Haygrove Limited

Proposed Polytunnel Development at Trumpet, Haygrove Farm, Ledbury, Herefordshire

Water Management Audit

Report 20114

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QUALITY MANAGEMENT REPORT

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EXECUTIVE SUMMARY

GENERAL

- Haygrove Limited is seeking planning permission to extend the existing 41.8 ha of polytunnels at Haygrove Farm, Ledbury, Herefordshire with 13.9 ha of new polytunnels on an existing adjacent orchard site known as Trumpet.

- The planning application must be accompanied by a water management audit (WMA) report to address primarily the issue of increased runoff from the proposed development and impacts on downstream stakeholders.

- This WMA report presents the results of detailed hydrological modelling of runoff from Trumpet under existing (rural) and future conditions, with particular reference to potential impacts at Priors Court, located downstream of Haygrove Farm.

- Proposed polytunnel development will be phased. Phase 1 involves the erection of 7.6 ha of polytunnels and will span two approximately years. This will be followed by Phase 2 development on the remaining area.

RUNOFF MODELLING FOR EXISTING (RURAL) CONDITION

- The HEC-HMS model was applied to simulate runoff from Trumpet under existing conditions. Model parameters were calibrated against the Flood Estimation Handbook (FEH) ReFH model representing the baseline rural condition.

- Modelling was carried out for design storms with return periods of 10 years (Q_{10}) and 100 years (Q_{100}). In line with Environment Agency stipulations, an allowance for future increases in runoff due to climate change was included (Q_{100+CC}).

- The HEC-HMS model including Trumpet successfully matched baseline (ReFH) hydrographs at Priors Court for summer and winter storms for all design frequencies.

PHASE 1 DEVELOPMENT

- The HEC-HMS model was modified to simulate runoff from Trumpet Phase 1 polytunnel areas. Runoff peaks and volumes at the Trumpet outfall were, as expected, significantly increased.

- Impacts translate downstream and there is a small but significant increase in flood peaks at Priors Court.

- There is only a 2% chance that the Q_{100} event will occur during the two-year design life of Phase 1. Furthermore, based on historic evidence, the small increase in peak flow and runoff volume at Priors Court for the 10-year events would <u>not</u> cause flooding of the property. Nevertheless, adopting a precautionary approach, Haygrove Limited is proposing to construct a temporary pool to provide additional storage and attenuation of peak runoff.

- Modelling confirmed that with the construction of a small (1,300 m^2) pool with inflow augmented by the diversion of some surplus flow from the northern field units, impacts at Priors Court of Trumpet Phase 1 development were either neutral or beneficial.

PHASE 2 DEVELOPMENT

- Full development at Trumpet will include the construction of a new pond on an existing polytunnel field 'Baeza'. The pond is required to provide essential irrigation supplies and to mitigate against increased polytunnel runoff.

- Some 1.5 ha of existing polytunnels on Baeza will be removed to accommodate the new pond.

- Allowing buffer zones between the pond external embankment footprint and existing trees and hedgerows and future access roads etc, the pond will cover an area (external) of 9,400 m^2, be up to 4.0 m deep with a capacity, below the spillway crest, of 19,000 m^3.

- Inflow to the pond will originate from the residual area (1.0 ha) of Baeza polytunnels to the north, and piped inflows linked to grass swales draining Trumpet polytunnel runoff.

- The HEC-HMS Phase 2 model was run with varying outlet arrangements and included the scenario of zero initial drawdown. This is extremely precautionary for summer conditions when pond levels will normally be lowered by reduced inflows, irrigation abstractions and evaporation losses.

- With the selected outlet arrangement (pipe diameter 350 mm set at invert level 74.5 mAOD), peak flows at Priors Court were **reduced** for all scenarios, with one exception.

- For the summer Q_{100+CC} event and zero initial drawdown, the peak at Priors Court is increased, but by only 2%. With an initial drawdown of only 0.1 m, there is no increase.

- With full development of Trumpet polytunnels, the impact on downstream flood peaks is effectively neutral or beneficial.

EXECUTIVE SUMMARY

CONTENTS

1 INTRODUCTION

1.1 BACKGROUND

Haygrove Limited (the Client) has operated Haygrove Farm, located 3 km west of Ledbury (Figure 1), for more than 25 years. The existing farm covers 72 ha and produces soft fruit under polytunnels (strawberries, raspberries, blackberries and cherries), watered by trickle irrigation sourced by pumping from on-farm ponds. In mid-summer, the maximum area covered by polytunnels (2008) is 41.8 ha, 58% of the farm total.

The ponds collect runoff from polytunnel covers providing essential irrigation supplies and play a vital role in mitigating flood risk downstream.

A Water Management Audit (WMA) for Haygrove Farm was submitted in 2009[1] in support of a retrospective planning application for the erection of polytunnels, as required under then changes to planning legislation. The WMA dealt primarily with flood risk issues associated with the erection of polytunnels, with particular reference to flood peaks and runoff volumes at Priors Court, located downstream of Haygrove Farm on Pixley Brook. The WMA demonstrated that, with a combination of storage and attenuation of polytunnel runoff in the ponds, and rural sustainable drainage (RSuDS) measures, there would be no significant increase in downstream flooding risk.

Haygrove has now acquired additional land, known as 'Trumpet', adjacent to the existing western farm boundary (Figure 2). It covers 17.3 ha and is currently orchard growing organic apples and pears. It is proposed to replace the existing orchard with polytunnels for the cultivation of organic strawberries. This will result in a future (maximum) polytunnel coverage of 54.2 ha at Haygrove Farm, a 30% increase over the present situation.

The proposed development at Trumpet is to be phased. Phase 1 involves the erection of 7.6 ha of polytunnels and will span approximately two years. This will be followed by Phase 2 development on the remaining area.

1.2 SCOPE OF WATER MANAGEMENT AUDIT

The WMA addresses water-related issues raised in Planning Policy Statement 25 (PPS25)[2, 3] and the Supplementary Planning Document (SPD) concerning polytunnel development produced by Herefordshire Council[4]. Issues concerning water resources and water quality were addressed in the previous WMA and apply equally to the proposed Trumpet extension.

The primary scope of this WMA is to identify potential impacts of proposed polytunnel development, for both Phase 1 and Phase 2, on <u>flood risk beyond the development</u>. The main concern expressed by the Environment Agency (EA) and the SPD is that polytunnel development should not increase flooding risk for all events up to the 1% (100-year) design storm, including allowance for future increases resulting from climate change. Future development should incorporate comprehensive sustainable drainage proposals to mitigate and manage the inevitable increase in surface runoff.

Runoff from Haygrove Farm drains to Pixley Brook which flows southwards past Priors Court (Figure 1). Priors Court is the closest downstream development that could potentially be affected by development at Haygrove Farm and has been flooded in the past, most notably in July 2007, a nationally severe event. The potential impact of polytunnel development at Trumpet is assessed, as in previous studies, in the context of future flooding risk at Priors Court.

1.3 STRUCTURE OF REPORT

The WMA is presented in the following sections:

- Section 2 summarises Trumpet physical characteristics, topography, drainage and present land use.
- Section 3 describes runoff modelling for Trumpet, and the downstream catchment of Pixley Brook to Priors Court, under baseline rural conditions.
- Section 4 analyses impacts of the Trumpet Phase 1 development and proposed measures for short-term mitigation of increased flood runoff.
- Section 5 presents proposals for Phase 2, incorporating the construction of a new pond to provide essential irrigation supplies and to mitigate against increased flood risk associated with full development of Trumpet.

Tables, Figures and Plates are included separately after the main report text. Audit tables for runoff modelling are contained in Annex A.

2 TRUMPET PRESENT CONDITION

2.1 SITE LOCATION

The Trumpet extension limit is shown in Figure 2, located west of the existing Haygrove polytunnel field F ('Baeza'). It is bounded to the north by the A438 and to the south by an existing tree-lined watercourse that flows to the North Branch (HG North), a tributary of Pixley Brook.

2.2 TOPOGRAPHY

Detailed topographic data for previous studies was obtained from LiDAR (aerial survey) data purchased from the EA. An additional 'tile' (SO6539) was purchased to extend coverage to include the Trumpet area. The area was flown in December 2005 giving gridded elevations at 2.0 m horizontal resolution. Raw data were processed using industry standard MapInfo Professional[5] and Vertical Mapper[6] software to produce contours and cross sections, and to compute field and pond surface areas.

Contours across Trumpet are shown in Figure 3. Elevations range from more than 100 mAOD on the northern boundary to 74 mAOD in the south-eastern corner. There is a distinct natural depression and ridge running eastwards through the site, an important feature relating to existing and future drainage.

The location of cross sections through the site is shown in Figure 4, with sections plotted in Figure 5. Section F approximately defines the existing watercourse profile on the southern boundary. Remaining sections clearly show the west-east ridge dividing the main depression and the southern boundary.

2.3 CURRENT LAND USE

Trumpet is currently under orchard growing organic apples and pears, divided into three main units (T1, T2, T3) separated by earth tracks (Figure 6). Orchard tree rows are generally aligned downslope with intermediate grass cover (see Plates 1-4). The existing ground cover is extremely effective in promoting infiltration and reducing surface runoff from the site under existing conditions.

3 RUNOFF MODELLING FOR RURAL CONDITION

3.1 RAINFALL-RUNOFF MODELLING TOOLS

Two industry-standard rainfall-runoff modelling applications were applied. Catchment-scale modelling, applicable to Pixley Brook at Priors Court and the main tributary sub-catchments, was carried out using procedures from the Flood Estimation Handbook (FEH)[7] and associated software.

Previous runoff modelling for Haygrove Farm[1], with and without polytunnel cover, was based on ad-hoc spreadsheet models for individual field runoff, routing through on-farm ponds using HEC-RAS[8] and integrating results to give design runoff hydrographs for the farm and the overall catchment to Priors Court.

This procedure has been superseded by application of the Hydrologic Modeling System (HEC-HMS)[9] developed by the US Army Corps of Engineers. HEC-HMS provides comprehensive procedures for rainfall-runoff modelling on a plot or catchment scale, flood routing through channels and pipes, reservoir modelling, and direct comparison with observed runoff hydrographs.

3.2 MODEL DESIGN CRITERIA

3.2.1 Design Storm Return Periods

Three design return periods have been applied:

- 10-year ($\mathbf{Q_{10}}$), used to dimension post-development farm drainage infrastructure (eg swales and pipes). This is based on the pragmatic assumption that temporary disruption of farm activities, for example through overspill of open grass swales, can be tolerated for more extreme events.

- 100-year ($\mathbf{Q_{100}}$), the baseline frequency for assessing the impact of post-development runoff on downstream flooding risk. This was applied for the proposed Trumpet Phase 1 development (§4.2).

- 100-year with allowance for future increases due to climate change ($\mathbf{Q_{100+cc}}$). This is an EA requirement for evaluating the long-term impact of post-development runoff on downstream flooding risk and was applied for Trumpet Phase 2 development. For design storm rainfall, a climate change 'surcharge' of 20% was applied, in line with EA standards for the expected design life for polytunnels (30 to 50 years).

3.2.2 Design Storm Profiles

Design storm profiles common to both FEH and HEC-HMS modelling were based on a storm duration of 3.7 hours, the default for the Pixley Brook catchment to Priors Court based on the FEH 'Revitalised' rainfall-runoff model (ReFH)[10, 11]. Design storm profiles for summer and winter, derived from the FEH depth-duration-frequency (DDF) model on the FEH CD-ROM[12] are plotted in Figure 7. Storm profiles, and runoff modelling for both ReFH and HEC-HMS models, were based on a 0.1-hour (6 minute) time increment, commensurate with the time of concentration for polytunnel field runoff.

Peak intensity for summer profiles is 2.6 times higher than for equivalent winter profiles. With application of the DDF seasonal adjustment factor, design storm total rainfall is also higher by about 50%.

3.3 Baseline Design Flood Hydrographs

The default baseline condition for runoff in the rural condition is represented by design ReFH hydrographs. Descriptors for the main sub-catchments and the total catchment of Pixley Brook at Priors Court, abstracted from the FEH CD-ROM are given in Table 1. It should be noted that descriptors relate to the baseline **rural** situation. They do not reflect existing polytunnel development at Haygrove Farm (urban extent, URBEXT2000, for HG North and HG South is zero), nor of the on-farm ponds (lake index, FARL, is 1.0). Assessment of the impact of Trumpet development is therefore based on the relative impact at Priors Court assuming the upstream catchment is rural.

ReFH hydrographs include direct runoff representing excess storm rainfall after subtracting losses, and a significant baseflow component that, for HG North, increases peak flows by about 6% in the summer and 9% in the winter. The baseflow contribution to the volume of runoff is more significant, in excess of 20% of the total runoff volume (Table 2).

HEC-HMS model parameters were initially calibrated against HG North direct runoff hydrographs after subtracting baseflow. Storm losses were based on the SCS curve number (CN) approach that allows for an initial abstraction (Ia) loss and ongoing losses through the storm determined by CN which varies with ground cover, underlying soil characteristics and antecedent moisture conditions.

ReFH design rainfall-runoff parameters for HG North are summarised in Table 2. Direct runoff is typically 15% of rainfall in the summer and 25% in the winter. With the addition of baseflow, total runoff increases to 19-23% (summer) and 33-38% (winter). Equivalent runoff values for the Trumpet site under present conditions are likely to be lower due to the dense orchard/grass ground cover.

3.4 Downstream Model Routing Calibration

The ReFH model was used to generate design flood hydrographs for the main tributaries, and for Pixley Brook at Priors Court, based on catchment descriptors from the FEH CD-ROM[12], summarised in Table 1. Catchment outfall locations are shown in Figure 2.

The HEC-HMS model schematic for the Pixley Brook catchment is shown in Figure 8. Haygrove North has a catchment area of 0.86 km^2 (86 ha) that includes the 17 ha Trumpet site. Runoff from HG North combines with tributary runoff from HG South and Pixley US at the US Confluence and is then routed down the Pixley Brook channel to Priors Court. In line with previous studies, runoff from the residual catchment between the US Confluence and Priors Court (Pixley DS, 0.9 km^2) is added to the Priors Court runoff without routing.

Figure 8 shows ReFH hydrographs for the design summer Q_{100} event. Peak runoff for HG North, HG South and Pixley DS occurs at about 3.7 hours. Storage and routing effects result in the peak of 2.2 m^3/s at Priors Court occurring later, at 4.5 hours.

The HEC-HMS downstream model used ReFH tributary hydrographs (total runoff including baseflow) as inputs. The hydrograph at the US confluence was then routed down the Pixley Brook channel, combined with the Pixley DS hydrograph, and compared with the Priors Court ReFH hydrograph. The routing effect is illustrated by the hydrograph plot for Priors Court in Figure 8, showing the combined upstream 'inflow' and the routed 'outflow'.

The Muskingham routing method was applied, modifying routing parameters until a good match between ReFH ('observed') and HEC-HMS (simulated) hydrographs was obtained for all design events. Adopted Muskingham routing parameters (K = 0.95 and X = 0.05) produced consistent results for summer and winter events for all design frequencies.

Results at Priors Court for the final calibration are plotted in Figure 9, confirming robustness of the HEC-HMS downstream routing model for all design events.

3.5 RUNOFF FROM TRUMPET UNDER PRESENT CONDITIONS

Natural drainage paths for the Trumpet plot are shown in Figure 10. There are no formal watercourses within the plot, but the natural depression will convey runoff from the north (T1-A and T3-A) and from the central ridge (T1-B and T3-B) eastwards. The depression feeds to a ditch (E Ditch) that flows down the eastern boundary to the North Branch watercourse which has heavily vegetated banks (Plate 5). Runoff south of the ridge (T2, T3-C) flows directly to North Branch, crossing the existing access track (Plate 6).

The HEC-HMS model schematic representing the existing drainage regime is shown in Figure 11. Individual Trumpet drainage units (T1-A etc) are modelled as sub-basins connected by channels representing the natural depression (CH-1, CH-2) and the North Branch watercourse (NB-1, NB-2). Runoff is combined at the Trumpet outfall junction.

The Trumpet model was linked to the existing downstream catchment model, modified as follows:

- The HG North catchment was changed to HG North Residual, representing the sub-catchment area excluding Trumpet (69 ha).
- All inflows from the downstream sub-catchments were represented by ReFH design hydrographs.
- Runoff from the Trumpet Outfall was routed downstream (channel NB Upper) and combined with HG North Residual at HG North.
- Combined runoff was routed downstream to Priors Court based on the previous calibration.

Sub-basin areas and channel length and slope parameters were derived from LiDAR data. Modelling of Trumpet sub-basin runoff was based on the SCS curve number (CN) loss model approach. It allows for an initial abstraction (Ia) loss and ongoing losses through the storm determined by CN which varies with ground cover, underlying soil characteristics and antecedent moisture conditions. Preliminary estimates of CN were abstracted from standard tables, further refined by comparing resultant runoff for HG North with ReFH output.

The kinematic wave (K-W) procedure, suitable for conditions of overland flow with no defined drainage channel, was applied to transform excess rainfall to give the sub-basin runoff hydrograph. In the rural condition, the simplest form of K-W model was adopted comprising a single plane (overland flow) with variable length and slope, feeding lateral inflow to a 'channel' at the downstream boundary of the sub-basin (see Figure 16).

Sub-basin runoff was combined and routed through channels (natural depression and existing North Branch watercourse) using the Muskingham-Cunge method based on channel length, channel slope and Manning's roughness coefficient. Nominal channel losses, higher in summer than in winter, were incorporated.

Model parameters for the existing (rural) condition are summarised in Table A.1. Model global summaries for summer and winter design events are presented in Table 3, giving peak flow and runoff volume for each model unit. Runoff from individual sub-basins represents 21-26% of storm rainfall in the summer, increasing to 32-39% in the winter. Allowing for attenuation and conveyance losses, runoff at the Trumpet Outfall represents 17-19% of storm rainfall in the summer, increasing to 29-32% in the winter. Modelled rural runoff from Trumpet is proportionate to 'observed' total runoff values for the HG North sub-catchment (Table 2).

Example elements of HEC-HMS output are plotted in Figure 12, illustrating sub-basin rainfall-runoff, channel routing, and total runoff for the Q_{10} event. It shows:

- higher sub-basin storm losses in the summer (12A) than winter (12B);
- runoff attenuation and loss by conveyance along the natural depression (12C);
- flow at the Trumpet Outfall (12D), with a significantly higher contribution from the northern sub-units via E Ditch;
- combined flow at the Pixley Brook US Confluence (12E);
- routed flow at Priors Court compared with the 'observed' baseline rural condition.

