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Pumped catchments – worked example of hydrological assessment and hydraulic modelling

Project: SC090006/R3

Flood and Coastal Erosion Risk Management Research and Development Programme

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Miranda Kavanagh

Director of Evidence

Executive summary

This document provides an addendum to the updated '*Pumped catchments – Guide for hydrology and hydraulics*' (Report SC090006/R003, 2012). As part of the guidance update, it was decided that practitioners in the operating authorities responsible for flood risk management in low lying catchments would benefit from being able to view illustrative examples. This document presents a worked example which compares results for assessing flood risk derived from the new method to that obtained by applying the processes set out in the earlier guidance (*The Hydraulics and Hydrology of Pumped Drainage Systems – An Engineering Guide,* Report SR 331 November 1993).

In addition to presenting worked examples within the guidance document, a series of presentations were also given via webinars:

- Webinar 1 – the structure of the guide, how it was developed and an overview of the available methods for hydrological and hydraulic analysis within pumped catchments.
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1 Introduction

1.1 Background

This document provides an addendum to the updated '*Pumped catchments – Guide for hydrology and hydraulics*' (Report SC090006/R003, 2012). As part of the guidance update, it was decided that practitioners in the operating authorities responsible for flood risk management in low lying catchments would benefit from being able to view illustrative examples. This document presents a worked example which compares results for assessing flood risk derived from the new method to that obtained by applying the processes set out in the earlier guidance (*The Hydraulics and Hydrology of Pumped Drainage Systems – An Engineering Guide*, Report SR 331 November 1993).

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1.2 Worked example – study area

The example chosen to illustrate this comparison is based on analysis of North Level Internal Drainage Board's (IDB) Tydd pumped catchment spanning across the Lincolnshire/Cambridgeshire border, the location of which can be seen in Figure 1. A comparison is made between the different methods of hydrological analysis that are available. Hydraulic modelling has been undertaken in conjunction with hydrology established using the new (2012) guidance only; although a comparison of potential methods is discussed for such activities.

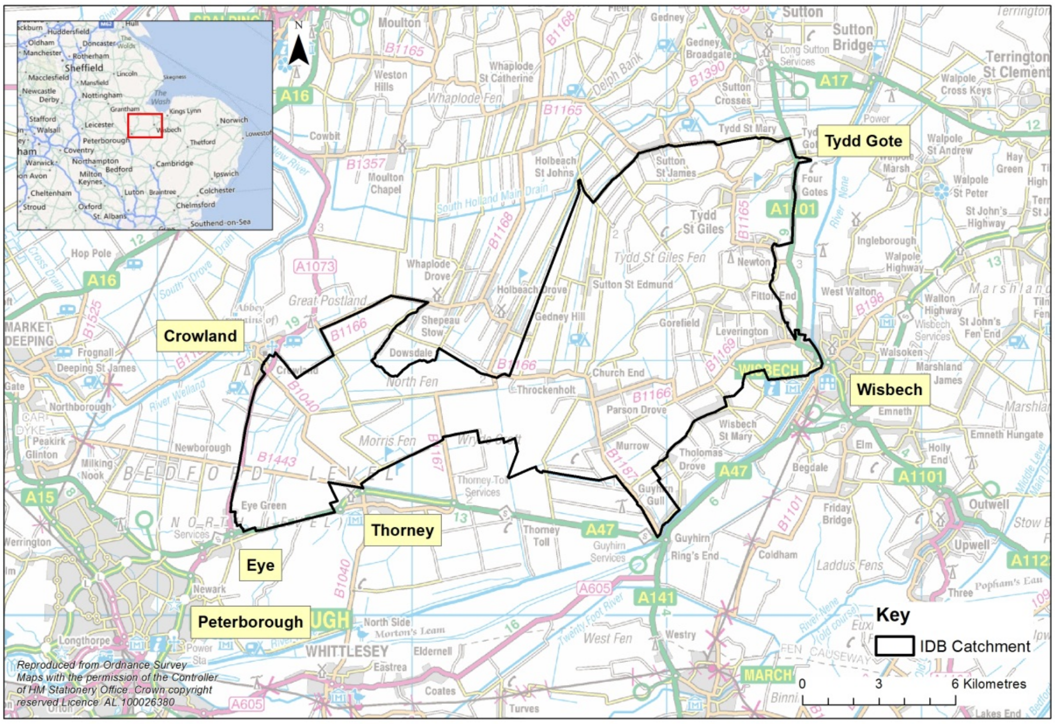

Figure 1: Location plan

1.3 Step 1 – Define the catchment (common to both examples)

The Tydd catchment, see Figure 2, is defined from North Level IDB mapping of their drainage system. The catchment is self-contained, with no drainage linkages to adjacent areas. The total area is 163.9km^2 and the total length of IDB maintained watercourse is 317km. The catchment drains to Tydd Gote pumping station where discharge is to the tidal River Nene downstream of Wisbech. There are also four internal pumping stations within the catchment. Station characteristics are listed in Table 1.

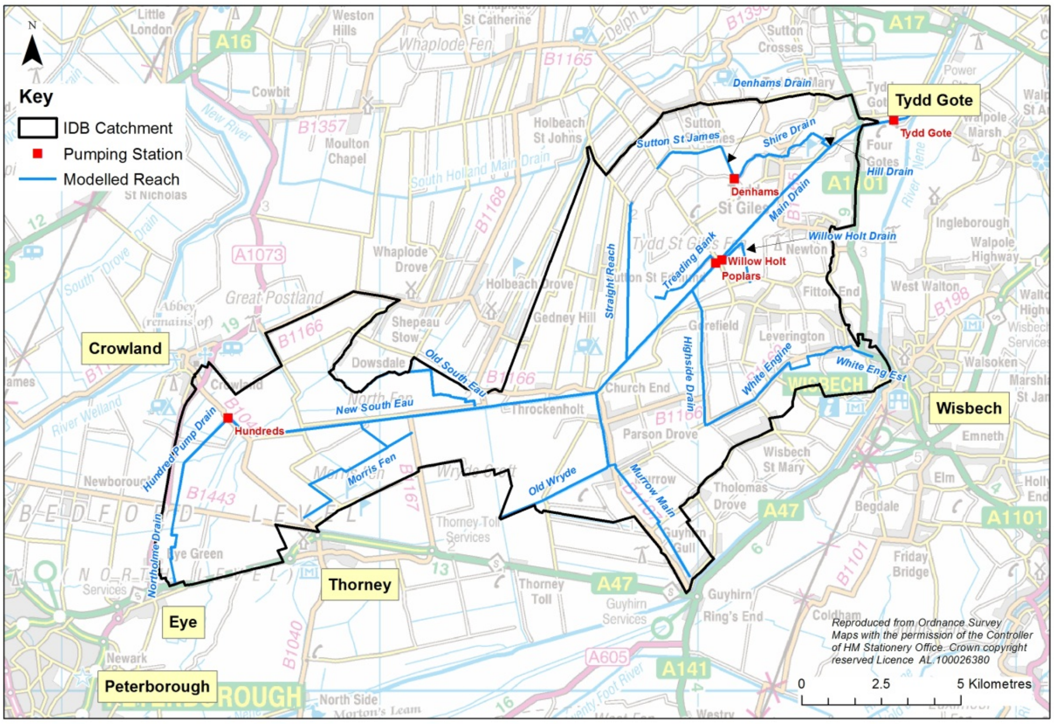

Figure 2: Catchment definition

Table 1: Pumping stations in the Tydd catchment

Station	Number of pumps	Pump capacity (cumecs)	Pumping station primary use	Total station capacity (cumecs)	Receiving watercourse
Tydd Gote	6	3.36	Lifting water into high level carrier	20.16	Tidal River Nene
Denhams	1	1.05	Lifting water into high level carrier	1.05	Shire Drain
Willow Holt	2	0.41	Protecting agricultural land and 120 properties	0.82	Main Drain
Poplars	2	0.8	Protecting agricultural land and 150 properties	1.6	Main Drain
Hundreds	2	0.75	Drainage of agricultural land	1.5	South Eau

2 Step 2 – Hydrology

2.1.1 Step 2.1 – Catchment characteristics

New (2012) method	Old (1993) method
Catchment characteristics are found from the FEH CD_ROM Version 3.0. The automated process does not delineate the catchment properly, so characteristics are found in three parts: the upper reach, the middle reach and the downstream reach. See Figure 3a to 3c. The overall catchment characteristics are mostly taken as the mean from the three parts. Exceptions are values for AREA, DPLBAR and DPSBAR which are estimated from IDB catchment data, actual drainage route lengths from IDB survey and (for DPSBAR) also using ground levels from LiDAR survey. Some resultant characteristics are given in Table 2.	The characteristic of interest is the Winter Rain Acceptance Potential Class, seen from the FSR WRAP mapping. The whole catchment has soil of Class 2, 'High'.

