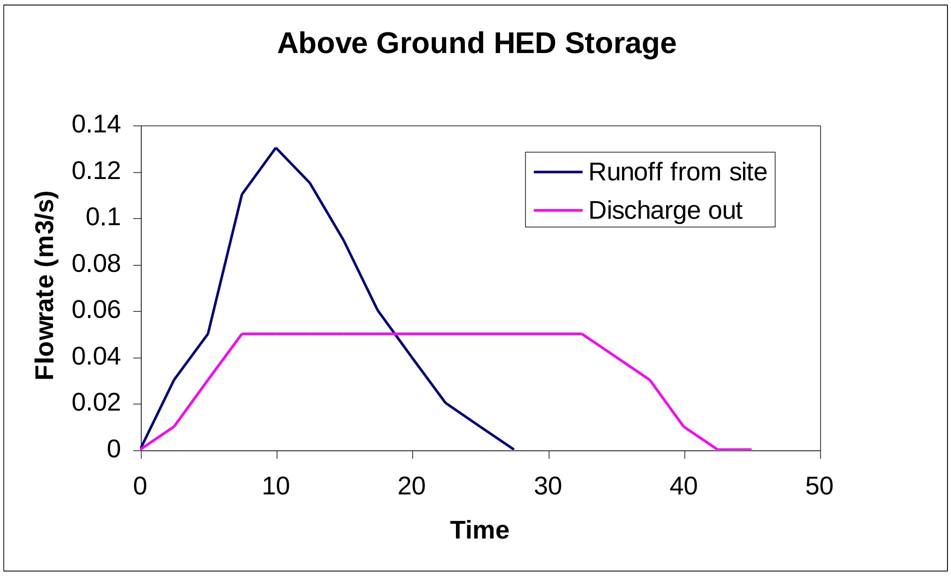


Figure 15. Above Ground High Early Discharge OSD Storage

A below ground HED storage consists of 2 disconnected chambers (figure 16). Runoff from the site flows into the first chamber which has a small volume. This chamber quickly fills, allowing discharges to quickly rise to the PSD value. When the first chamber fills, water flows over a connecting weir into the second, larger chamber. This main chamber takes some time to fill, so that the discharges from the OSD remain at the PSD for some time. When the inflow to the OSD from the site ends, both chambers slowly empty and the discharge from the OSD slowly falls.

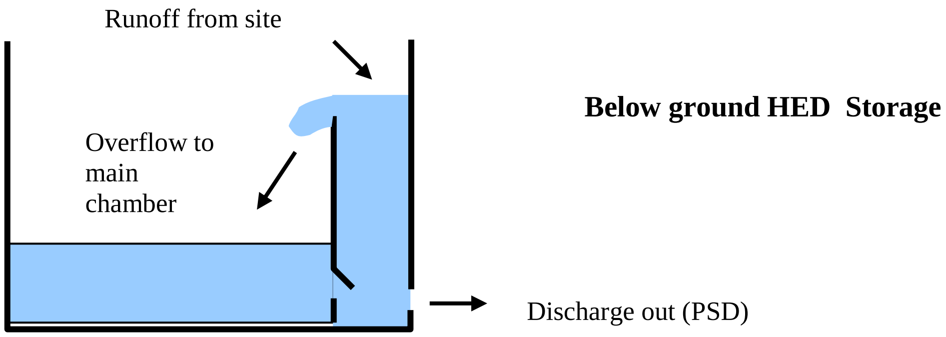


Figure 16.   Below Ground High Early Discharge OSD Storage

Strictly, below ground HED storages can only be modelled by considering the changing water levels in the first and main chambers, since they control flows between the chambers, and also outflows from the storage.
It is possible to approximate this using a looped storage-discharge relation (displaying hysteresis), figure 17. Calculations for the rising hydrograph follow a similar storage-discharge relation to the above ground storage (see above), but follow a different one for the falling hydrograph. This is because both the main and first storage chambers empty together (Boyd, 2001).