The trumpet plot represents 4.4% of the total catchment area to Priors Court. Simulated runoff volumes are of a similar order (Table 3). HEC-HMS simulated hydrographs at Priors Court, incorporating the Trumpet rural model, are plotted in Figure 13 with 'observed' baseline hydrographs. The HEC-HMS model closely preserves hydrograph peak flows, timing of the peak and runoff volume.

4 TRUMPET PHASE 1 DEVELOPMENT

4.1 POLYTUNNEL COVERAGE

Proposed polytunnel coverage for Phase 1 Trumpet development, with a life span of approximately two years, is shown in Figure 14. Sub-plot ID's were modified to be consistent with Haygrove designations. Phase 1 involves the removal of existing orchard and erection of polytunnels on Pear 2 (P2), Pear 1 (P1) and Emotion 7 (E7), covering a gross area of 9.8 ha. Polytunnels will be covered year-round, an increasing trend that provides additional crop protection and avoids the cost and labour involved with traditional winter de-skinning, with additional benefits related to health and safety and the working environment.

Individual tunnels will be 8.5 m wide and 4.65 m high, separated by leg rows covered with permeable Mypex fabric. Each tunnel will contain five longitudinal raised bunds with a plastic cover growing organic strawberries, with intermediate areas also covered with Mypex to suppress weed growth.

Field areas were derived from LiDAR data. The net area to be covered with polytunnels was based on Haygrove field measurements, adjusted to allow for the future provision of grass swales to convey polytunnel runoff (§4.2.3). The residual (non-tunnel) field area will accommodate a combination of undeveloped ground, access tracks, lateral grass verges and areas required for a reservoir and vehicle turning. Under Phase 1, some 44% of the Trumpet plot gross area will be covered with polytunnels.

PHASE 1 DEVELOPMENT

	PEAR 2	PEAR 1	TOPAZ	GALA	EMOTION 7	TOTAL
Field Area (ha)	2.51	3.41	4.29	3.21	3.91	17.33
Tunnel Area (ha)	1.88	2.49			3.20	7.57
% Tunnels	75%	73%	0%	0%	82%	44%

4.2 PHASE 1 HEC-HMS MODEL

4.2.1 Drainage Units

The rural condition HEC-HMS model (Figure 11) was modified to reflect Trumpet Phase 1 development as shown in Figure 15. Model parameters are detailed in Table A.2. The model to the Trumpet Outfall was linked to the downstream catchment model as for the rural condition.

4.2.2 Modelling Polytunnel Runoff

Runoff from polytunnels was modelled using the Kinematic Wave (K-W) procedure[13] that closely represents the physical processes involved, as described in Figure 16.

Rain falling on the tunnel plastic (plane) drains laterally (overland flow element) to collector channels representing the Mypex leg rows, leading to the main channel at the base of each tunnel block. A second plane represents the residual (non-tunnel) area and was modelled as a single sub-unit with drainage controlled by the average field length and slope.

Runoff from the Phase 1 development was based on curve number (CN) values established for the baseline (rural) condition, adjusted for tunnel areas by the inclusion of a factor representing the impervious percentage. The area covered by tunnel plastic is in reality 100% impervious. However, this does not allow for percolation losses that will occur as lateral runoff from polytunnels flows down the Mypex-covered leg rows.

On-farm observations confirm that Mypex can absorb all low intensity rainfall, with reduced percolation during intense storms and/or when underlying soils are saturated. Percolation rates quoted by manufacturers and suppliers of Mypex fabric are highly variable, but for Mypex aligned downslope with percolation through the fabric enhanced by infiltration through sumps at leg struts, a loss rate of 20-25% is considered to be realistic. The K-W model does not include an option for infiltration through collectors (Mypex leg-rows). It was modelled indirectly by reducing the impervious percentage of the tunnel plane to 70% which, combined with underlying CN values, produced rates for tunnel runoff entering the downstream channel of 78% in the summer and 82% in the winter.

4.2.3 Runoff Conveyance

The existing natural depression will be replaced at the base of each polytunnel unit by shallow grass swales (eg SW-1), dimensioned to provide a conveyance capacity capable of passing the design 10-year peak runoff rate at full development (Phase 2). For more extreme events runoff will exceed the conveyance capacity and spill to adjacent fields or watercourses, potentially causing some disruption to farm activities, albeit for brief periods. Swales will have a trapezoidal section with shallow side slopes (3-H to 1-V) to allow the passage of farm vehicles. With natural longitudinal slopes, a standard swale will have a bottom width of 0.5-1.0 m and a nominal depth of about 0.5 m, giving a standard top width of 4.0 m. The area to be covered with polytunnels was adjusted to account for the land required for swales.

Along the southern boundary, runoff in excess of swale capacity will drain naturally to the North Branch watercourse. This was explicitly included in the HEC-HMS model by the inclusion of inflow-diversion functions (eg DIV-A).

4.2.4 Phase 1 Modelling Results

Model parameters for Phase 1 development are given in Table A.2. The model was run for design Q_{10} and Q_{100} summer and winter events. The Q_{100+CC} event was not applied as it is not logical to allow for climate change for a development that will only span two years.

Example elements of HEC-HMS output are plotted in Figure 17, illustrating sub-basin rainfall-runoff with polytunnel coverage, channel routing, and total runoff at the Trumpet Outfall for summer and winter Q_{100} events. It shows:

- significant increase in runoff (reduced storm losses) with polytunnel development (17B) compared with undeveloped sub-basins (17A);
- significantly higher peak runoff rates with polytunnel development in summer (17B) than in winter (17C);
- reduced attenuation and infiltration through conveyance in grass swales compared with the existing rural condition (17D);
- runoff at Trumpet Outfall higher in summer (17E) than in winter (17F).

HEC-HMS model global summaries for summer and winter design events are presented in Table 4, giving peak flow and runoff volume for each model unit. The impact of polytunnel development is shown by comparing runoff peaks and volumes for developed (eg P2-C) with undeveloped (eg TP-N) sub-basins. Peak runoff rates and volumes are more than doubled in the summer when peak rainfall intensity is higher. In the winter, peak runoff rates are marginally increased, but runoff volumes are also doubled.

At the Trumpet Outfall, peak runoff rates and runoff volume for winter Q_{10} and Q_{100} events are increased by approximately 50%. The impact for summer events is more dramatic especially for the Q_{100} event for which peak runoff rate and runoff volume increase by more than 100%. Impacts are clearly shown by hydrograph plots for rural and Phase 1 development conditions, in Figure 18. More rapid runoff associated with polytunnel development also reduces the time-to-peak at the outfall. Downstream impacts are more muted but still significant, illustrated by hydrograph plots for the HG North sub-catchment, representing routed runoff from Trumpet combined with the HG North residual runoff.

Phase 1 development impacts at Priors Court are shown in Figure 19. There is a small but systematic increase in runoff peak for all design events, most notably for summer Q_{100}.

4.3 MODIFIED PHASE 1 DEVELOPMENT (PHASE 1P)

4.3.1 Addition of Temporary Pool

There is only a 2% chance that the 100-year design event will occur during the two-year design life of Phase 1. Furthermore, based on historic evidence, the small increase in peak flow and runoff volume at Priors Court for the 10-year events would not cause flooding of the property. Nevertheless, adopting a precautionary approach, Haygrove Limited is willing to construct a temporary pool to provide additional storage and attenuation of peak runoff.

Topographically, the favoured site for the pool is in the south-east corner of the Trumpet site immediately upstream of the outfall. This will allow gravity-feed of surface runoff from southern Trumpet unit P1-S via grass swales along the southern boundary (Figure 14). Runoff from unit E7-S would drain directly to the pool. This arrangement will not, under natural conditions, allow gravity feed from the northern units that drain to E Ditch.

Trial HEC-HMS model runs were made with a small pool covering about 1,000 m^2 located between the 74.0 and 75.0 m contours, with a nominal storage capacity of 1,000 m^3. A low-level pipe outlet discharges water from the pool to the Trumpet Outfall. Inclusion of the pool caused a significant reduction in peak flows at the Trumpet Outfall, but not sufficiently to completely negate the increase at Priors Court.

This can be explained by considering runoff volumes given in Table 4. Total inflow to the pool is via the southern swale, SW-C and tunnel unit E7-S. The combined inflow for 10-year events is less than 1,000 m^3, ie the pool does not fill to capacity and is under-used. Total inflow volume for 100-year events does exceed the pool capacity, but allowing for outflows, the pool only just fills in the summer and remains below capacity in the winter.

The temporary pool dimensions were therefore modified and inflow arrangements changed to take some runoff currently discharging from the northern units to E Ditch via grass swale SW-3:

- The pool outline was expanded between the 74.0 and 75.5 m contours giving a surface area of about 1,300 m^2 and a nominal capacity below the spillway of 1,800 m^3 (Figure 20).
- A low-level pipe outlet, 200 mm diameter and invert level 74.0 mAOD, discharges to the Trumpet Outfall.
- A nominal spillway, 2.0 m wide with crest elevation 75.4 mAOD was included.
- Inflow to the pool was augmented by diverting some runoff from swale SW-3 via a pipe connection. The pipe will be 300 mm diameter and extend southwards for

about 70 m along the eastern Trumpet boundary. It will be located on the western side of the existing access track and will have no disruptive impact on E Ditch or the existing hedgerow boundary. The pipe may have to be part buried to give sufficient gradient.

The pipe was added to the HEC-HMS model as an Inflow-Diversion function (DIV-E), assuming a nominal pipe capacity of 100 lit/sec. Flows in swale SW-3 in excess of the pipe capacity will spill to E Ditch as before. The modified HEC-HMS schematic representing Phase 1P development is shown in Figure 21.

4.3.2 Phase 1P Model Results

Example elements of HEC-HMS output are plotted in Figure 22, illustrating operation of the temporary pool, including effects of the pipe inflow diversion, and total runoff at the Trumpet Outfall:

- the pool successfully stores and attenuates inflows, reducing peak outflow to 25-30% of the inflow peak (22A, 22B);
- operation of the pipe diversion from swale SW-3, with flow up to the pipe capacity of 100 l/s diverted to the pool and the excess discharging to E Ditch (22C, 22D);
- runoff at the Trumpet Outfall significantly modified with contributions from the residual flow in E Ditch and outflow from the pool (22E, 22F).

HEC-HMS model global summaries for summer and winter design events are presented in Table 5, giving peak flow and runoff volume for each model unit, to be compared with the no-pool scenario given in Table 4. Sub-basin runoff for polytunnel and rural units is unchanged, except for unit E7-S where runoff is marginally reduced by land-take for the temporary pool. Flows spilling to E Ditch are significantly reduced by the pipe diversion to the pool.

At the Trumpet Outfall, peak runoff rates for all design events are approximately halved. Impacts are clearly shown by hydrograph plots for rural and Phase 1P development conditions in Figure 23. Inclusion of the temporary pool and inflow diversion reduces peak runoff below the rural condition rates except for the summer Q_{100} event. The impact on HG North sub-catchment runoff is also significant with Phase 1P peaks slightly higher for the summer Q_{100} event, but similar or lower for remaining design events.

Routed hydrographs at Priors Court (Figure 24) confirm that, with the temporary pool, proposed Phase 1P polytunnel development at Trumpet will have no adverse impact on downstream flood risk.

5 TRUMPET PHASE 2 DEVELOPMENT

5.1 GENERAL CONCEPT

Phase 2 development will involve the erection of polytunnels on 13.9 ha of the Trumpet site. This will inevitably increase surface runoff at the Trumpet Outfall and result in additional demands for irrigation supplies. Available resources for irrigation at Haygrove Farm, comprising several existing on-farm ponds (Top Pond, New Pond and Lakeside Lake) would benefit from the provision of additional water storage. Polytunnel coverage at full development is summarised below.

PHASE 2 DEVELOPMENT

	PEAR 2	PEAR 1	TOPAZ	GALA	EMOTION 7	TOTAL
Field Area (ha)	2.51	3.41	4.29	3.21	3.91	17.33
Tunnel Area (ha)	1.88	2.49	3.65	2.64	3.20	13.86
% Tunnels	75%	73%	85%	82%	82%	80%

At full development the area of polytunnels for the Trumpet extension represents less than 4% of the total catchment area to Priors Court. The future gross polytunnel area for Haygrove Farm (existing and planned) is less than 15% of the total area.

The basic concept for Trumpet Phase 2 development is shown in Figure 25, incorporating the following features:

- removal of existing polytunnels from the southern half of the Baeza field (Field F) that lies immediately east of Trumpet;
- construction of a new pond ('Baeza Pond') to regulate runoff from Trumpet and increase available water resources for irrigation;
- the temporary pool constructed located in the south-east corner of Trumpet for Phase 1 development will be infilled and the diversion pipe from SW-3 removed;
- inflow to Baeza Pond will be from three sources:
    - North Trumpet via a pipe link to swale SW-3;
    - South Trumpet via the pre-existing swales on the southern boundary, leading to a pipe buried beneath polytunnels in unit E7-S;
    - runoff from the residual area of Baeza polytunnels to the north of the pond. This will provide an additional flood mitigation benefit since runoff from the existing Baeza polytunnel field currently discharges directly to the North Branch watercourse virtually unregulated.

5.2 BAEZA POND

5.2.1 General

The external footprint and dimensions of Baeza Pond are determined by a number of factors:

- from a water resource perspective, maximise pond capacity while not exceeding the current Reservoirs Act threshold of 25,000 m^3;

- provide adequate buffer zones to existing trees on the southern boundary of Baeza, and the hedgerow along the E Ditch alignment on the western boundary (see Figure 6);
- provide space for a new access track between the northern pond limit and remaining Baeza polytunnels;
- restrict embankment height to a visually acceptable limit with allowance for future screening;
- ensure pond inlet works allow for gravity-feed from upstream runoff sources;
- minimise habitat disruption that might be caused by inlet and outlet works.

5.2.2 Pond Location and Dimensions

Figure 26 shows the proposed Baeza Pond external embankment footprint. It is designed to meet the above objectives and was established after consultation with the Client and the Landscape Architect. It allows ample buffer space to the south and west and leaves room for the access track to the north. It assumes that all existing polytunnels south of the 76.0 m contour, covering about 1.5 ha, will be removed. Approximately 1.0 ha of polytunnels to the north will be retained.

The pond external footprint is indicative. The final footprint is likely to be more rounded and ecologically sympathetic, but it is assumed that it will retain overall pond dimensions and the elevation-capacity relationship.

The external embankment footprint covers an area of 9,400 m^2 (LiDAR). Existing ground levels increase from 72.0 mAOD in the south-east corner to 75.5 mAOD to the north and west. Six cross sections spanning the external embankment toe boundary, located as shown in Figure 26, were abstracted using Vertical Mapper[6] and are plotted in Figure 27.

The pond was dimensioned assuming an embankment crest level of 75.5 mAOD and a minimum pond bed level of 71.5 mAOD, giving a maximum depth of 4.0 m. Figure 27 shows embankment sections and potential excavation below existing ground level based on a nominal crest width of 1.0 m and embankment side slopes of 1 : 3 (V:H), as adopted for the existing Haygrove Farm New Pond. The embankment height above existing ground level is minimal on the northern and western margins, increasing to 3.5 m in the south-east corner. Within the embankment crest, average pond dimensions are just over 90 m (W-E) and just under 80 m (N-S).

5.2.3 Pond Elevation-Capacity

A combination of pond cross sections and LiDAR data was used to derive the elevation-capacity curve for Baeza pond, plotted in Figure 28. Gross capacity to the embankment crest (75.5 mAOD) is 22,700 m^3, below the Reservoirs Act threshold. An emergency spillway will be required to prevent overtopping of the embankment. A freeboard allowance of 0.5 m has been assumed giving a storage capacity of 19,000 m^3 at the spillway crest elevation of 75.0 mAOD.

5.2.4 Pond Inlet Works

Inflow from the residual Baeza polytunnels north of the pond will discharge directly to the pond, under the new access track, most likely through a series of small diameter pipes. Proposed arrangements for the north and south pipe inlets are shown in Figure 29.

The north inlet will convey runoff from swale SW-3 close to where, under the Phase 1P layout, it discharges to E Ditch. It will need to be piped across E Ditch then underground to the pond with an assumed outfall at 74.5 mAOD. The pipe will be approximately 22 m long with an average gradient of 0.05 m/m.

The south inlet will convey runoff from southern Trumpet through a pipe buried beneath polytunnels of unit E7-S following back-filling of the temporary pool. Elevation, and hence location, of the pipe inlet is constrained by the need to allow sufficient gradient for gravity feed to the pond without an excessively large pipe diameter. Under the assumed arrangement, the pipe inlet takes off from just below the 76.0 m contour, passing underground about 0.5 m below the surface (for protection) before crossing E Ditch, continuing underground beneath the embankment, to the pond. The pipe outfall to the pond, maintaining the natural pipe gradient, is at elevation 74.3 mAOD. The total pipe length is about 145 m with an average gradient of 0.01 m/m.

The diameter of inlet pipes was based on the magnitude of source flow peaks at the pipe inlets. It is desirable to maximise the volume of flood runoff captured. However, it is not logical to dimension pipes for rare events whereby, for most of the time, they will not flow at full capacity.

Preliminary estimates of inlet pipe dimensions were based on average Q_{10} (summer and winter) flood peaks, derived from preliminary runs of the Phase 2 HEC-HMS model with full polytunnel development. Capacity curves for the north and south inlet pipes, applicable to the pipe gradients, are plotted in Figure 29.

Inflow to the north inlet pipe is from swale SW-3. Design peak Q_{10} runoff rates are 0.55 m^3/s (summer) and 0.29 m^3/s (winter). A pipe diameter of 350 mm was initially selected, with a design capacity of 0.5 m^3/s. Flow in excess of the pipe capacity will spill to E Ditch.

Inflow to the south inlet pipe is from swale SW-B with design peak Q_{10} runoff rates of 0.24 m^3/s (summer) and 0.13 m^3/s (winter). A pipe diameter of 350 mm was initially selected, with a design capacity of 0.2 m^3/s. Flow in excess of the pipe capacity will spill to swale SW-C combining with runoff from tunnel unit E7-S.