Table 2: Tydd Catchment characteristics from FEH CD-ROM

Final Catchment characteristic	Value
AREA	163.9km^2
BFIHOST	0.70
DPSBAR	0.03 m/Km
FARL	1.0
SAAR	551mm
SPRHOST	26.5
URBEXT1990	0.01

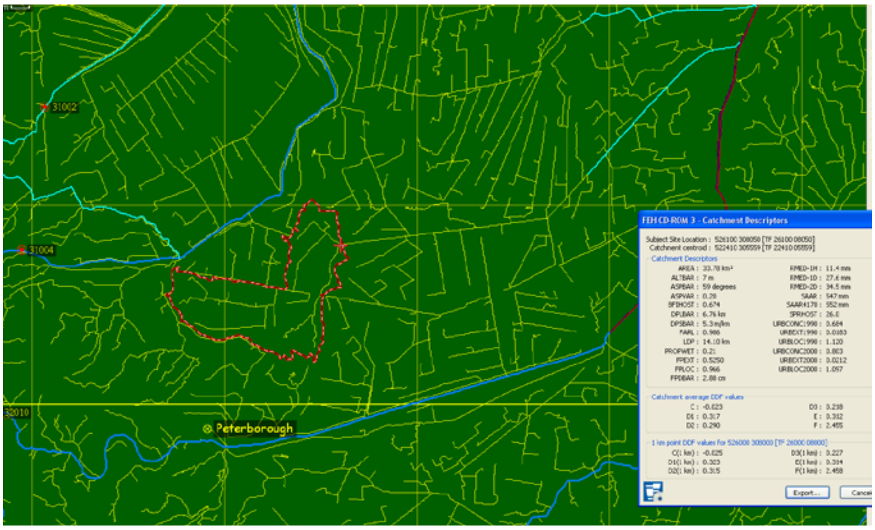

Figure 3a: Catchment characteristics of the upstream reach as defined by FEH CD ROM V3.0

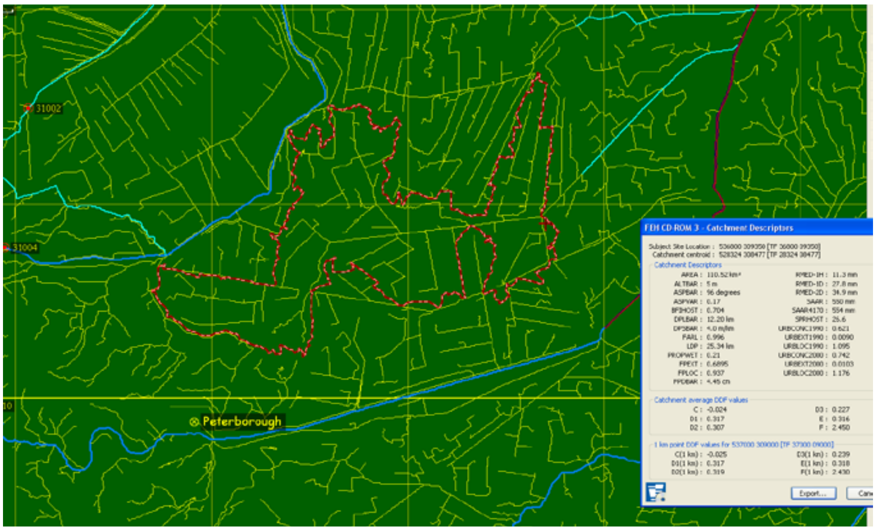

Figure 3b: Catchment characteristics of the middle as defined by FEH CD ROM V3.0

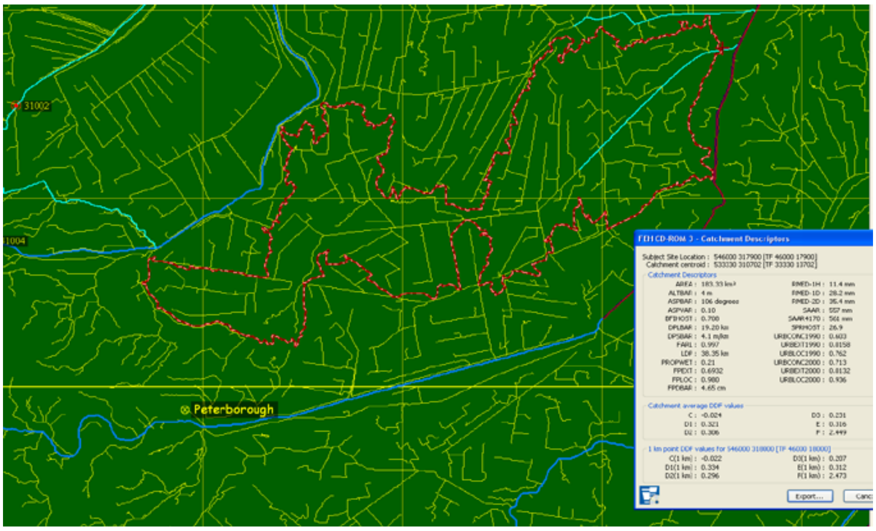

Figure 3c: Catchment characteristics of the downstream as defined by FEH CD ROM V3.0

2.2 Step 2.2 – Time to peak (Tp)

New (2012) method	Old (1993) method
The method (4.3.6) suggests taking a default value of Tp = 24 hours in the absence of local data.	The method (2.2) similarly suggests taking a default value of Tp = 24 hours in the absence of local data.
In this instance it is also possible, as suggested (4.3.5), to estimate Tp from event analysis. The only significant event for which there are coincident rainfall and pumping records is the event of April 1998.	It also suggests estimating Tp from historic event data, but without elaborating on how this should be done. Here we assume the process used in the new (2012) method is used to find the event LAG.
The rainfall pattern record from Crowland is used, being the nearest site with a 15-minute record, together with the pump record from Tydd Gote. The Crowland rainfall pattern and total is supported by a very similar record from Whittlesey. The total is also supported by the daily record from Thorney.	The catchment Tp(0) is estimated using the FSSR 16 formula: Tp(0) = 0.604 LAG 1.1444 Thus Tp(0) becomes 32.4 hours. This would be rounded to 36 hours.
The rainfall hyetograph is shown in Figure 4. The calculated centroid of the rainfall is 19.5 hours.	In the final design run, 24 hours has been used for comparison between the two methods not 36 hours, to allow a direct comparison between the results of the two methods.
The pump record is converted to an approximate hydrograph using the method given in Text Box 4.2 of the guidance, see Figure 5. The approximate peak is at time 52 hours.	
Comparison to the rainfall record gives the event LAG, found to be 32.5 hours.	It is important to stress however that the two methods would give a significantly different Tp(0).
The catchment Tp is estimated by FEH Vol 4 equation 2.9: Tp(0) = 0.879 LAG 0.951	
Thus Tp(0) becomes 24.1 hours.	
Since this is close to the standard value of 24 hours, Tp = 24 hours will be used as the initial estimate in this assessment.	