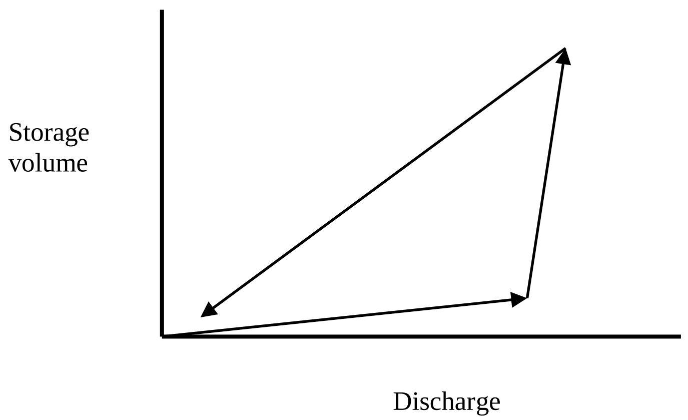


Figure 17.   Below Ground High Early Discharge OSD Storage-Discharge Loop

In reality, the single looped storage-discharge relation shown above only applies if the below ground storage completely fills. If the main chamber only part fills, a set of hysteresis loops must be used, one for each stage of filling (Boyd, 2001).

WBNM adopts above ground routing procedures for both above ground and below ground storages. Therefore WBNM models above ground storages accurately. While WBNM is only approximate for the falling limb of below ground storages, it gives reasonable approximations. Mass balance is always achieved. The following figures 18 and 19 show typical hydrograph shapes.

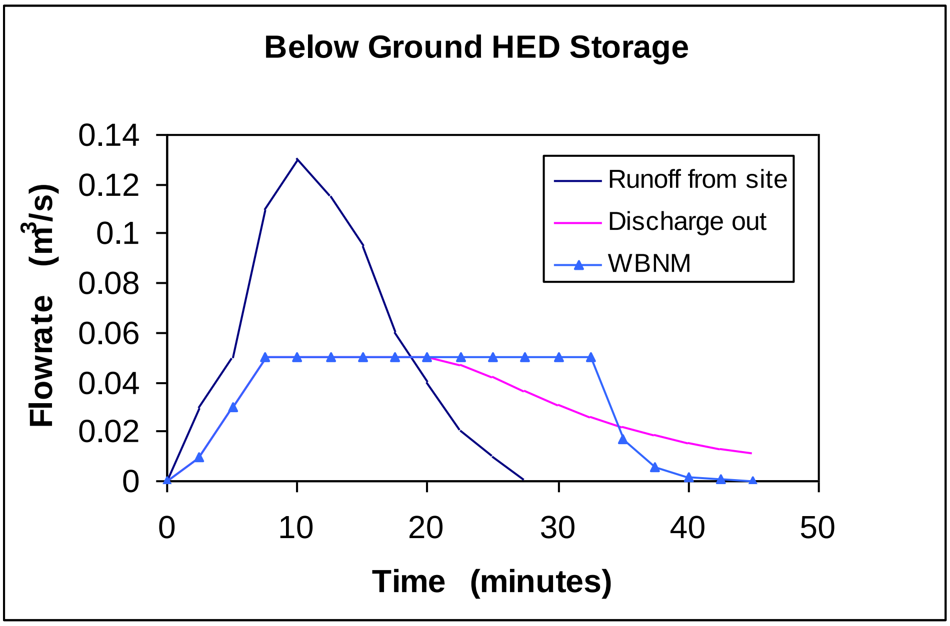


Figure 18. Below Ground HED Storage – Design PSD = 0.05 m^3/s

Note that WBNM calculates the rising and peak discharge of the hydrograph correctly for both above ground and below ground HED storages. Note also that WBNM gives correct values of the maximum discharge (and so of the PSD) and of the storage volume required (the SSR). It is only the shape of the below ground hydrograph recession which is approximate.

If the PSD is larger, say 0.1 m^3/s, (figure 19), the WBNM falling limb is closer to the correct outflow discharge.

Note that when modelling HED storages, because the elevation-discharge relation has an abrupt change in slope, you should use a **small time step for calculations**. You should also ensure that you include the elevation at which water commences flowing from the first to main chamber in the elevation-storage relation.

See figure below.