5.2.5 Pond Outlet Works

Baeza Pond outlet works will comprise an emergency free-overflow spillway with a crest invert level of 75.0 mAOD, and an outlet pipe located slightly east of the south-west corner of the pond (to avoid existing trees), shown in Figure 29. The outlet is assumed to pass through the embankment and then underground, beneath the existing access track, to discharge to the North Branch watercourse.

Figure 30 shows the indicative outlet works arrangement with an outlet pipe length of 32 m and an average gradient of 0.05 m/m. The final outlet pipe diameter and invert level were determined by modelling.

5.2.6 Cut & Fill Issues

It is evident from Figure 27 that the volume of cut required to achieve a pond base elevation of 71.5 mAOD will significantly exceed the volume of fill that can be used for construction of the embankment. An estimate based on pond cross sections indicates a surplus cut volume of approximately 11,000 m^3. This will be disposed of within the Haygrove Farm boundary to avoid the need for off-site transport.

The most likely disposal site is within the existing depression that runs W-E through the Trumpet site (Figure 3) that, based on cross sections shown in Figure 5, can easily absorb such a volume.

5.3 HEC-HMS Phase 2 Model

5.3.1 Model Layout

The HEC-HMS model schematic for Phase 2 development is shown in Figure 31. It shows:

- addition of Baeza Pond;
- inflow to the pond from the remaining area of Baeza polytunnels, BZ-3 (1.0 ha);
- northern pipe inlet downstream of swale SW-3, modelled by an inflow-diversion function (DIV-BP-N) with flow in excess of the pipe capacity spilling to E Ditch;
- southern pipe inlet downstream of swale SW-B, modelled by an inflow-diversion function (DIV-BP-S) with flow in excess of the pipe capacity spilling to swale SW-C;
- outflow from Baeza pond (pipe outlet and spillway), assumed to discharge to the North Branch watercourse at the Trumpet Outfall.

The model was linked to the downstream sub-catchment model to Priors Court.

5.3.2 Model Inputs

Model sub-basins were modified to reflect full polytunnel development on the Trumpet site by converting remaining undeveloped units (TP-N etc). Phase 2 model parameters are detailed in Table A.3.

The Q_{100} design storm profiles used for Phase 1 were replaced to include the climate change allowance (Q_{100+CC}). The increase in storm rainfall (20%) is illustrated in design profiles shown in Figure 7.

Model results are ultimately evaluated by comparing simulated flood hydrographs at Priors Court under Trumpet polytunnel development with baseline conditions which, for existing and Phase 1 models, assumed the residual Pixley Brook catchment was rural, as represented by ReFH output.

Under Phase 2 development, runoff from the residual Baeza polytunnels (1.0 ha) will flow to Baeza Pond. Priors Court baseline hydrographs were modified for the Phase 2 model to include runoff from the existing Baeza polytunnel field (2.52 ha) which currently drains directly to the North Branch watercourse, virtually unregulated. Existing Baeza runoff was added to the Trumpet Outfall rural model hydrograph and routed downstream. The difference at Priors Court was added to the rural condition hydrographs to give revised baseline hydrographs. The difference at Priors Court is small as shown in Figure 32, ranging from 0.03 m^3/s (W_{10}) to 0.08 m^3/s (S_{100+CC}), but potentially significant in the context of sensitivity of the Baeza Pond operation.

Inflow to Baeza Pond (from north and south inlet pipes and residual Baeza polytunnel runoff) is limited by inlet pipe capacities, as shown in Figure 33. There is no constraint for smaller events (Q_{10}), but inflow peaks are truncated for more extreme events.

5.3.3 Impact of Phase 2 Development on Trumpet Runoff

Table 6 shows the HEC-HMS global summary for Phase 2 development. This is an example for an assumed Baeza Pond outlet pipe set at 74.5 mAOD, diameter 350 mm, with the pond assumed to be full at the start of each event (initial level 74.5 mAOD). Comparison with Table 5 (Phase 1P) shows:

- increased runoff peak and volume from previously developed units (eg P2-C) due to the addition of the climate change allowance for the 100-year events;
- major increase in peak runoff and volume from previously rural units (eg TP-N);
- no spill to E Ditch for summer and winter Q_{10} events. The northern inlet pipe to the pond has sufficient capacity to convey the entire runoff from upstream (SW-3). There is a small volume of spill for the winter Q_{100+CC} event, increasing significantly for the summer Q_{100+CC} event;
- performance of the southern inlet pipe is similar. The residual runoff in swale SW-C for the Q_{10} events originates from polytunnel field E7-S downstream of the pipe inlet;
- the contributory catchment area to the Trumpet Outfall increases to 18.3 ha due to the inclusion of the residual Baeza polytunnel field;
- with pond outlet arrangements used for this example run, runoff peak and volume at Priors Court is marginally reduced for summer and winter Q_{10} events. The increase for Q_{100+CC} events compared with Phase 1P is due to the higher storm rainfall including the climate change allowance.

5.4 PHASE 2 MODEL SIMULATIONS

5.4.1 Baeza Pond

From the flood risk perspective the prime consideration is that, for all design events, the post-development flood peak for Pixley Brook at Priors Court should not exceed the baseline condition peak. At the same time, it is essential that the Baeza Pond outlet pipe should be set as high as possible to maximise available irrigation resources.

The emergency spillway invert elevation is assumed to be fixed at 75.0 mAOD, giving 0.5 m freeboard below the embankment crest. The maximum outlet pipe invert elevation is determined by the pipe diameter ensuring that the pipe soffit remains below the spillway crest.

5.4.2 Model Operation

Operation of the HEC-HMS Phase 2 model is illustrated by plots in Figure 34:

- restriction of pipe diversions to Baeza Pond, and excess spill, for north (34A) and south (34B) inlets for extreme events;
- significant attenuation of pond outflows for summer (34C) and winter (34D) Q_{10} events, with peak pond elevation remaining below the spillway crest with zero initial drawdown;
- increased outflow including significant flow over the spillway for the summer Q_{100+CC} event with the pond starting full (34E), totally resolved with a small initial drawdown (34F), the normal summer condition;
- moderate spillway flow for the winter Q_{100+CC} event with zero initial drawdown (34G);
- flow at Trumpet Outfall for summer Q_{10} combining residual flow in swale SW-C (originating from polytunnel field E7-S) with attenuated outflow from Baeza Pond (34H);

- enhanced flow at Trumpet Outfall for summer Q_{100+CC} with excess flow from pond inlet pipes (SW-C and E Ditch) combined with runoff including spill from Baeza Pond with zero initial drawdown (34I), reduced to primarily excess inlet pipe flows with a small initial drawdown (34J);

- flow at Trumpet Outfall for winter Q_{10} (34K) and Q_{100+CC} (34L) events.

5.4.3 Selection of Outlet Arrangements

Multiple model runs were made varying:

- outlet pipe diameter (300 to 450 mm diameter);

- outlet pipe invert level (74.6 to 74.0 mAOD);

- initial pond level. It should be noted that under normal conditions, the pond will be drawn down in the summer due to a combination of reduced inflows, irrigation abstractions and evaporation losses. Adopting a precautionary approach, all runs included the worst-case scenario with the pond starting full which could occur in extreme circumstances in the summer, as seen in July 2007. Guidance on the minimum initial level was based on known inflow volumes for the various design events compared with the pond elevation-capacity curve (Figure 28).

Results are summarised in Table 7 detailing, for alternative outlet pipe arrangements, peak pond levels and outflows, and the peak flow at Priors Court compared with the baseline peak. For Summer Q_{10}, Winter Q_{10} and Winter Q_{100+CC} events, there is no constraint on outlet pipe diameter or invert level. Peak flow at Priors Court is reduced in all cases, even with the pond starting full (zero initial drawdown). The outlet pipe diameter can be reduced to the minimum 300 mm and the invert level set as high as 74.6 mAOD.

The situation is more sensitive for the **Summer Q_{100+CC}** event. With zero initial drawdown, peak pond levels exceed the spillway crest resulting in a significant increase in peak outflow that translates to Priors Court. However, in all cases an initial drawdown of only 0.2 m provides sufficient storage to attenuate downstream peak flows back to the baseline condition.

The adopted outlet arrangement was a pipe diameter of 350 mm with an invert elevation of 74.5 mAOD. This provides almost 16,000 m^3 of irrigation storage (Figure 28) and is a reasonable compromise between restricting outflow and minimising the rise in peak pond levels.

5.5 IMPACT AT PRIORS COURT

The impact of Trumpet Phase 2 development on design flood hydrographs at Priors Court is shown in Figure 35. The result is **BETTERMENT** for summer Q_{10}, winter Q_{10} and winter Q_{100+CC} events, even with zero initial drawdown of Baeza Pond.

Peak flow at Priors Court is increased for the summer Q_{100+CC} event with zero initial pond drawdown, but the increase is only 0.05 m^3/s, less than 2% of the baseline peak flow. The pond will normally be drawn down in the summer months, in which case peak flows at Priors Court will be reduced. Only in exceptional circumstances, such as occurred in July 2007, will the pond be full in the summer. It is considered that the small risk and minimal consequences of such an event are acceptable given the overall improvement over the baseline condition for all other scenarios.

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[2] Communities and Local Government (March 2010); Planning Policy Statement 25 : Development and Flood Risk; HMSO

[3] Communities and Local Government (December 2009); Planning Policy Statement 25 : Development and Flood Risk Practice Guide; HMSO

[4] Herefordshire Council (December 2008); Polytunnels Supplementary Planning Document; (adopted 5 December 2008)

[5] MapInfo Corporation (May 2007); MapInfo Professional, Version 9.0.2

[6] MapInfo Corporation (April 2005); Vertical Mapper, Version 3.0

[7] Institute of Hydrology (1999); Flood Estimation Handbook (5 volumes)

[8] US Army Corps of Engineers, Hydrologic Engineering Center (March 2008); HEC-RAS River Analysis System (Version 4.0.0)

[9] Hydrologic Engineering Center (December 2013); Hydrologic Modeling System HEC-HMS (Version 4.0); US Army Corps of Engineers

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[11] CEH Wallingford (December 2005); The Revitalised FSR/FEH Rainfall-Runoff Method, spreadsheet implementation

[12] Centre for Ecology & Hydrology (2009); FEH CD-ROM (Version 3.0)

[13] Hydrologic Engineering Center (July 1993); Introduction and Application of Kinematic Wave Routing Techniques Using HEC-1; US Army Corps of Engineers