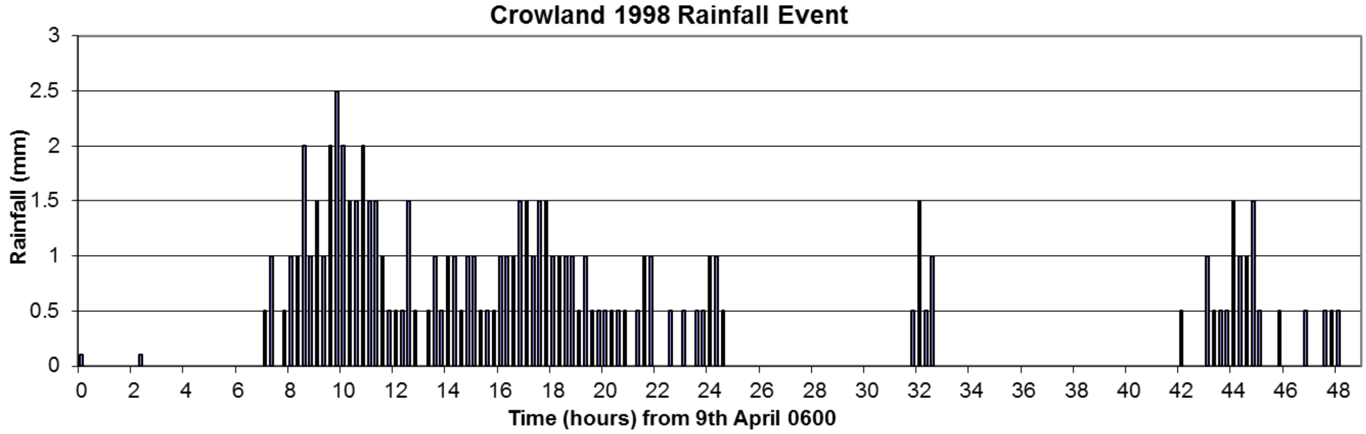

Figure 4: April 1998 rainfall at Crowland

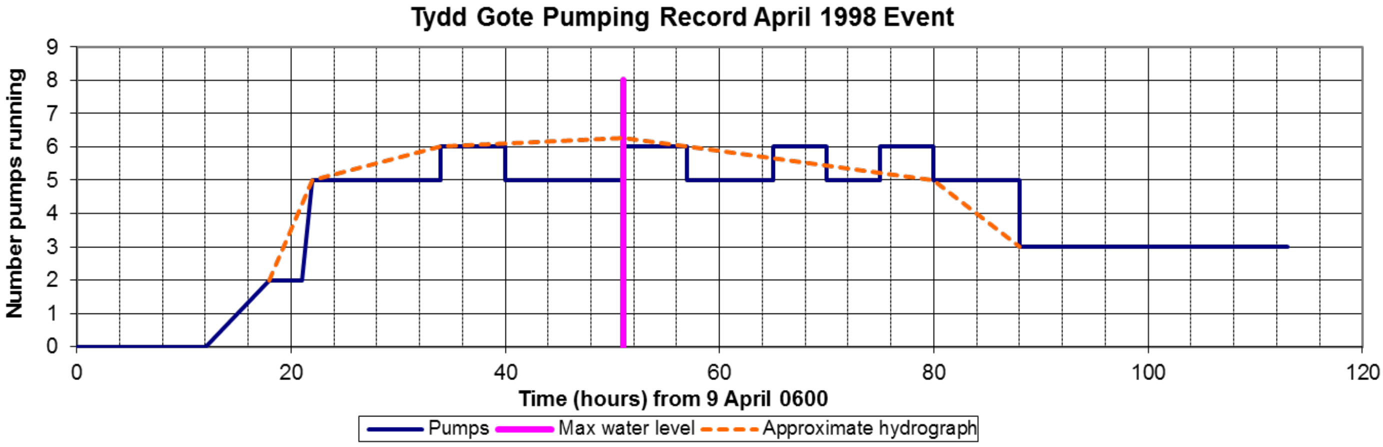

Figure 5: April 1998 event approximate pumped flow hydrograph

2.3 Step 2.3 – Catchment unit hydrograph

New (2012) method	Old (1993) method
The lowland unit hydrograph in Figure 4.2 of the guide has been reproduced using data from the catchment analysis. With Tp = 24 hours, the unit hydrograph becomes as seen in Figure 6. The peak flow rate is calculated using equation in 4.3.6 of the guide: Qp = 1.5873 AREA / Tp Thus Qp becomes 10.8 cumecs per 10mm net rain. The modelling uses units of rain in millimetres, so we take the peak flow rate as 1.08 cumecs per mm net rain. The Tp and unit hydrograph derived as above are carried forward to the design flood hydrograph of this example. In reality the calibration process for the modelling (described later) may lead to revision of Tp and hence revision of the unit hydrograph. Those iterative steps are omitted here for brevity.	The same form of unit hydrograph applies (Figure 2.1 of the guide). With Tp = 36 hours the unit hydrograph becomes as seen in Figure 6. The equation for Qp is the same as for the new (2012) method so the peak flow rate becomes 0.64 cumecs per mm net rain.

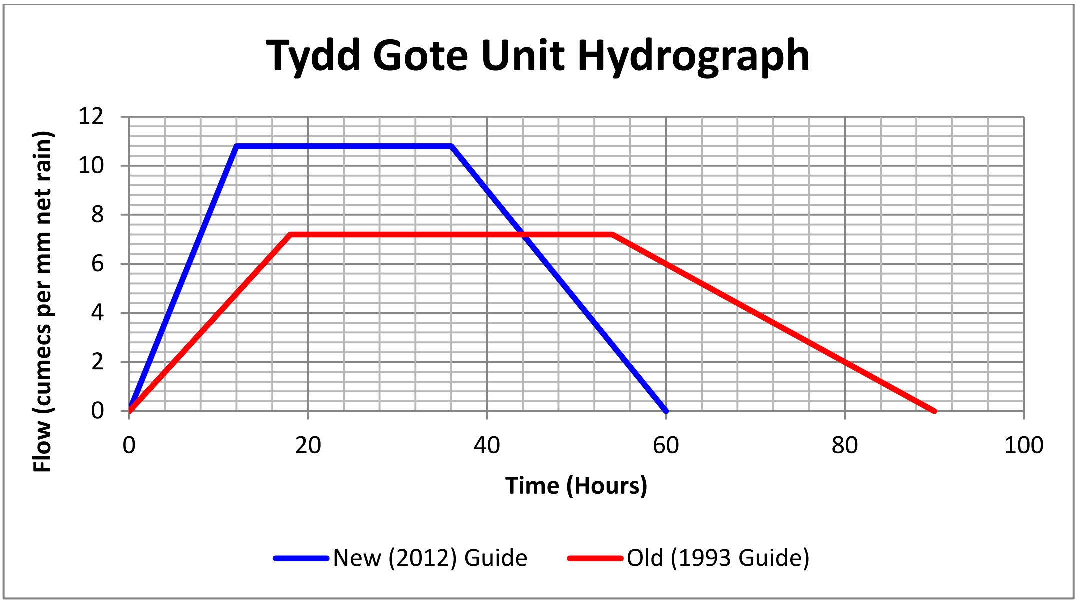

Figure 6: Unit hydrograph at Tydd Gote pumping station

The unit hydrograph is for a six-hour block of rain. However, it can be used without modification for shorter rainfall data intervals that are normally used in hydrodynamic modelling. If an ISIS Flood Estimation Handbook (FEH) boundary unit is to be used to generate flows, it is important that the data interval for the unit hydrograph ordinates matches the time period of any rainfall data that may be used (primarily for generating calibration flows).