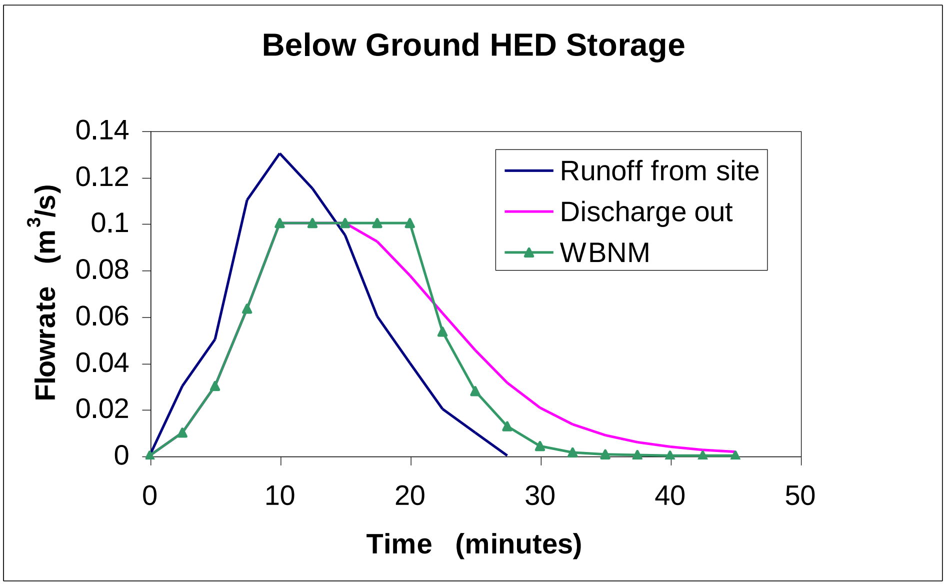


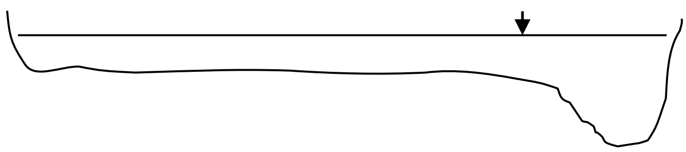
Figure 19.   Below Ground HED Storage – Design PSD = 0.10 m^3/s

Modelling Offstream Storages

Storages in Local and Outlet Structures are normally considered to by “on-stream”, that is, the structure is located in the channel or reservoir, and water is stored upstream of the structure, in the channel or reservoir. Both stored volumes and discharges increase relatively uniformly as water levels increase.

An alternative arrangement is to provide an *offstream storage*, so that flows from the stream are diverted into a storage which is adjacent to the stream, whenever flows in the stream exceed a specified level. Two types of offstream storage could be used:

a) the offstream storage is directly adjacent to the stream and is directly and continuously connected to it. When flows in the stream exceed a specified level, water flows from the stream into the storage. As water levels drop, water flows back into the stream. As a result, the volume of water stored is small until water levels reach the specified level, above which volumes increase rapidly. This type of offstream storage will divert flows above a threshold into the storage, leaving a flattened hydrograph in the main stream. The result is similar to the Above Ground High Early Discharge storage, discussed previously. Figure 20 displays this concept.



Cross Section of Stream and Offstream Storage

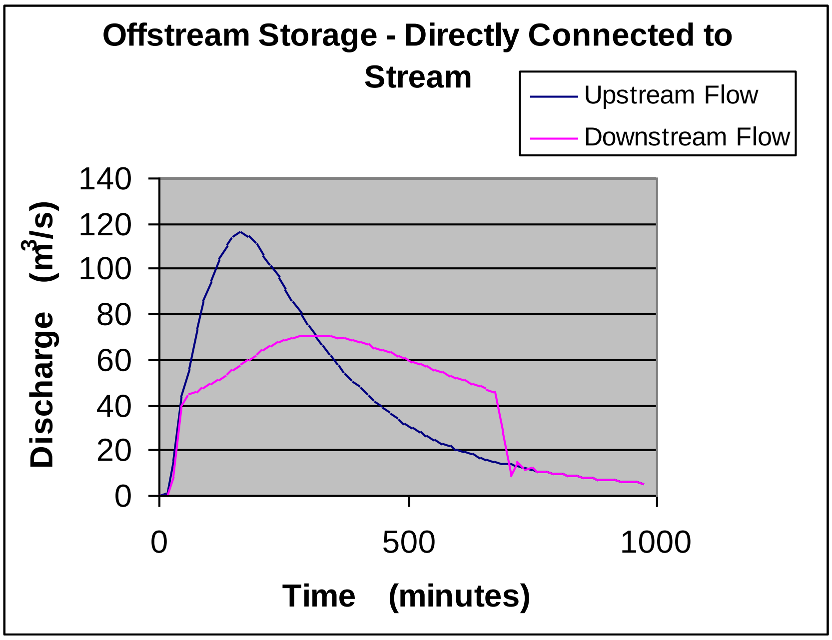
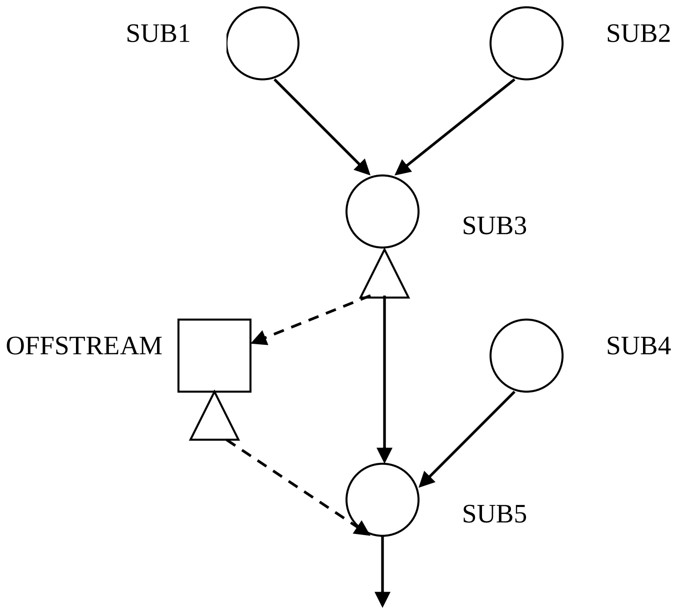