TABLES

FIGURES

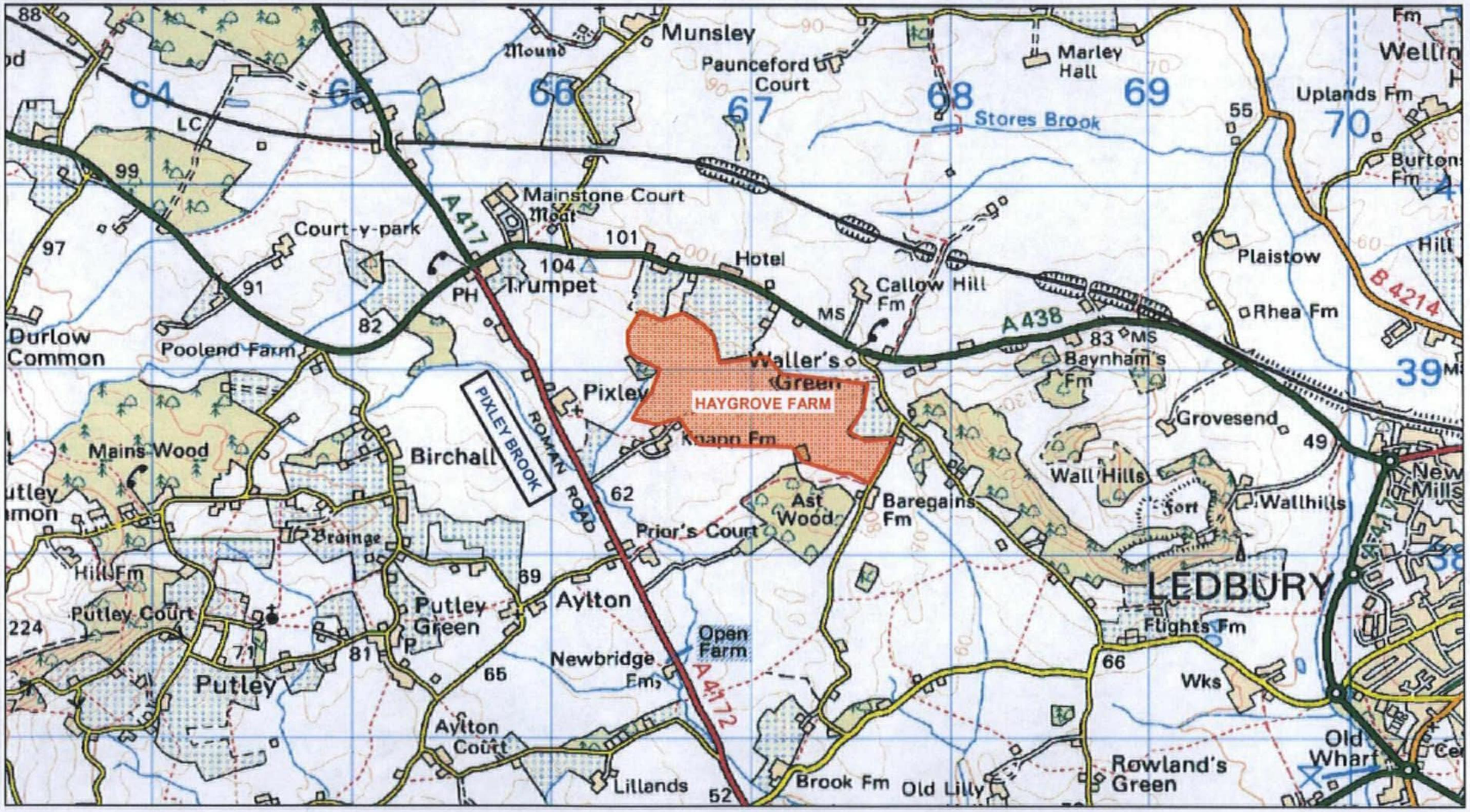
PLATES

ANNEX A

HEC-HMS MODEL AUDITS

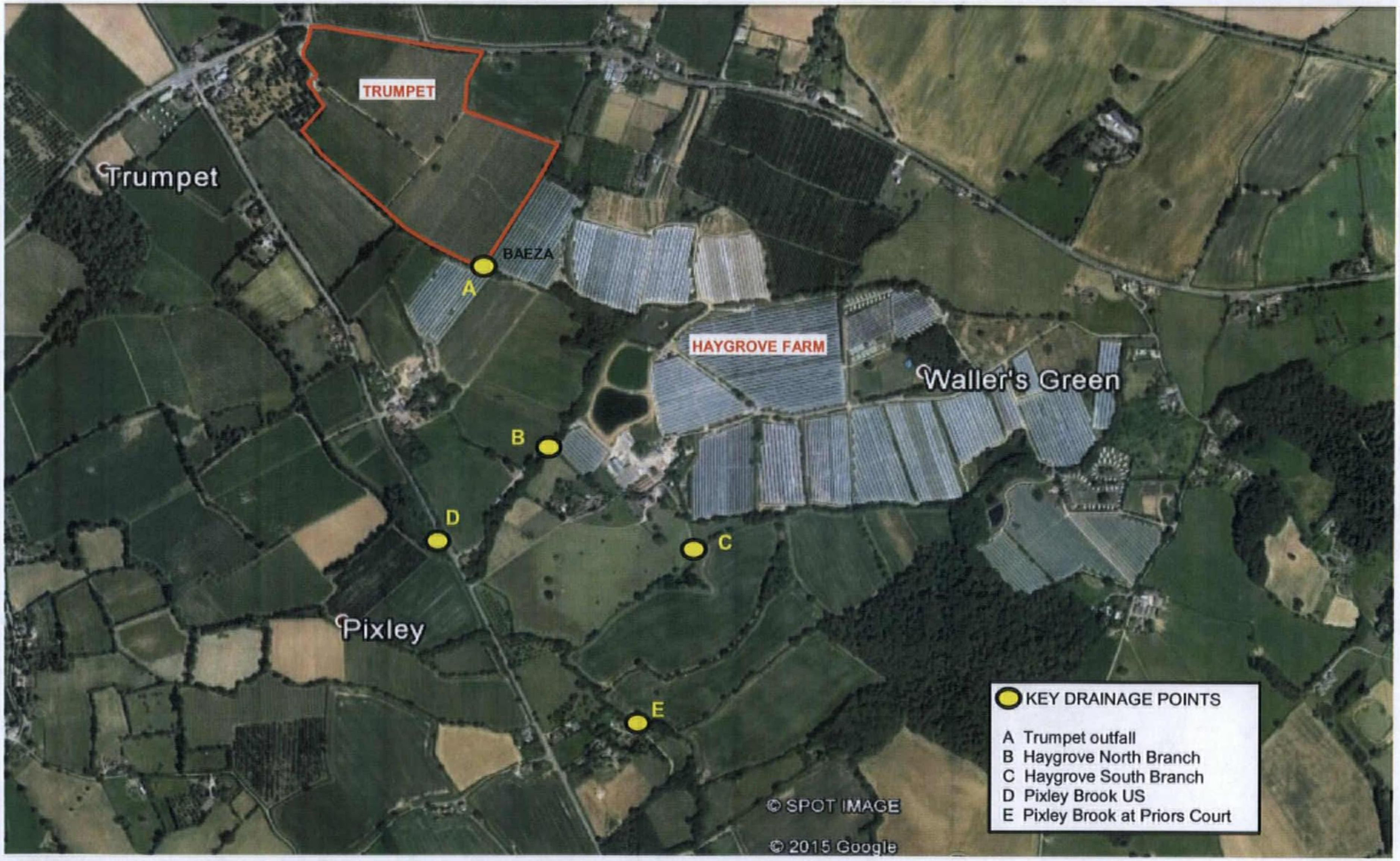
Location Map

Figure 1

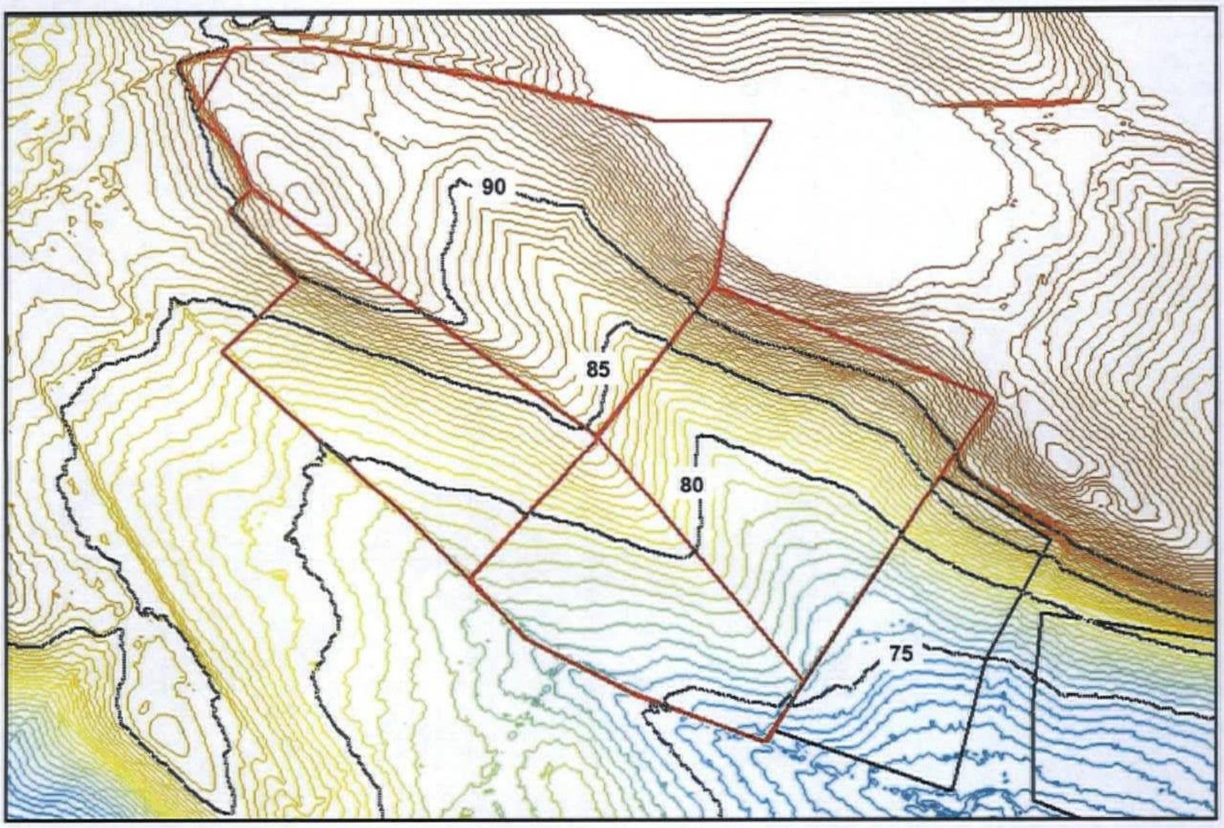


Trumpet Boundary and Key Drainage Points

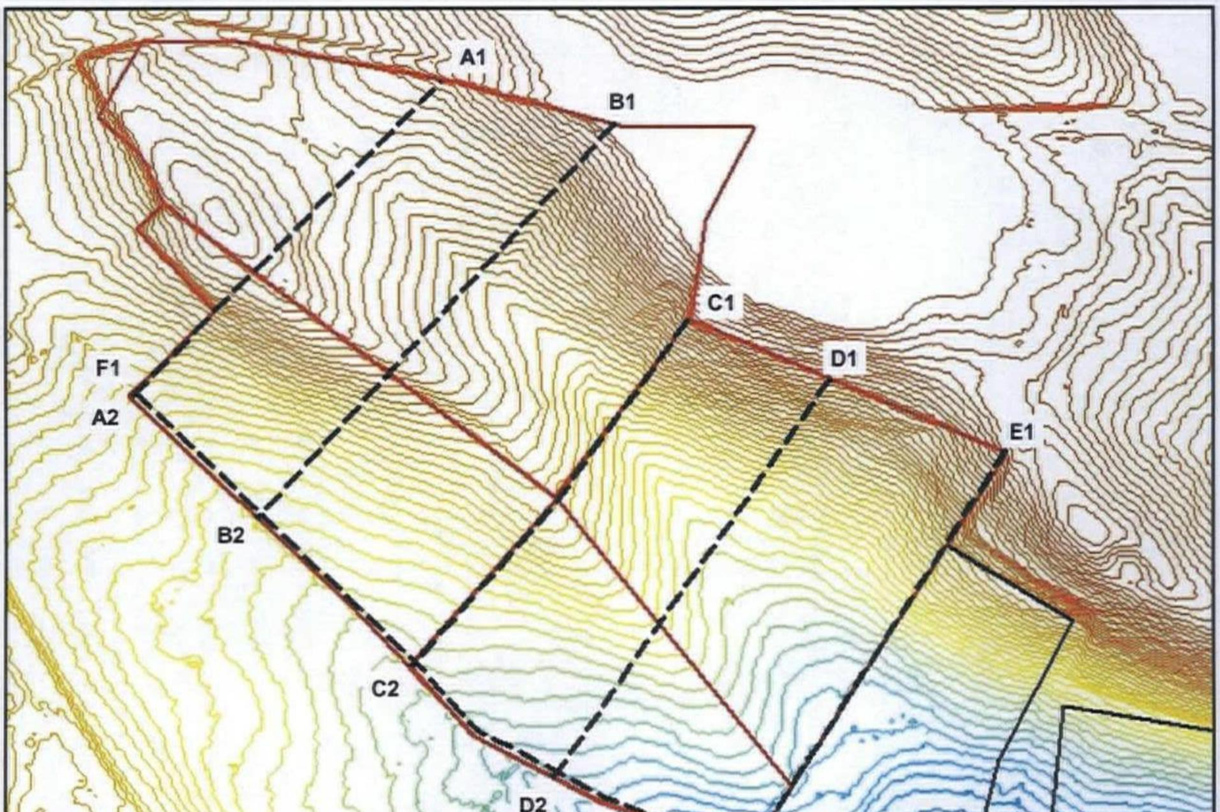
Figure 2



Topography **Figure 3**



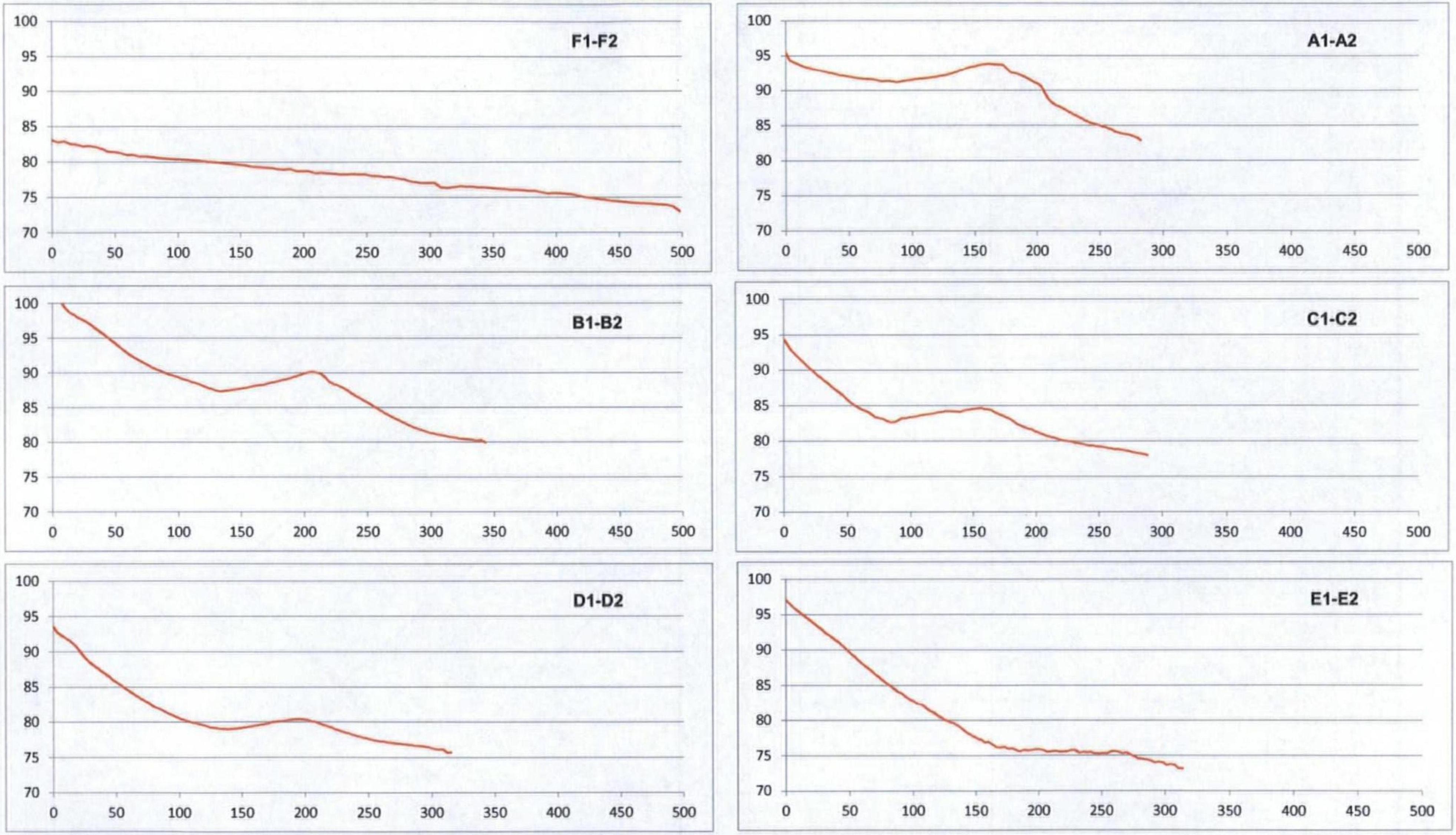
Cross Section Locations **Figure 4**



sections plotted in Figure 5

Trumpet CrossSections

Figure 5



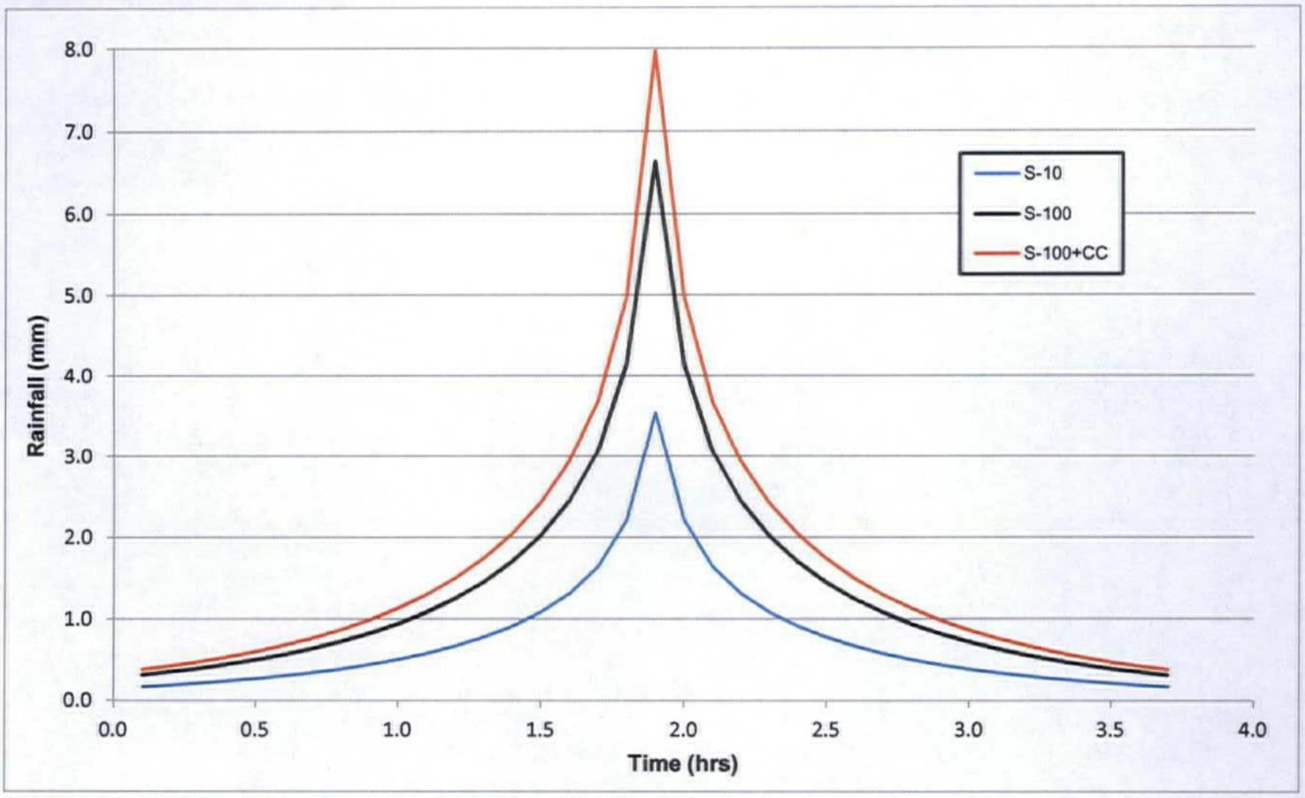
see Figure 4 for section locations

Trumpet Pre-Development Condition

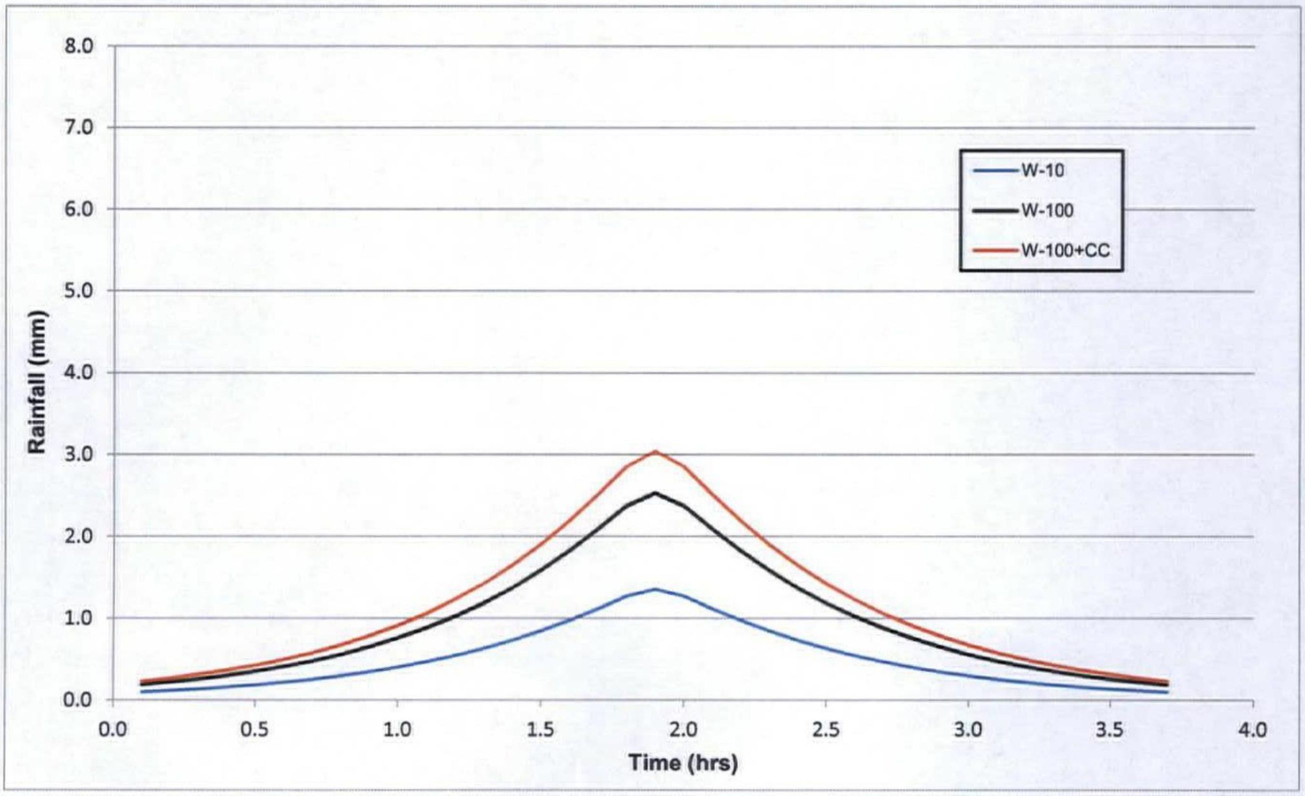
Figure 6

Design Storm Profiles - Summer