2.4 Step 2.4 – Sub-catchments

New (2012) method	Old (1993) method
In order to improve the level of detail obtained from the modelling (guide 4.3.9), the catchment runoff will be distributed as inflows from 22 sub-catchments. The sub-catchments are shown in Figure 7. A unit hydrograph for each sub-catchment is derived from the 'whole catchment' unit hydrograph. Each sub-catchment unit hydrograph is derived by factoring Qp pro rata by the respective area of the sub-catchment to the size of the 'whole catchment'. It should be noted that the sub-catchment unit hydrographs retain the same timescale; there is no change to Tp. Thus for a sub-catchment with AREA 7.0km^2, the Qp value in the unit hydrograph becomes (7.0/163.9) * 1.08= 0.046 cumecs per mm net rain.	This method acknowledges that the lowland trapezoidal unit hydrograph is only applicable at the pumping station outfall. No guidance is given on generating sub-catchment inflows. In this example, only the 'whole catchment' inflow is calculated. The inflow could be divided to each sub-catchment based on their area relative to the total catchment area. In practice, this is the same as adopted for the new (2012) method example.

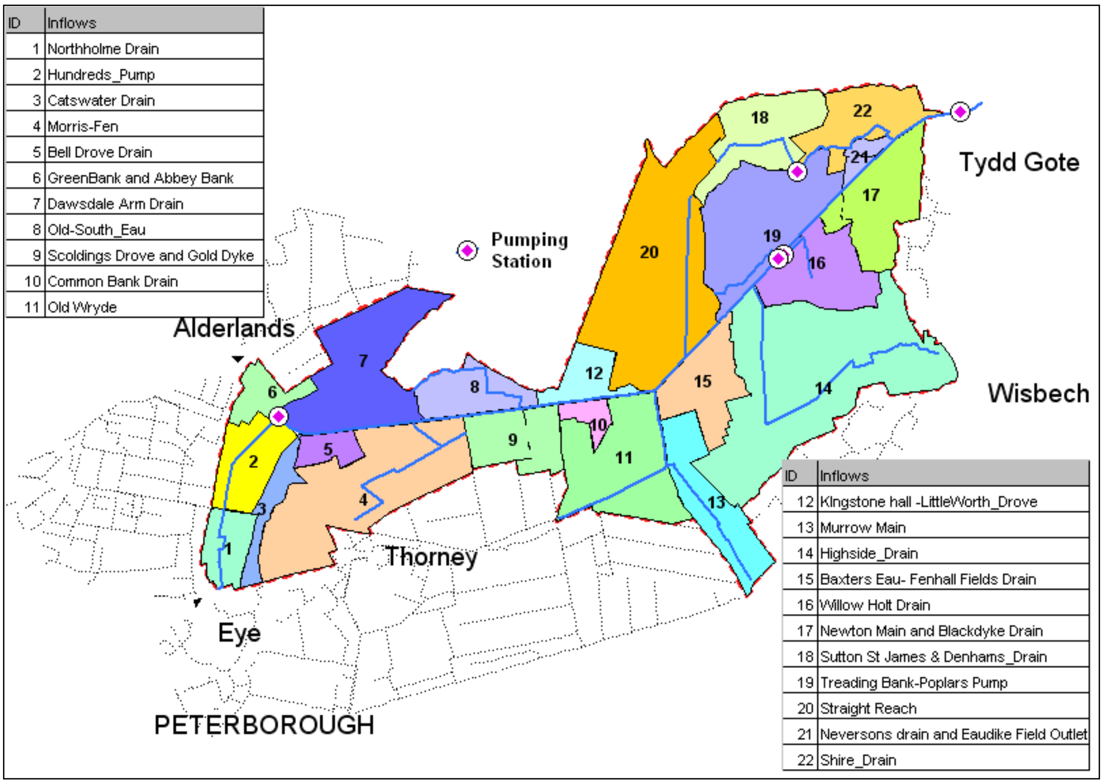

Figure 7: Tydd sub-catchment schematisation

2.5 Step 2.5 – Percentage runoff

New (2012) method	**Old (1993) method**
The method (guide, 4.3.7) recommends estimating SPR by back-calculation from event data. The same process will also give the overall percentage runoff (PR) of the event.	The method (2.2) advises estimation of PR using the procedure described in FSSR 16 (part 3.2 applies).
We have data from the April 1998 event where: Total rainfall is 86 mm.	The catchment all has WRAP Soil Class 2, so SPR = 30.
Total pump run hours (from record) is 416. Over the catchment area of 163.9km^2, the rainfall volume becomes 14.1 million m^3.	The catchment average annual rainfall is 551mm. From FSR Figure I.6.62, the nominal catchment wetness index, CWI = 77.
Since each pump has nominal discharge capacity 3.36 cumecs, the pump run time suggests the total discharged volume is 5.0 million m^3.	From the equation DPR CWI = 0.25(CWI-125), DPR CWI = minus 12.0
Thus the apparent PR is 35.5 %.	The event rainfall total (P) is 86 mm. From the equation DPR RAIN = 0.45(P – 40) 0.7 DPR RAIN = 6.6
The Soil Moisture Deficit (SMD) at the time of the April 1998 event is not known exactly. However, the engineering report on the event noted that the catchment had been saturated in advance of the event rainfall therefore a value SMD = 0 is assumed.	Combining, PR RURAL = 24.6 No urban adjustment was made due to the very low percentage within the catchment.
Back calculation of SPR is made using the following equations from FEH Vol 4:	On this basis PR = 24.6 % for the April 1998 event.
The five-day antecedent rainfall index (API5) in the event is 3mm. Taking SMD = 0 and using FEH equation A.1, CWI = 125 + API5 - SMD = 128	From the parallel example we can see that taking the nominal CWI value is not realistic for the April 1998 event.
Taking equation 2.14, DPR CWI = 0.25(CWI -125) = 0.75 The event rainfall (P) is 86 mm. taking equation	As a reasonable approximation we will take CWI = 125 to represent the antecedent wetness likely to be needed to give a significant flood.

2.15, DPR RAIN = 0.45(P -40) 0.7 = 6.6 Equation 2.13 states PR RURAL = SPR + DPR CWI + DPR RAIN No urban adjustment is needed for this catchment so SPR = PR – DPR CWI – DPR RAIN Thus we estimate SPR = 28.15 % We will round this to SPR = 28%. This is higher than the SPRHOST value given by the FEH CD-ROM data and presented at Step 2.1. The values of SPR and DPR CWI will be applied in design run modelling with DPR RAIN varying with the design event rainfall.	Revisiting the above equations, SPR = 30 DPR CWI = 0 DPR RAIN = 6.6 So PR = 36.6 % is an estimate for the April 1998 event.

2.6 Step 2.6 – Baseflow

New (2012) method	**Old (1993) method**
The method advises baseflow is derived using gauged data if it exists. However there was no such data for the Tydd catchment. The method advises (Figure 4.1 of the guide) estimating baseflow through the standard FEH equations. Experience in pumped catchments in east Lincolnshire suggests this method is likely to underestimate the baseflow. However, in the absence of a more definitive expression, the FEH method is used here. Taking FEH Vol 4 equation 2.19: BF = [33(CWI – 125) + 3.0 SAAR + 5.5] * 10 -5 *AREA From Step 2.1, SAAR = 551mm and AREA = 163.9km^2. From Step 2.5, and noting (under the parallel example) that a CWI based on SAAR is likely to underestimate flood flows, we take CWI = 128 as a reasonable value. Hence the baseflow at Tydd Gote is estimated as 2.9 cumecs. This is equivalent to 0.176 cumecs per 10km^2.	The method (2.2. of the guide) advises estimating baseflow through the FSR equation. From FSSR 16 part 3.3, the baseflow equation is the same as now included in the FEH. So, taking CWI = 125 as above and SAAR = 551mm, the baseflow (ANSF) = 0.166 cumecs per 10km^2. Thus the baseflow at Tydd Gote pumping station becomes 2.7 cumecs. This is equivalent to 0.165 cumecs per 10km^2.