Figure 20. Offstream Storage – Directly Connected to Stream.

Sample runfile *Offstream_storage.wbn* gives an example, and is discussed in more detail in WBNM_Tutorial.pdf. The offstream storage is located at the bottom of the stream channel in SUB3. A HSQ type structure is used to divert flows from the stream into the storage when flows exceed 2.4 m, which corresponds to 45 m^3/s in the stream.

b) the offstream storage is separated from the stream. When flows in the stream exceed a specified level, water flows from the stream into the storage. Water is released from the offstream storage to any nominated downstream point. Culverts and weirs can be used to control the release of stored water back into the stream. The stored water can also be delayed before it is released.

Figure 21 shows a typical arrangement:



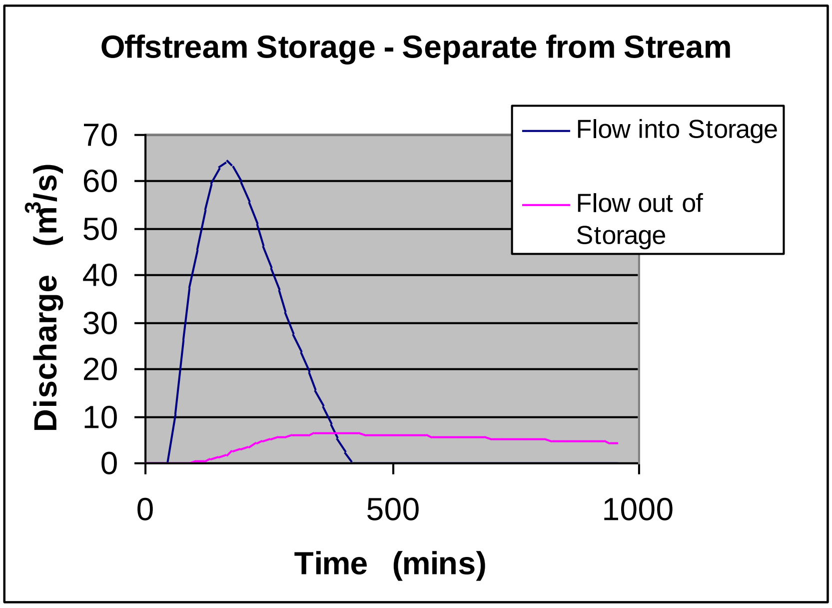


Figure 21. Offstream Storage – Separate from Stream

Sample runfile *Offstream_storage_1.wbn* gives an example, and is discussed in more detail in WBNM_Tutorial.pdf. A separate "subarea" OFFSTREAM is used to represent the offstream storage. An HSQ type structure is used in SUB3 to divert flows from the stream into the offstream storage when flows exceed 2.4 m, which corresponds to 45 m^3/s in the stream. The inflow into the OFFSTREAM storage is the difference between the hydrographs upstream of and downstream of the diversion, which are similar to the hydrographs shown in Figure 20.

Subarea OFFSTREAM has its own Outlet Structure, using culverts and weirs to release the water back into the stream at the bottom of SUB5. There are 5 – 900 mm culverts, and these release flows at a maximum of 6 m^3/s back into the stream, at the bottom of SUB5.

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