Figure 7



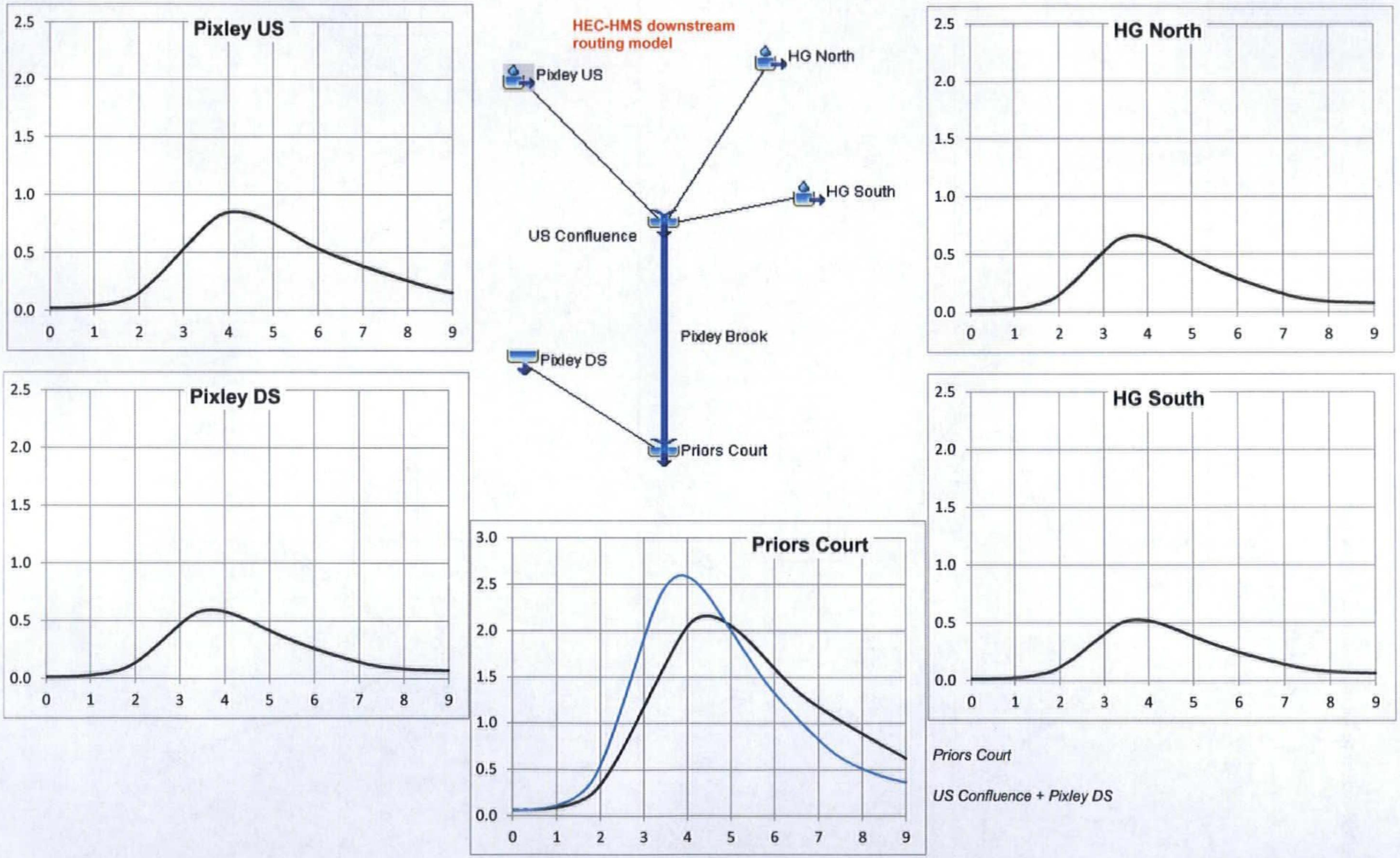
Design Storm Profiles - Winter



design storm profiles for default 3.7-hour storm derived from FEH DDF model

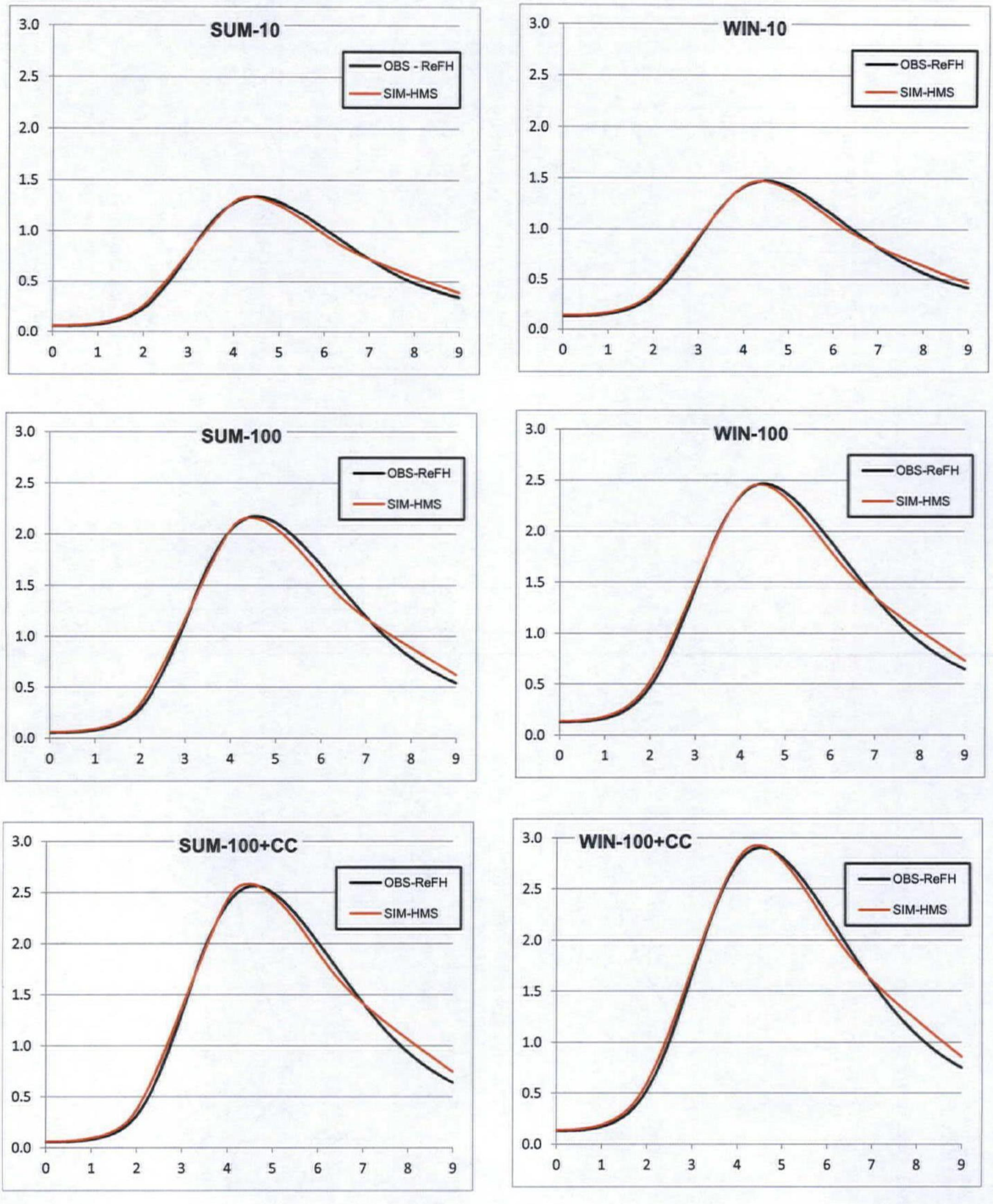
ReFH Sub-Catchment Hydrographs : SUMMER Q_{100}

Figure 8

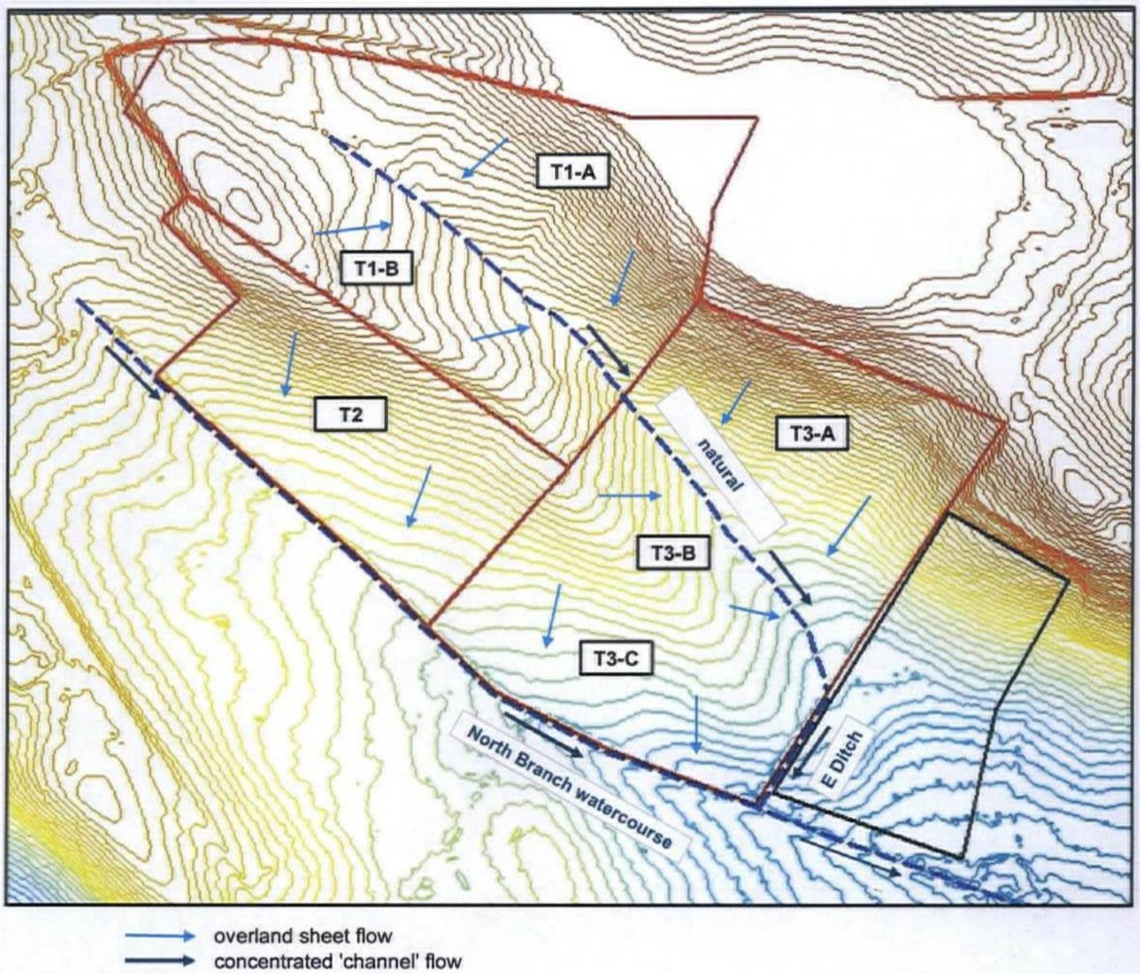


HEC-HMS Routing Calibration at Priors Court

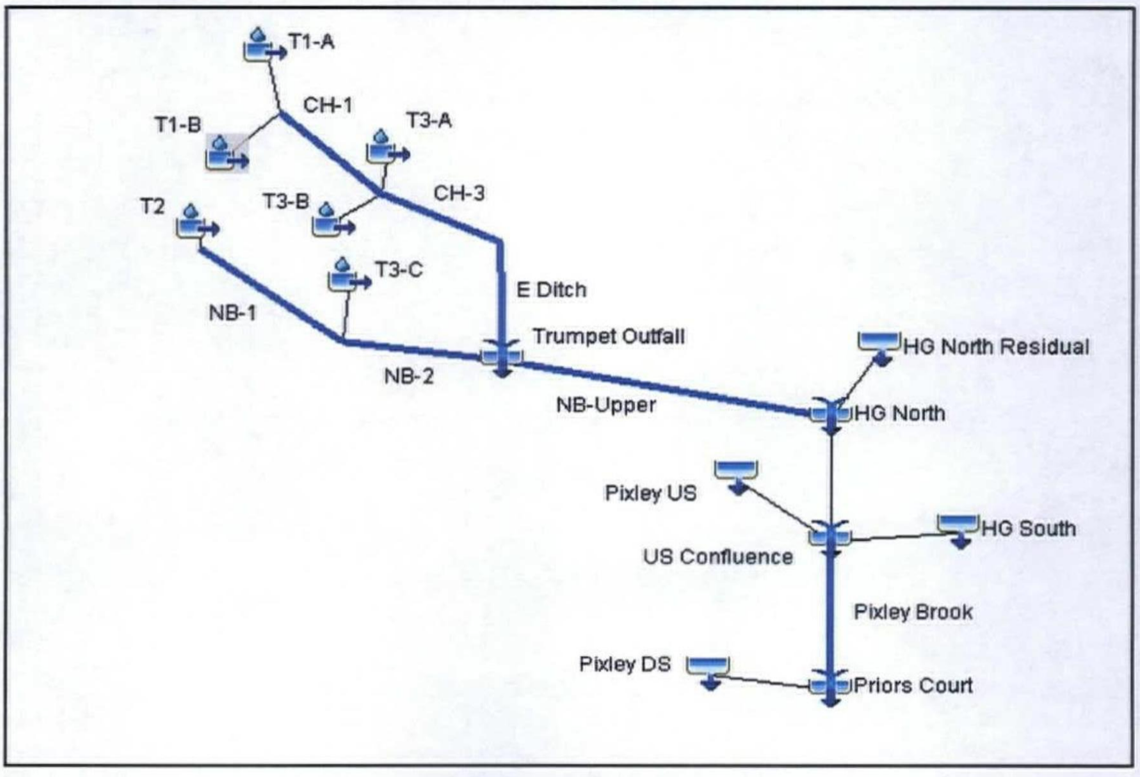
Figure 9



Trumpet Site Natural Drainage Paths **Figure 10**



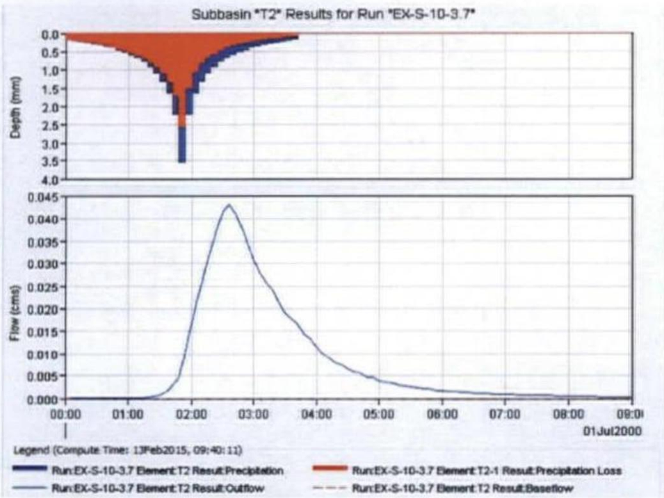
HEC-HMS Model Schematic : Existing (Rural) Condition **Figure 11**



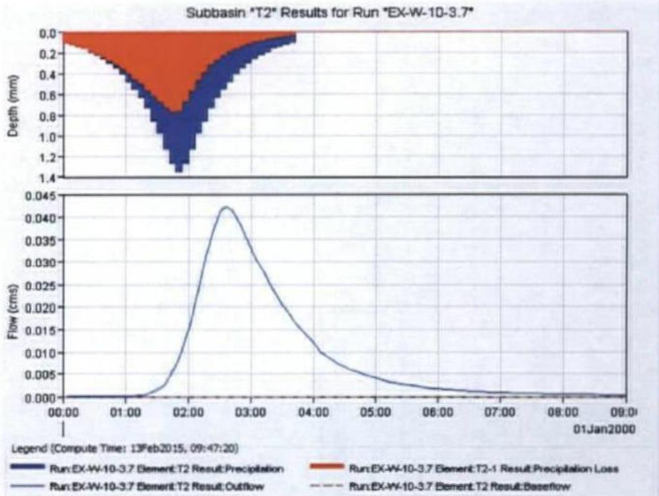
Elements of HEC-HMS Model : Rural Condition Q_{10}