Step 2.7 – Design storm duration

<table>
<tr><th>New (2012) method</th><th>Old (1993) method</th></tr>
<tr><td>The guide advises (4.3.8) first using the standard FEH equation to determine the design storm duration from Tp and SAAR.

Taking FEH Vol 4 equation 3.1:

Duration (D) = Tp (1 + SAAR/1000)

A value Tp = 24 hours is estimated at Step 2.2.

On this basis the initial design storm duration, D = 37.2 hours.

The guide (4.3.8) then advises following the process as in FEH Vol 4 Section 9.2.2 to find an improved critical duration for the catchment. In effect the FEH method equates to testing a range of storm durations, based around the first approximation as above, to find the worst case flood condition for each part of the system.

Most river modelling software readily permits running a suite of conditions each with different storm durations. There will not necessarily be a single critical duration applicable to all parts.

The finding from this example, through calibration and sensitivity testing, is that a critical duration of D = 42 hours gave the highest water levels throughout. Coincidentally, this duration is very similar to that of the April 1998 event.</td><td>The recommended design storm duration is obtained from FSR equation I.6.46.

D = (1.0 + SAAR/1000) * Tp

With SAAR = 551mm and Tp = 24 hours,

D = 37.2 hours.

For comparison purposes as previously stated in Step 2.2, a Tp of 24 hours has been used (rather than 36 hours) in the comparison of the two methods. Using Tp of 36 would give D = 56.</td></tr>
</table>

2.7 Step 2.8 – Design rainfall

New (2012) method	Old (1993) method
Design rainfall is taken from the FEH CD-ROM. For this example we will consider the 1 in 50 years flood event (2% annual probability event). From FEH Vol 4 Table 3.1, the recommended rainfall return period for a 1 in 50 years flood is 81 years. From the FEH CD-ROM, the 1 in 81 years point rainfall for a storm duration of 42 hours is 98.8mm. Thus the design rainfall is 98.8mm. The rainfall is taken as applying uniformly across the catchment. It is taken to be distributed over time in accordance with the 75% winter profile, since winter and spring are known to be the flood seasons. Rainfall depths for different return periods and storm durations can be automatically calculated using ISIS FEH boundary units populated with FEH catchment characteristics.	The rainfall information given in the FEH CD-ROM is much improved from that expressed through FSR procedures and available in 1993, the date of the old guide. Therefore rainfall totals will be taken from the FEH CD-ROM for this example. This will give a truer comparison between the example results since any differences will be attributable to process rather than the significant parameter of rainfall total. The rainfall is taken as applying uniformly across the catchment. It is taken to be distributed over time in accordance with the 75% winter profile.

2.8 Step 2.9 – Design hydrograph at Tydd Gote

New (2012) method	Old (1993) method
Convolution of the design rainfall (Step 2.7) with the Tydd Gote unit hydrograph (Step 2.2), combined with the baseflow (Step 2.6), gives the flow hydrograph seen in Figure 8. This is for the 1 in 50 years flood: Tp = 24 D = 37 hours SPR = 28 % CWI = 128 Baseflow = 2.9 Cumecs The calculation is made using an FEH boundary unit in the ISIS software, taking a time step of 0.25 hours.	Convolution of the design rainfall (Step 2.7) with the Tydd Gote unit hydrograph (Step 2.2), combined with the baseflow (Step 2.6), gives the flow hydrograph seen in Figure 8. This is for the 1 in 50 years flood with: Tp = 24 D = 37 hours SPR = 30 % CWI = 125 Baseflow = 2.7 Cumecs The calculation is made using an FEH boundary unit in the ISIS software, taking a time step of 0.25 hours. Figure 9 shows a comparison of the design hydrographs if Tp of 36 had been used (Step 2.2) in the old (1993) method for illustrative purposes of 'first' or 'best guess' estimation of the design hydrograph.

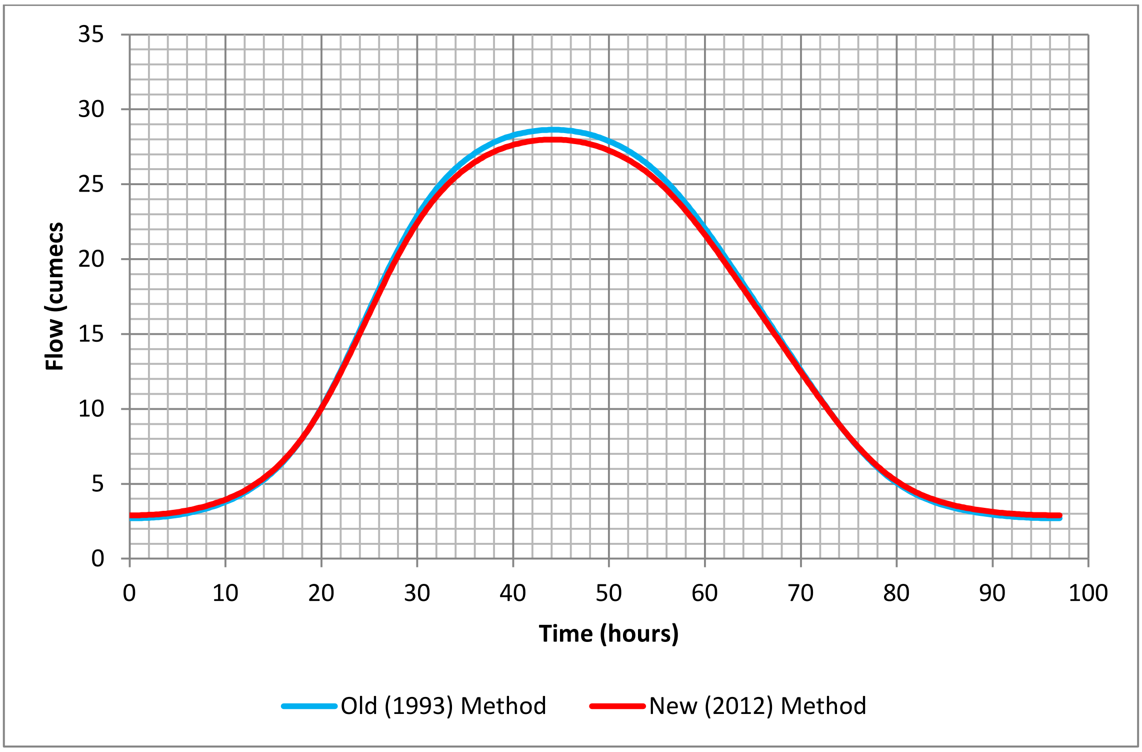

Figure 8: Flow hydrograph at Tydd Gote for 1 in 50 year flood

(both using Tp = 24)

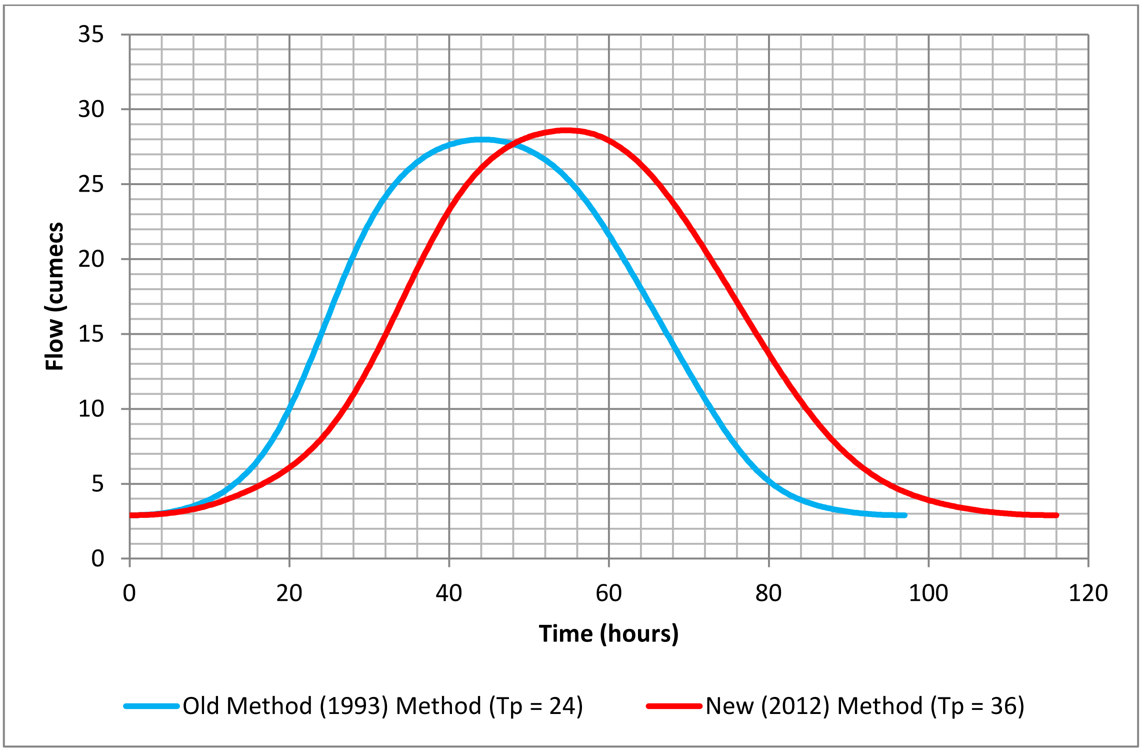

Figure 9: Flow hydrograph at Tydd Gote for 1 in 50 year flood

(using Tp = 24 (new) and Tp = 36 (old))

Figure 8 shows a comparison of 'first attempt' design hydrographs where Tp = 24 for both methods giving similar results. Figure 9 shows that using a Tp of 36 for the old (1993) method (which also modifies D to become 56) gives more contrasting design hydrograph results. In practice, however, the user would not apply these design events alone for their study; a range of design durations would be considered in order to determine the worst case scenarios for all locations within the study reach as explained in Step 2.7.

3 Step 3 – Hydraulics

3.1 Step 3.1 – Modelling type

New (2012) method	**Old (1993) method**
Hydrodynamic modelling is used for this study as it is necessary to produce information on the attenuation of runoff through storage and on the timing of pump runs (guide, 5.2.3). Floodplain flows and volumes are not expected to be significant to the results, therefore 1D modelling of the channels only is adopted (guide, 5.2.4). The modelling software HEC-RAS is used, therefore afflux at structures is calculated automatically by the model (ref guide 5.2.5).	Full dynamic modelling is deemed necessary as it is desired to track the performance of the drainage system under both historic and hypothetical design inflow conditions (guide, 2.6.2), however for this example no modelling had been undertaken.

3.2 Step 3.2 – Model construction

<table>
<tr><th></th><th>New (2012) method</th><th>Old (1993) method</th></tr>
<tr><td>(i)</td><td colspan="2">The dimensions and levels of drains and structures will be taken from up-to-date surveys.</td></tr>
<tr><td>(ii)</td><td colspan="2">Pump operation settings, such as cut-in and cut-out water levels, will be as advised by the IDB as being appropriate for the winter flood season.</td></tr>
<tr><td>(iii)</td><td>Channel roughness will be represented using Manning’s n parameter. Here the respective n value for different sizes of drain are estimated using Cowan’s method (not mentioned in the guidance but it is a suitable robust method) and checked using the methods outlined in the guidance also outlined in the MAFF-funded Newborough Fen research. Other methods of estimation are available, such as the Conveyance Estimation System’s Roughness Advisor, although the method of estimation used should be congruent with the level and cost of the study to be undertaken

An illustrative calculation is given as Figure 10.

Initially adopted values of Manning’s n are:
• Larger drains (Main Drain and South Eau) 0.035.
• Smaller drains 0.06.

For n values given by the broad-brush approach noted in 5.3 of the guide see the parallel example.</td><td>Channel roughness will be represented using Manning’s n parameter.

The guide (2.4) recommends adopting n = 0.04 where the hydraulic radius (R) is at least 1.0m.

A value of n = 0.04/(R) 0.5 is recommended where R is less than 1.0m.

In this catchment the hydraulic radius will be at least 1.0m, other than a few of the smaller drains.

For the latter, the hydraulic radius varies from 0.4 to 0.5m through the event. This suggests a value of n = 0.06.

Therefore a value n = 0.04 is adopted generally but n = 0.06 where appropriate.</td></tr>
<tr><td>(iv)</td><td colspan="2">Sensitivity testing could be carried out for a summer condition, with respective pump operation settings and higher drain roughness.</td></tr>
</table>

Smaller Drain Photos	Drain	Manning’s n calculation

	Names	**n=(n0+n1+n2+n3+n4)m5 (Open channel hydraulics by Ven Te Chow)**		
	Denhams Drain	Material involved	Earth	N0=.020
		Degree of irregularity	Smooth	N1=00
		Variation of channel Cross section	Gradual	N2=00
		Relative effect of obstruction	Negligible	N3=00
		Vegetation	high	N4=.04
		Degree of meandering	Minor	M5=1.0
		N=0.06 Used Manning's n =0.06 for all small drains		
	Northolme Drain			
	Willowholt			

Figure 10: Estimation of Manning's n for smaller drain in winter using Cowan's method

3.3 Step 3.3 – Calibration

<table>
<tr><th>New (2012) method</th><th>Old (1993) method</th></tr>
<tr><td>Calibration of parameters is strongly recommended (guide 4.2, 4.3.2) as well as being good modelling practice.</td><td>The guidance on this method does not discuss model calibration, other than through possible adjustment of the drain sizes (stock model calibration) to achieve the best representation of available storage (guide 2.6.3).

For this example no calibration has been undertaken.</td></tr>
<tr><td>Calibration is only possible for records from the April 1998 flood event due to a lack of complete or matching data sets for the various rainfall and pump/gauge records.

The water level record for comparison to modelled levels is not extensive; flow, flood extent and other parameters can be included in the calibration.

The calibration testing for the Tydd catchment found results were improved by:
Increasing Manning’s value to 0.05 for the Main Drain and New South Eau (the major channels).
Increasing the Tp to 36 hours.
Increasing the SPR of two sub-catchments by 7%
Calibration is an iterative process, initially focusing on aspects of greatest discrepancy to the water level record for this study.

Figures 12 to 14 show the results of the model calibration with varying degrees of agreement between recorded and modelled water levels.