Figure 12

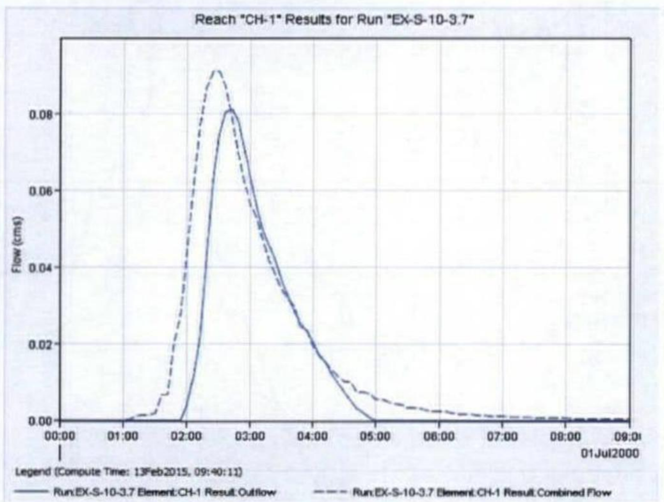
A **SUB-BASIN RUNOFF : T2 SUMMER**



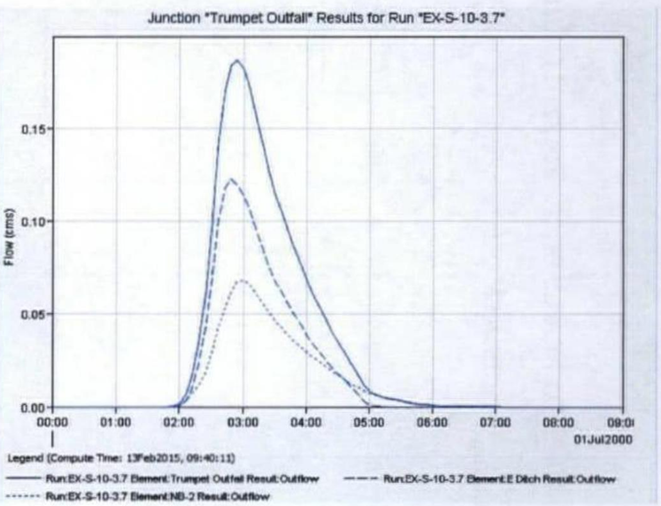
B **SUB-BASIN RUNOFF : T2 WINTER**



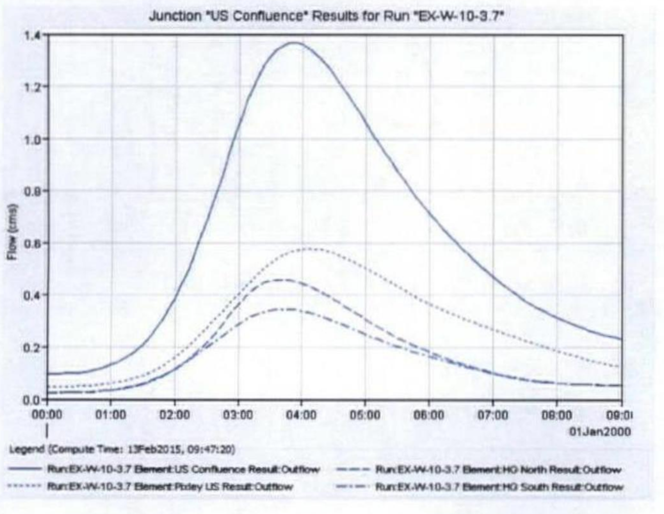
C **CHANNEL ROUTING : CH-1 SUMMER**



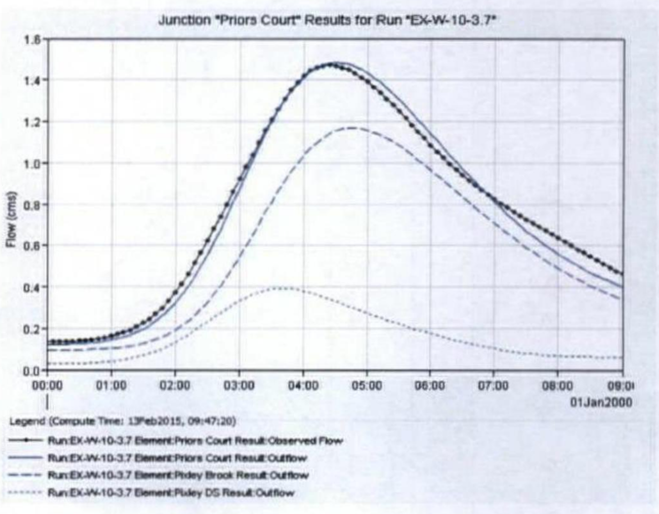
D **FLOW AT TRUMPET OUTFALL : SUMMER**



E **COMBINED FLOW AT US CONFLUENCE : WINTER**

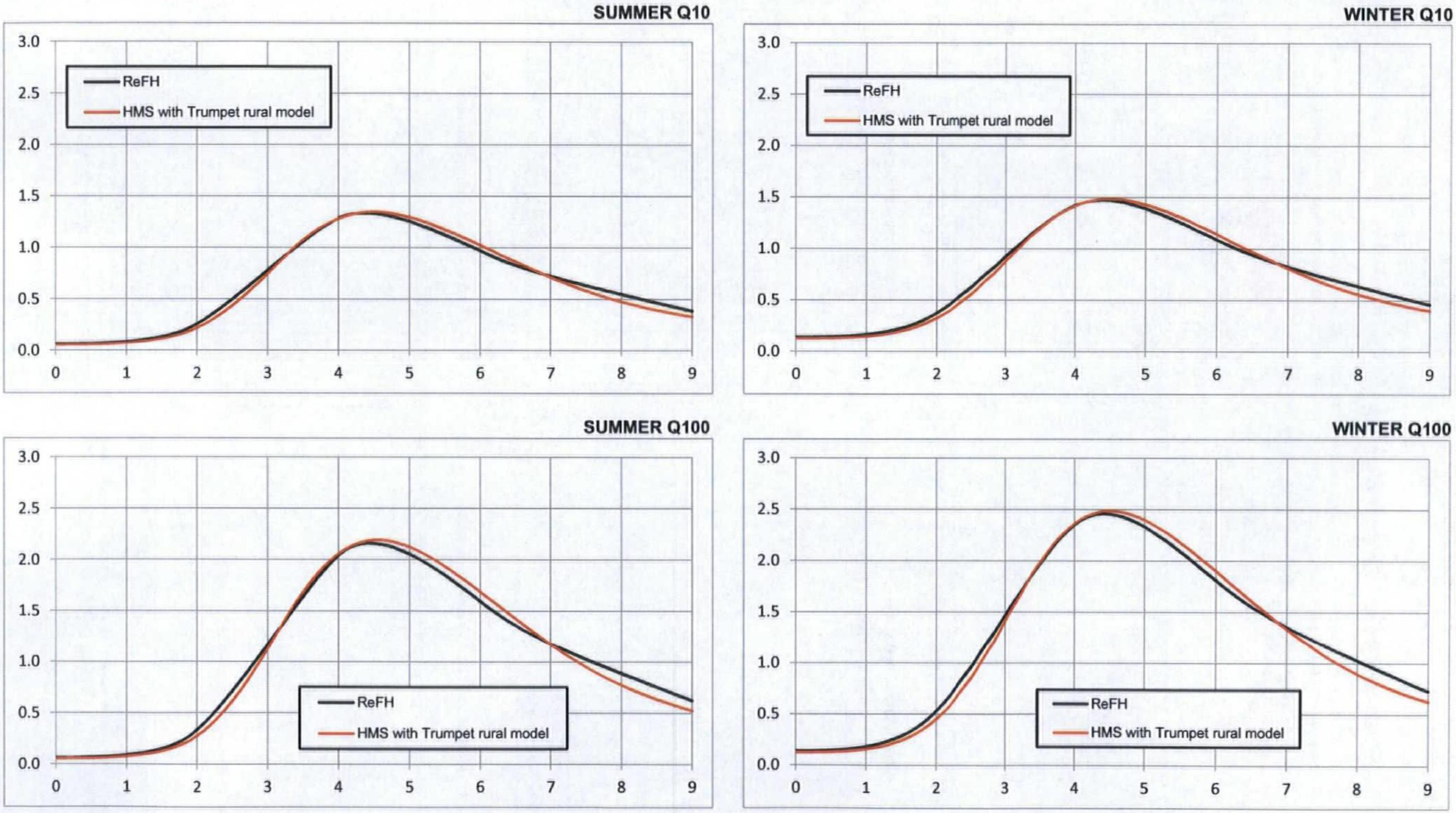


F **FLOW AT PRIORS COURT : SUMMER**

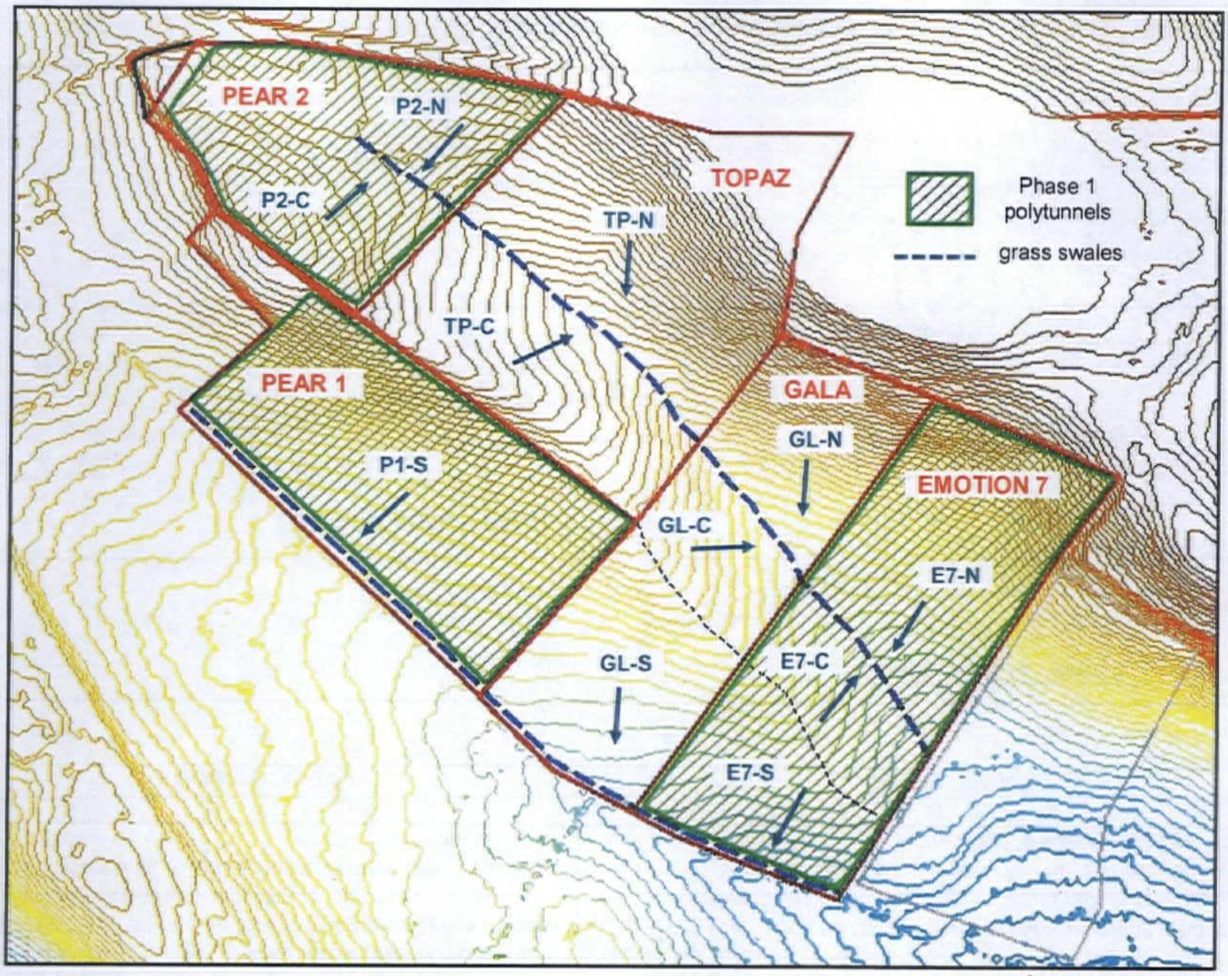


Comparison of Priors Court ReFH Model with HEC-HMS Trumpet Model : Present (Rural) Condition

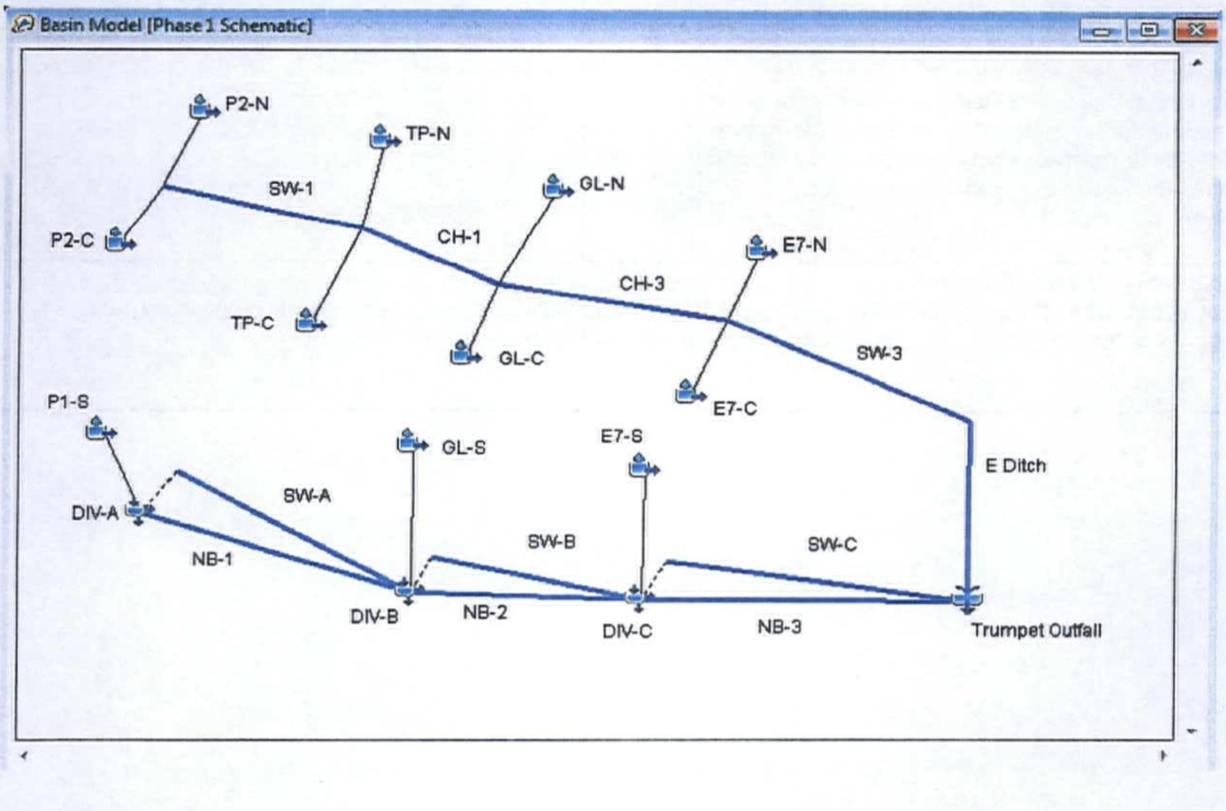
Figure 13



Proposed Phase 1 Development and Drainage Units **Figure 14**

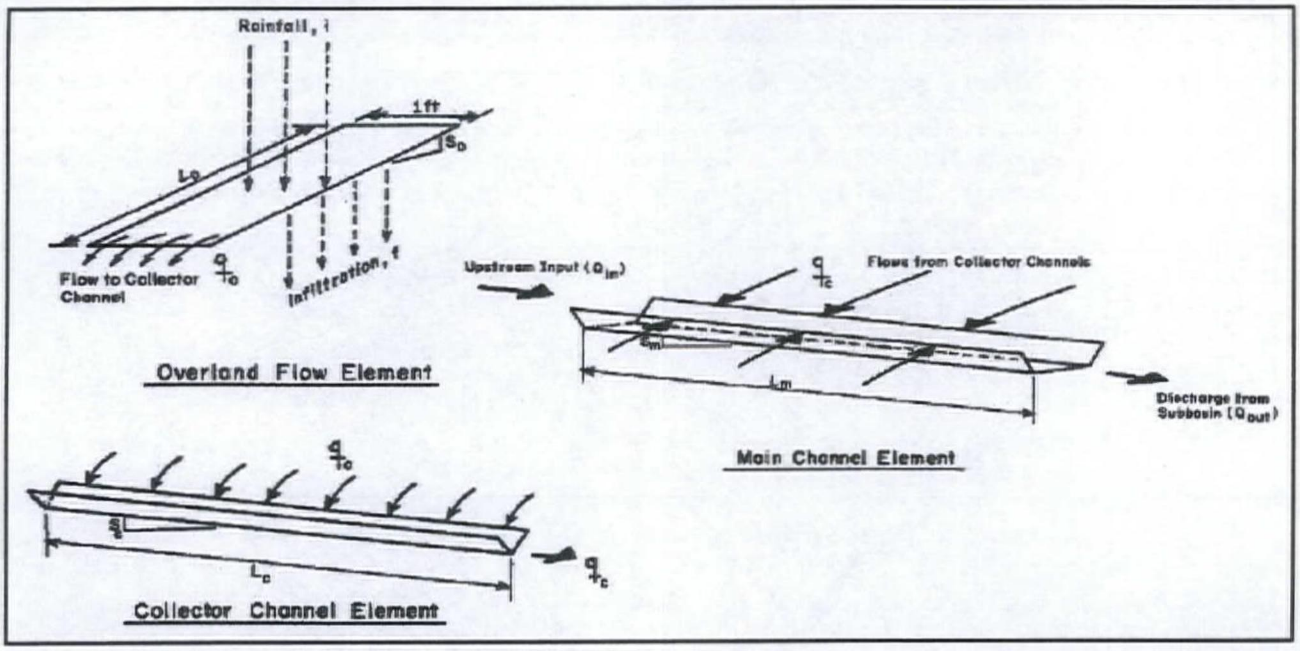


HEC-HMS Model Schematic : Trumpet Phase 1 **Figure 15**



Application of Kinematic Wave Transform for Sub-Basin Runoff

Figure 16

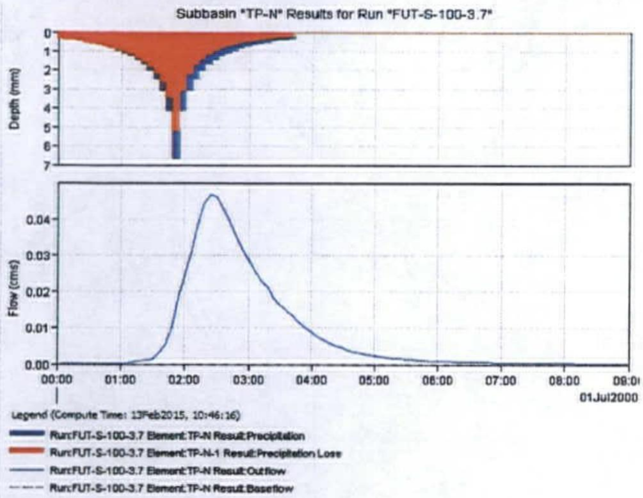


POLYTUNNEL COVER		RURAL CONDITION
Overland Flow Element (Plane) Represents runoff from individual polytunnel with overland flow length = tunnel width, slope = tunnel side slope, infiltration determined by % impervious and underlying CN, attenuation by overland flow roughness coefficient 'R'		Single plane with overland flow length = field length, slope = field longitudinal slope. Plane represents overland sheet flow. Zero % impervious. Infiltration determined by variable underlying CN, attenuation by variable overland flow roughness coefficient 'R'
Collector Channel Element (Collector) Mypex leg row receiving runoff from tunnel plastic as lateral inflow. Overland flow length = tunnel length, slope = tunnel longitudinal slope. Assumed rectangular shape with effective width 1.0 m. Infiltration determined indirectly by % impervious and underlying CN assigned to Plane, attenuation by Manning's roughness coefficient 'n'		No collector channel. Runoff passes directly from plane to main channel
Main Channel Element (Channel) Located at base of each tunnel block. Receives runoff from individual collectors as lateral inflow and optionally from upstream channel. Length = tunnel block width, slope = tunnel block lateral slope at base		

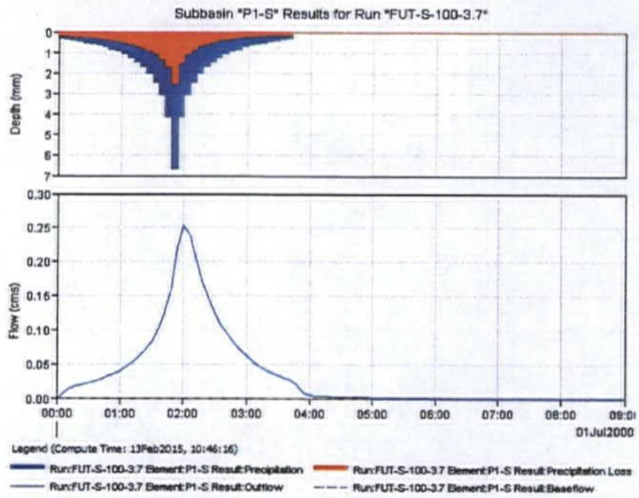
Elements of HEC-HMS Model : Future Condition Phase 1 Q_{100}