Disparities between these records have been attributed to:
Lack of sufficient pump information during event.
Over simplification of rainfall for entire catchment.
Only one significant event with sufficient amount of data to undertake calibration.
Vertical glass walled model may skew results for out of bank model reaches.</td><td>Calibration practices that have been adopted (see under new (2012) method) could also be used to calibrate a model utilising hydrology derived using the ‘old’ hydrological techniques. Although some of the input parameters would be different both methods would try and match modelled and recorded peaks, timings and hydrograph shapes.</td></tr>
</table>

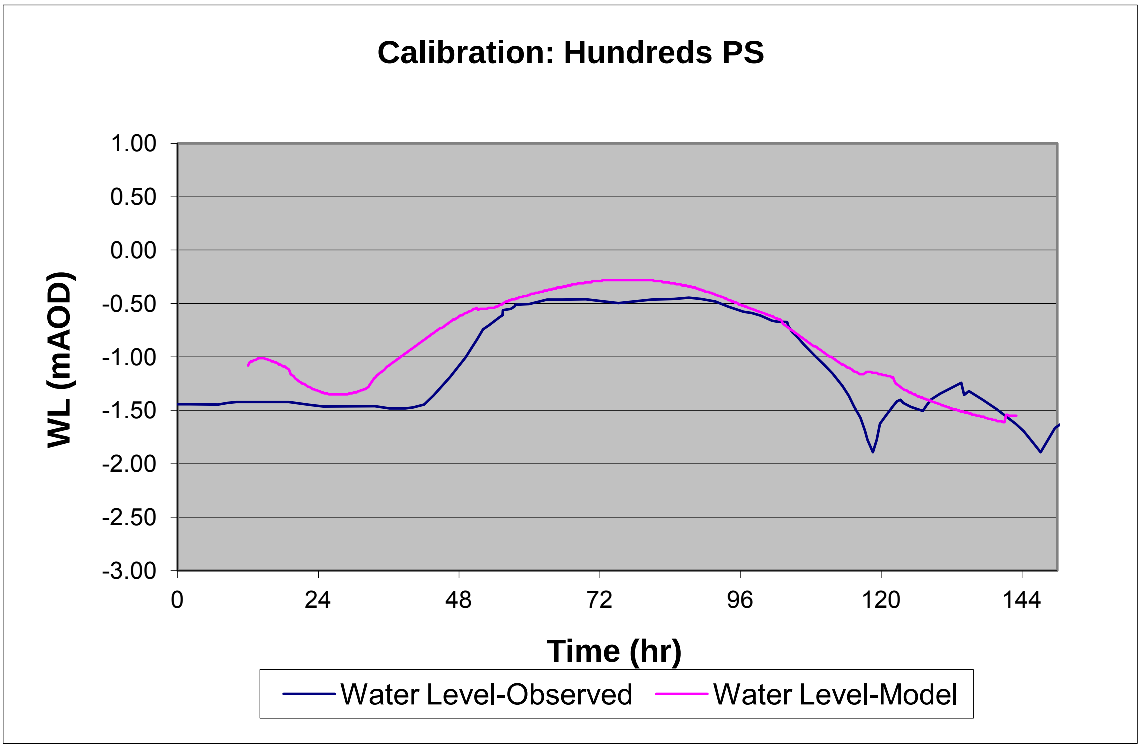

Figure 11: Hundreds PS calibration April 1998

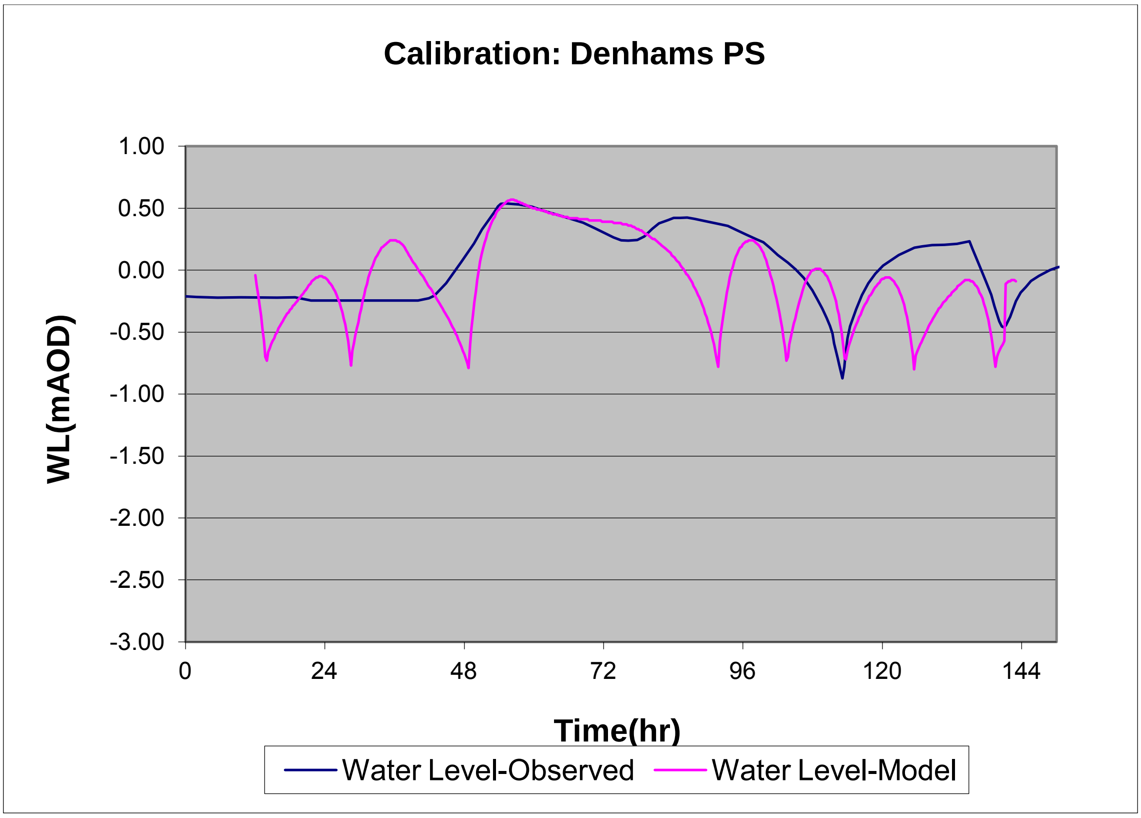

Figure 12: Hundreds PS calibration April 1998

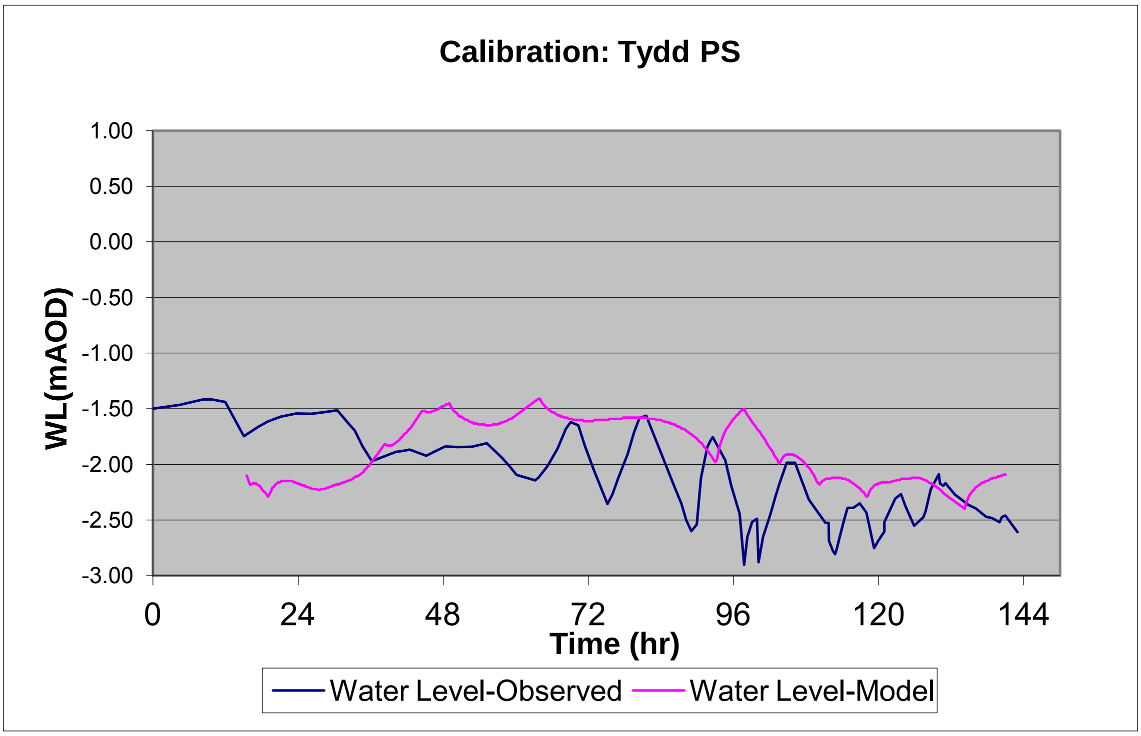

Figure 13: Hundreds PS calibration April 1998

Step 3.4 – Design run

New (2012) method	Old (1993) method
In this example a design run is made for the 1 in 50 years return period flood (2% annual probability) using the calibrated hydraulic model. Figure 14 shows the design hydrographs for the sub catchments of the model. Figure 15 shows the 3D view of 1 in 50 year peak water levels in the model reaches.	For this example no modelling had been undertaken.

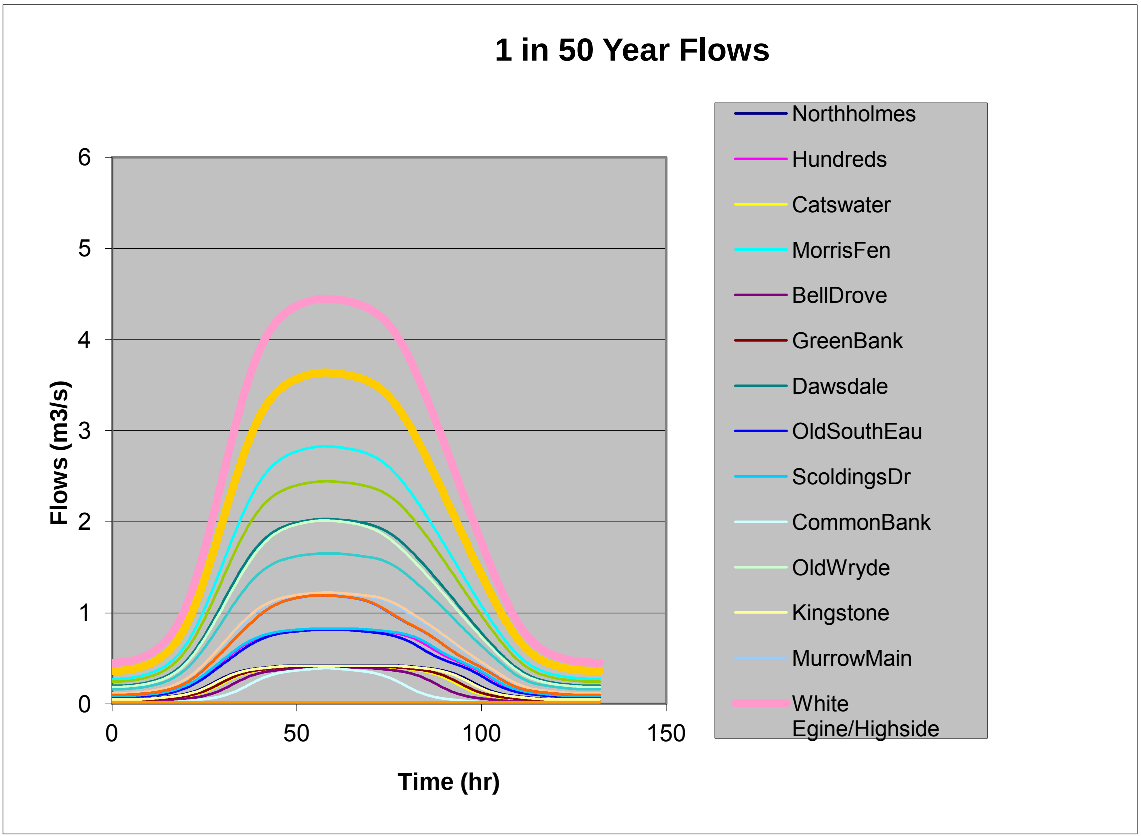

Figure 14: 1 in 50 year sub catchment design inflows

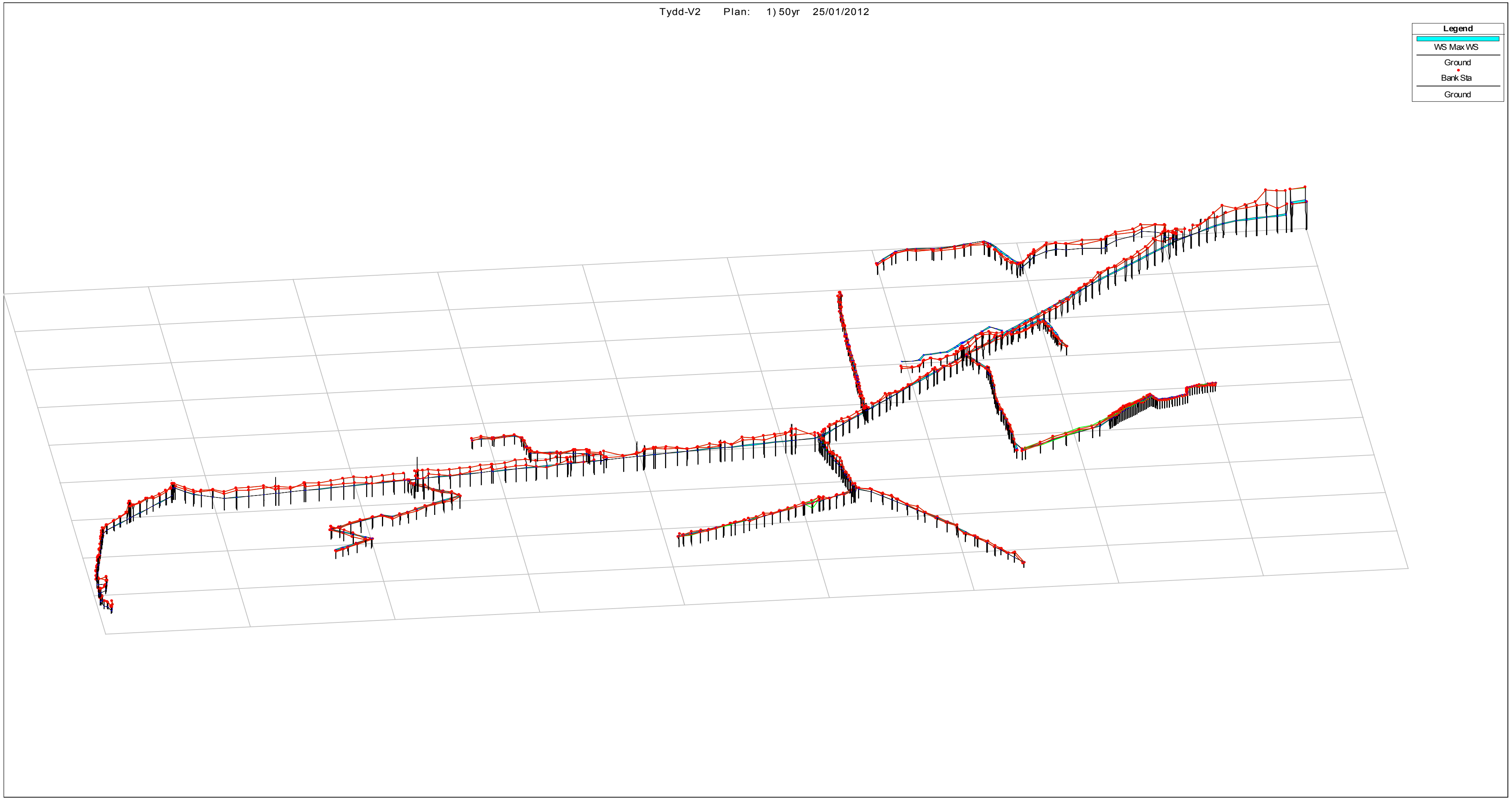

Figure 15: 3D View of 1 in 50 peak water levels in the model reaches

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