Figure 17

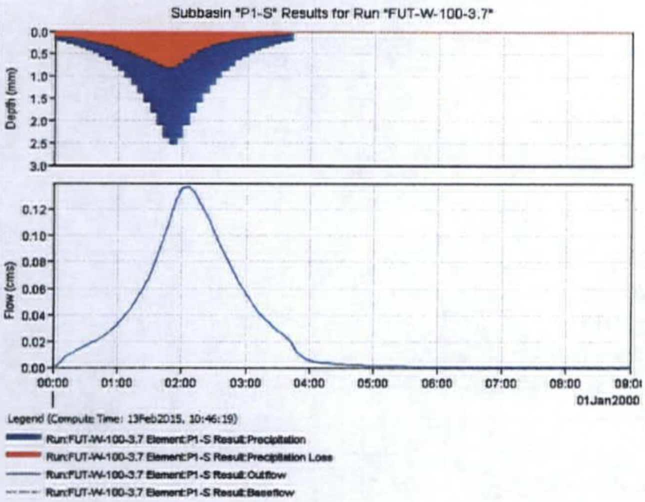
A RURAL SUB-BASIN RUNOFF : TP-N WINTER



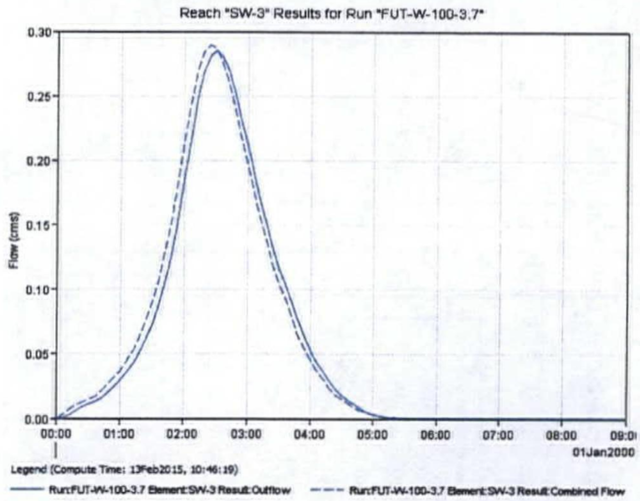
B POLYTUNNEL SUB-BASIN RUNOFF : P1-S SUMMER



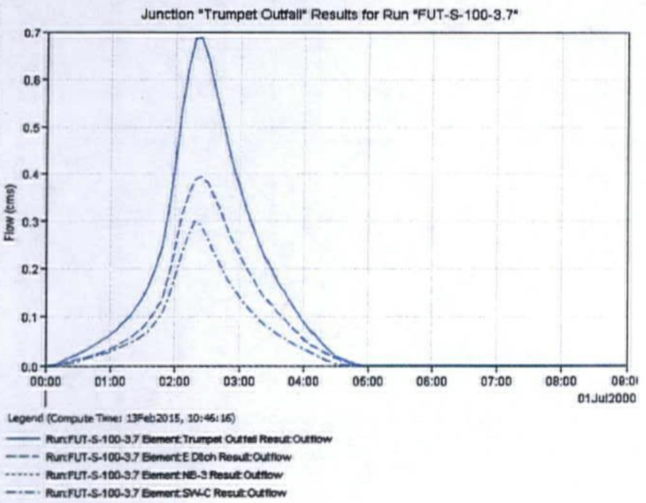
C POLYTUNNEL SUB-BASIN RUNOFF : P1-S WINTER



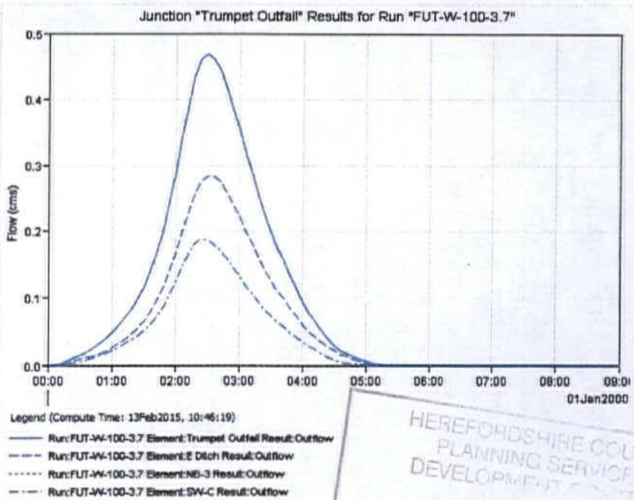
D CHANNEL ROUTING : SW-3 WINTER



E FLOW AT TRUMPET OUTFALL : SUMMER

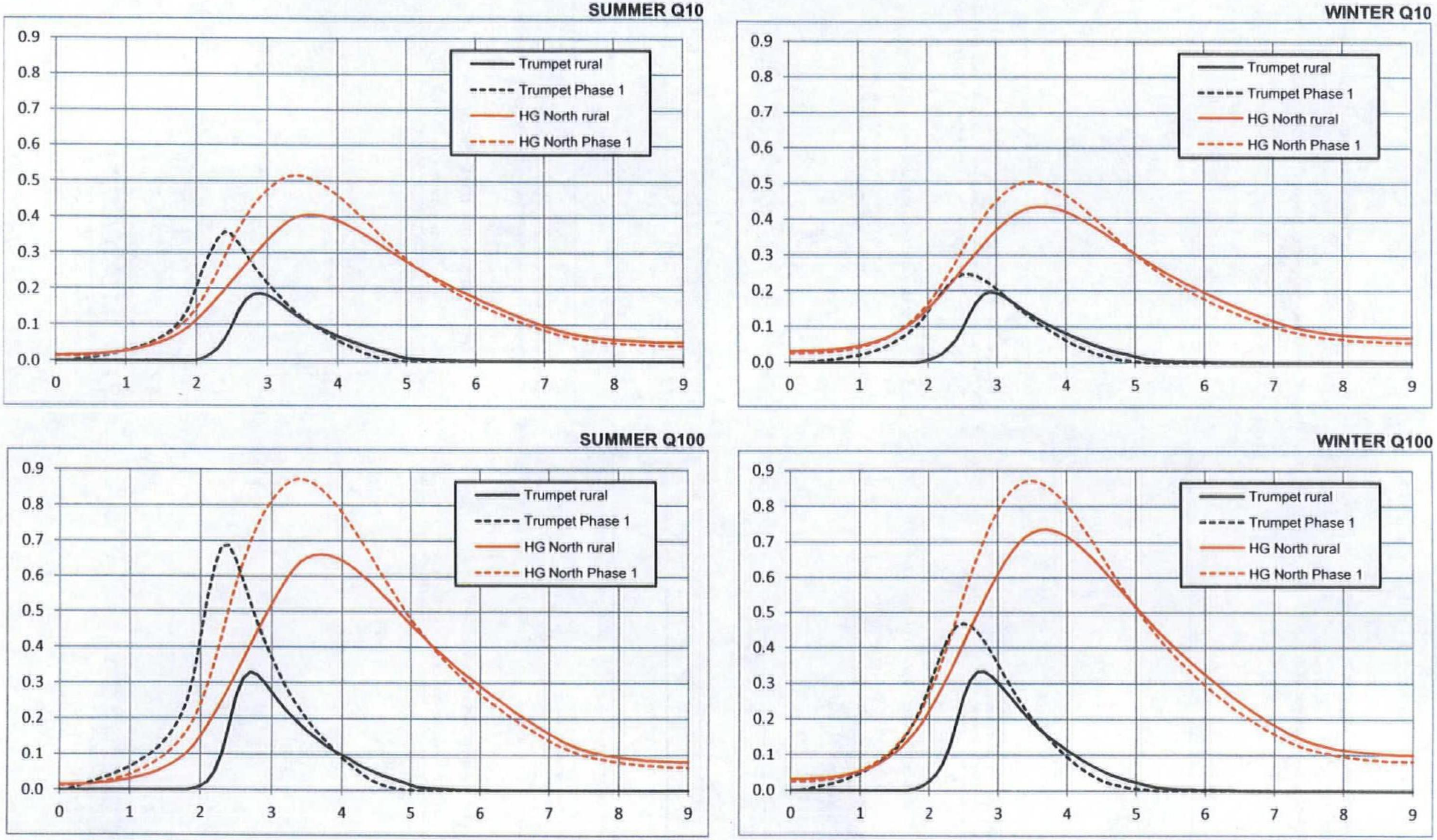


F FLOW AT TRUMPET OUTFALL : WINTER



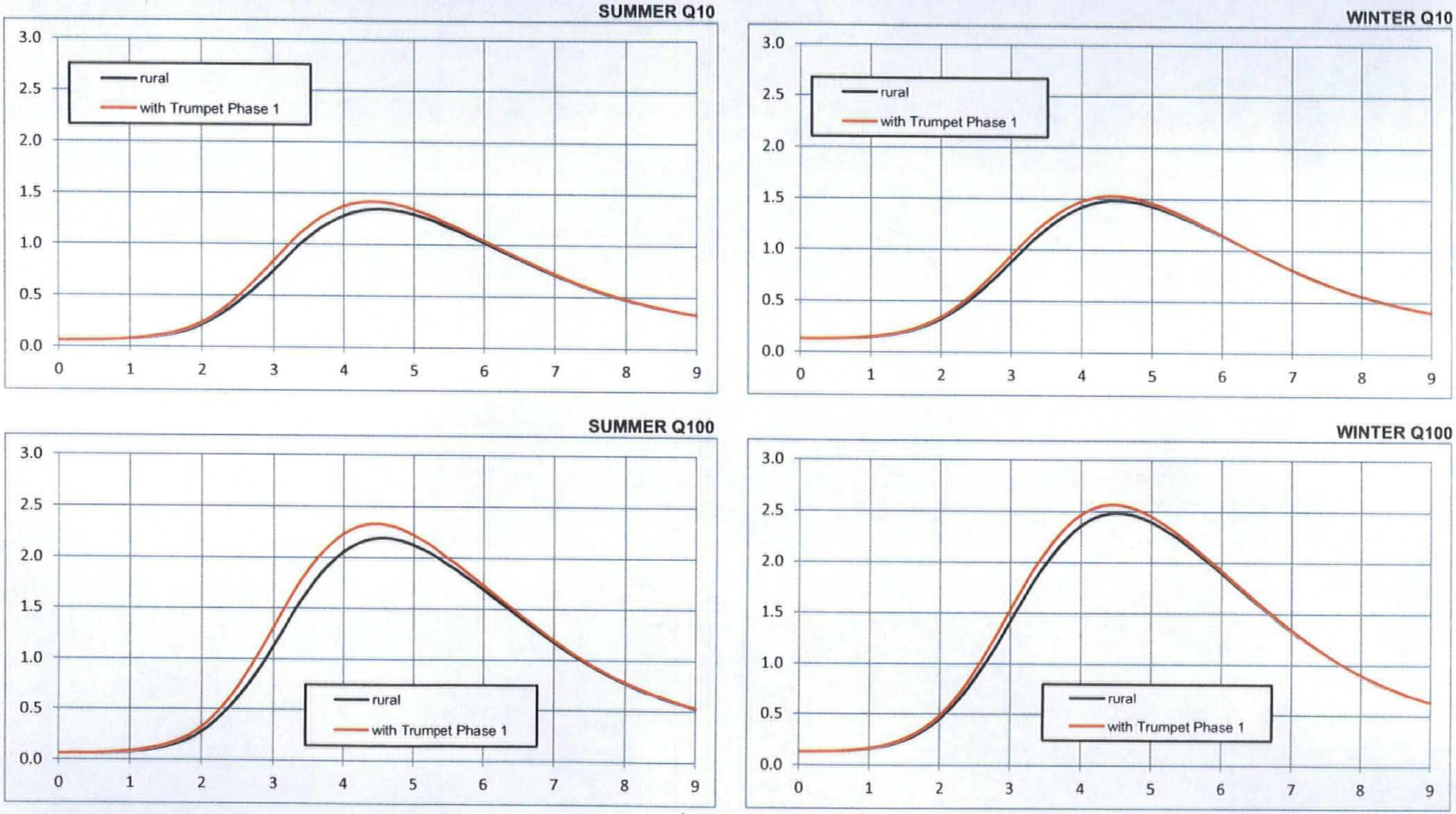
Impact of Phase 1 Development at Trumpet Outfall and HG North Catchment

Figure 18

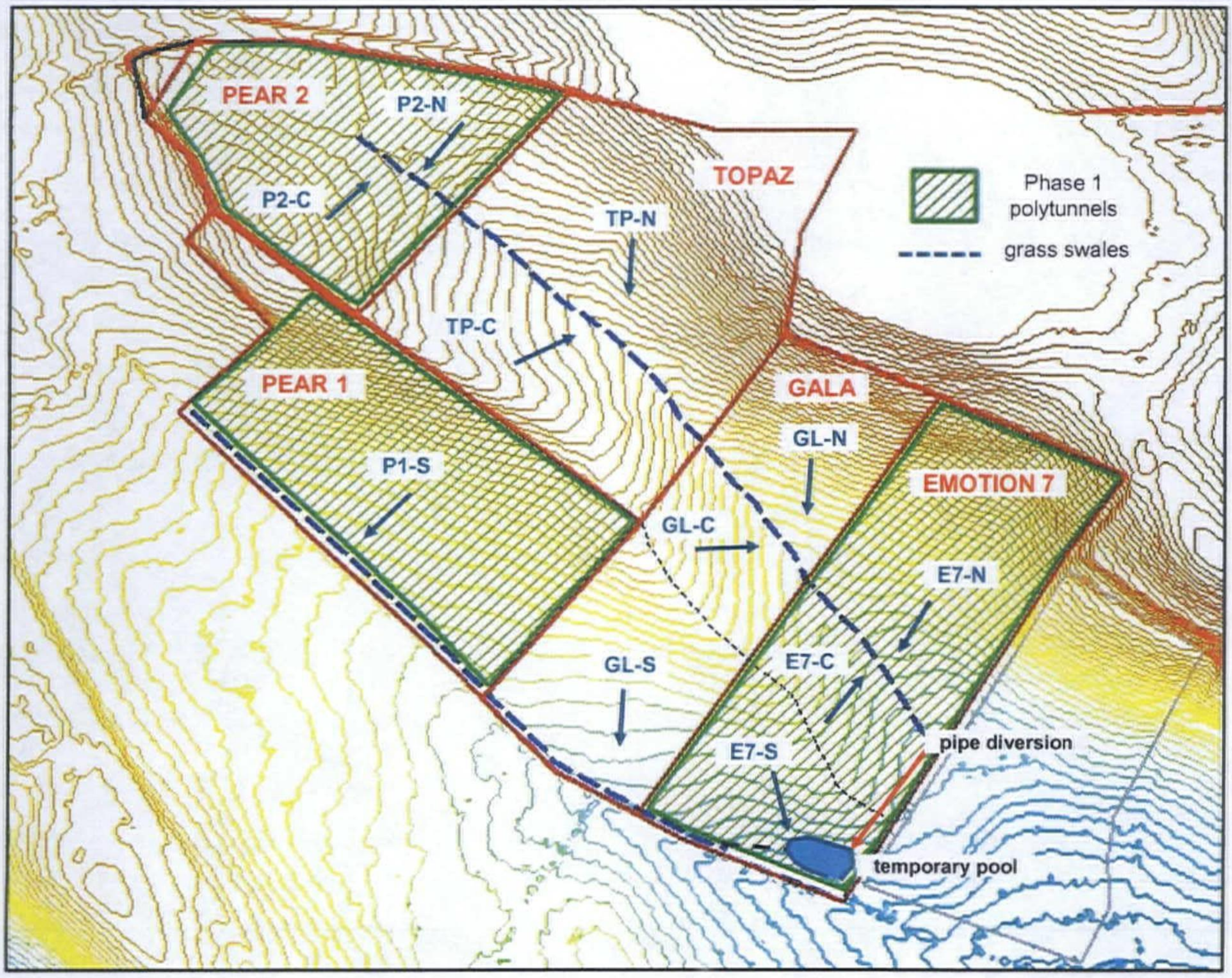


Impact of Trumpet Phase 1 Development at Priors Court

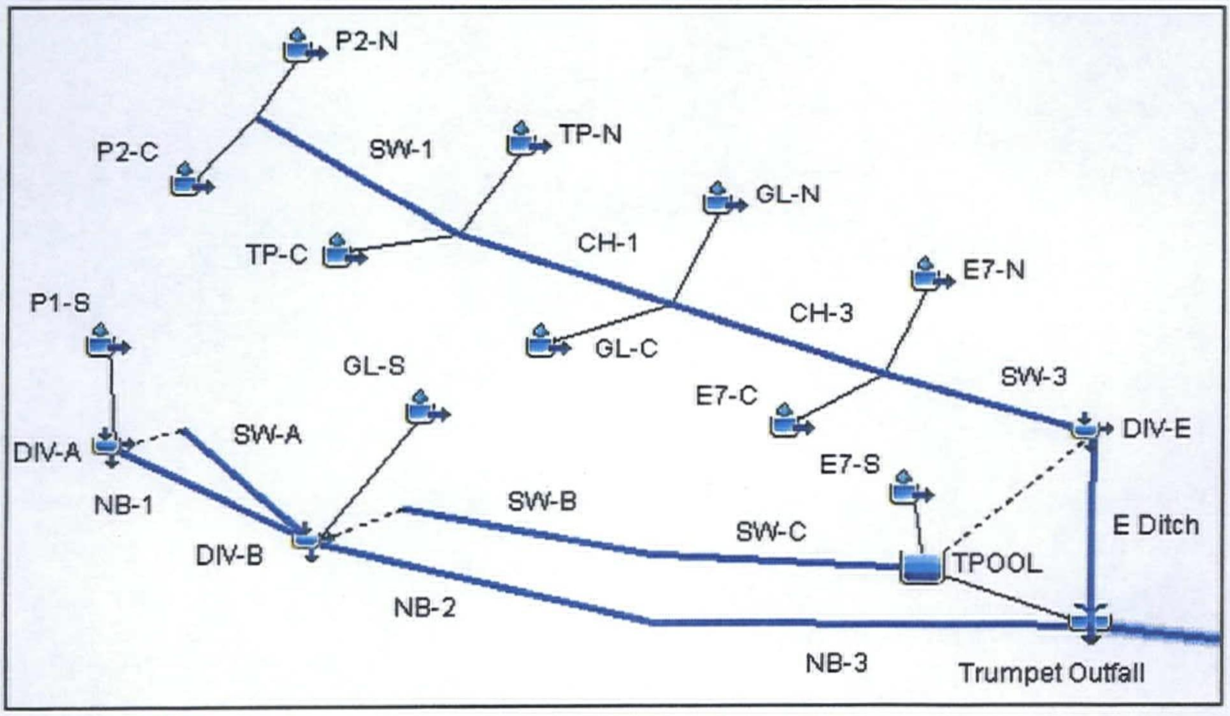
Figure 19



Phase 1P Development with Temporary Pool **Figure 20**



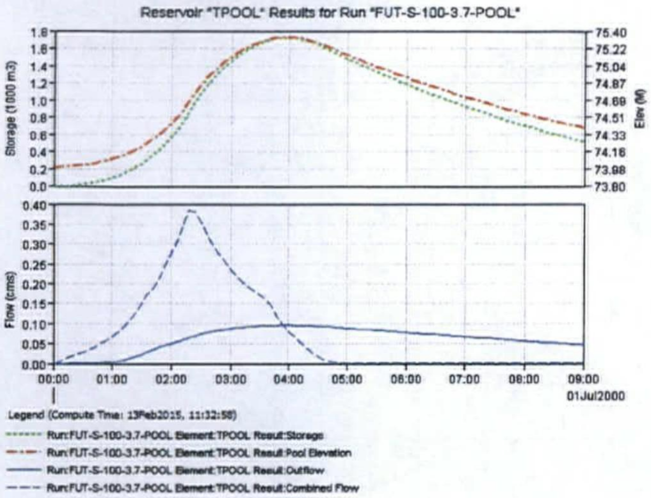
HEC-HMS Model Schematic : Phase 1P (with temporary Pool) **Figure 21**



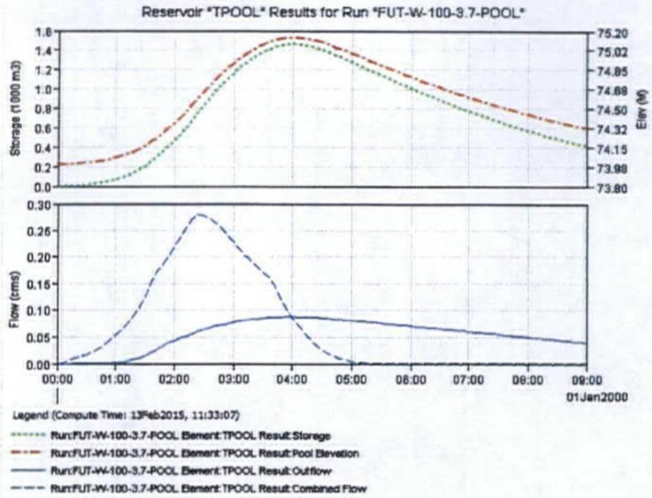
Elements of HEC-HMS Model : Future Condition Phase 1P

Figure 22

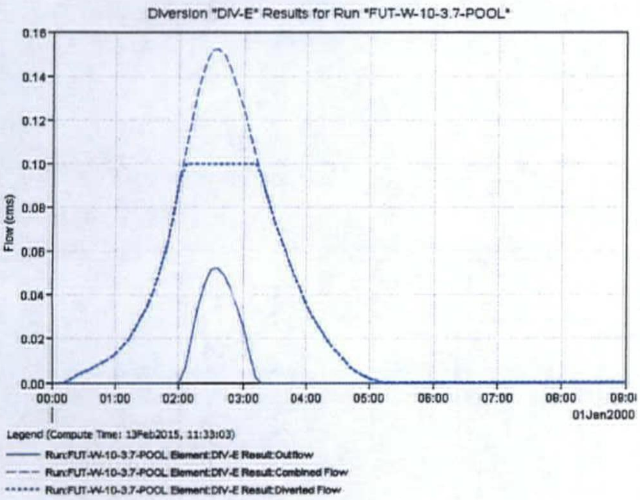
A **POOL OPERATION : Q100 SUMMER**



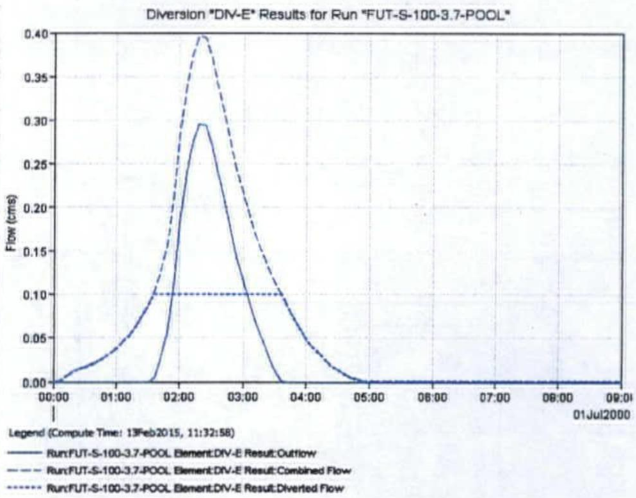
B **POOL OPERATION : Q100 WINTER**



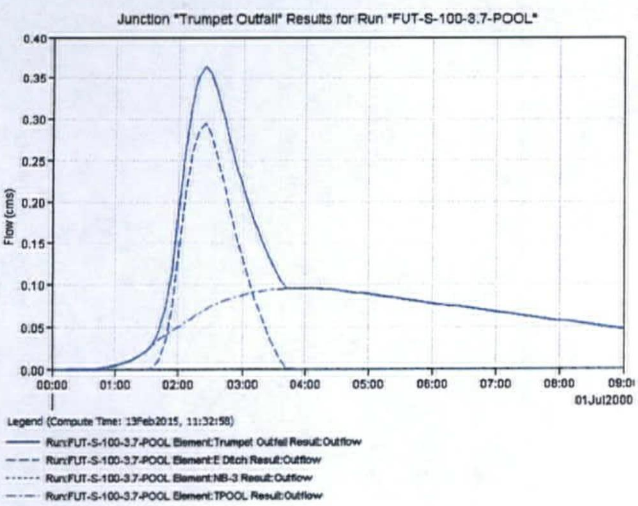
C **INFLOW TO POOL from SW-3 : Q10 WINTER**



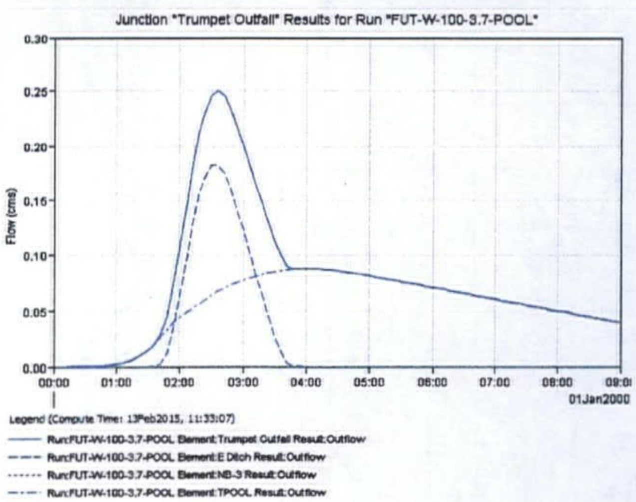
D **INFLOW TO POOL from SW-3 : Q100 SUMMER**



E **FLOW AT TRUMPET OUTFALL : Q100 SUMMER**

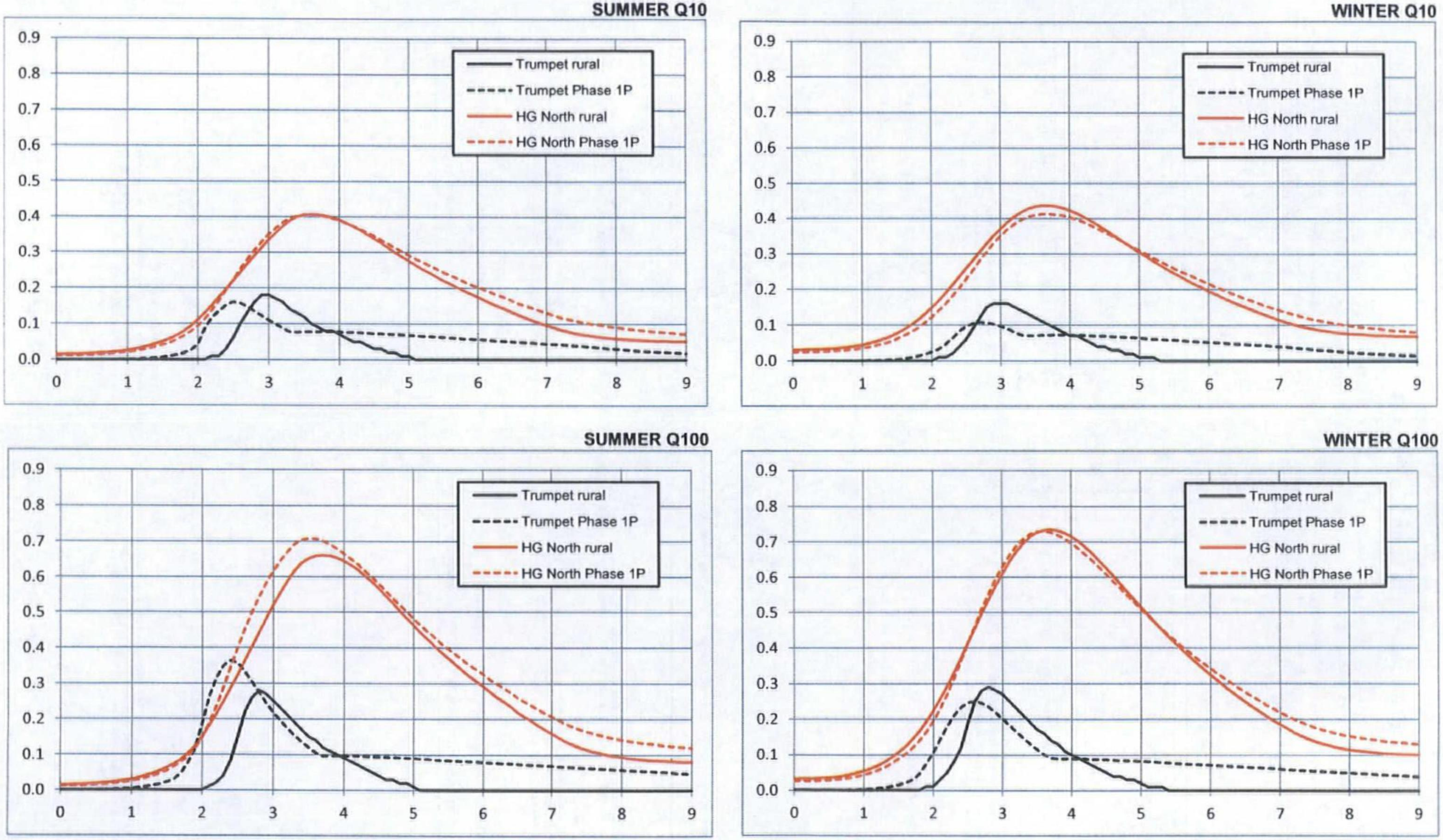


F **FLOW AT TRUMPET OUTFALL : Q100 WINTER**



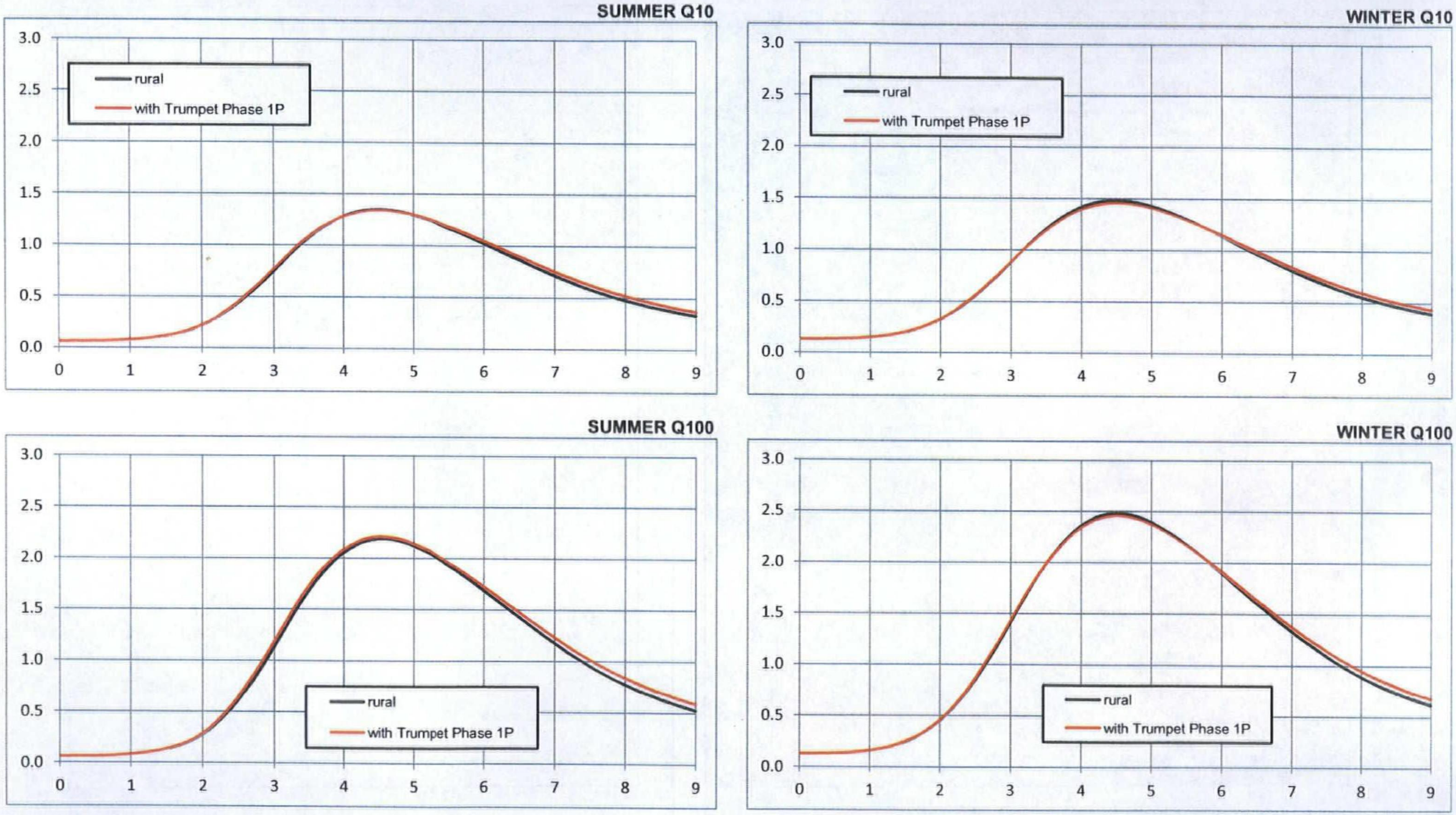
Impact of Phase 1P Development at Trumpet Outfall and HG North Catchment

Figure 23

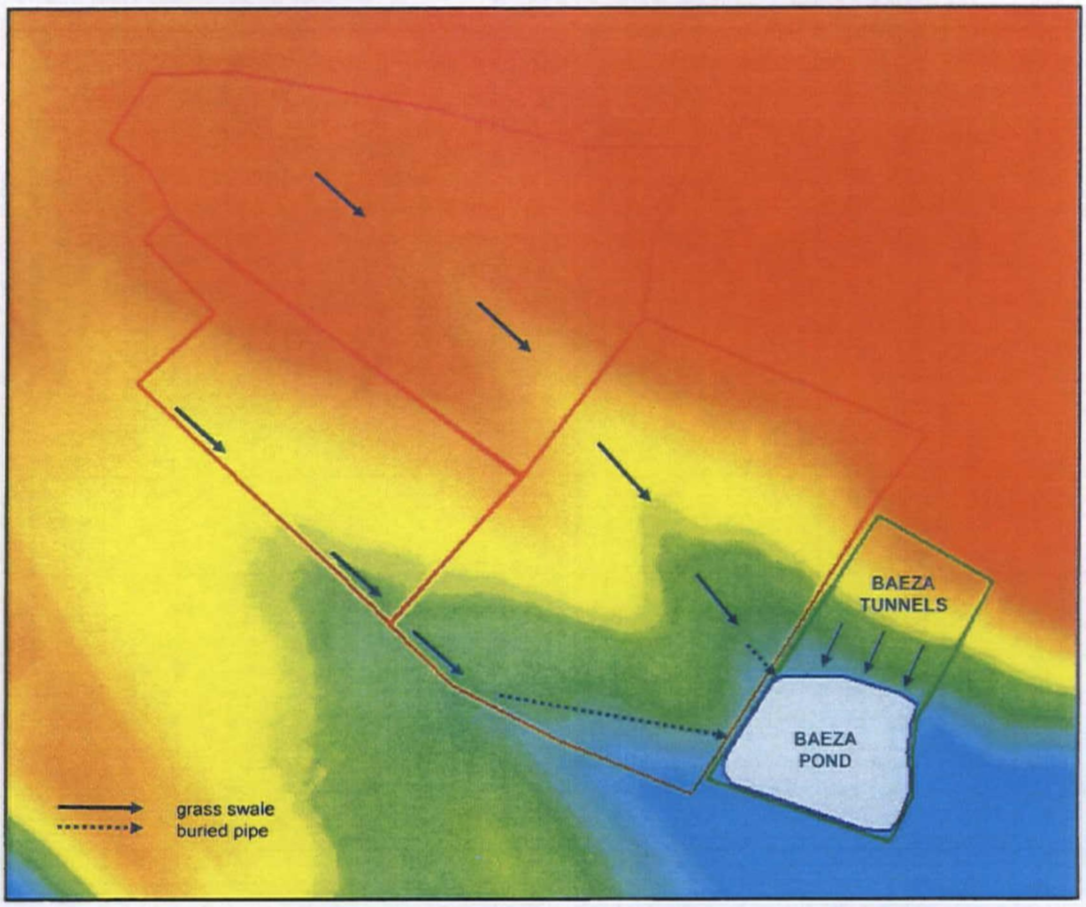


Impact of Trumpet Phase 1P Development at Priors Court

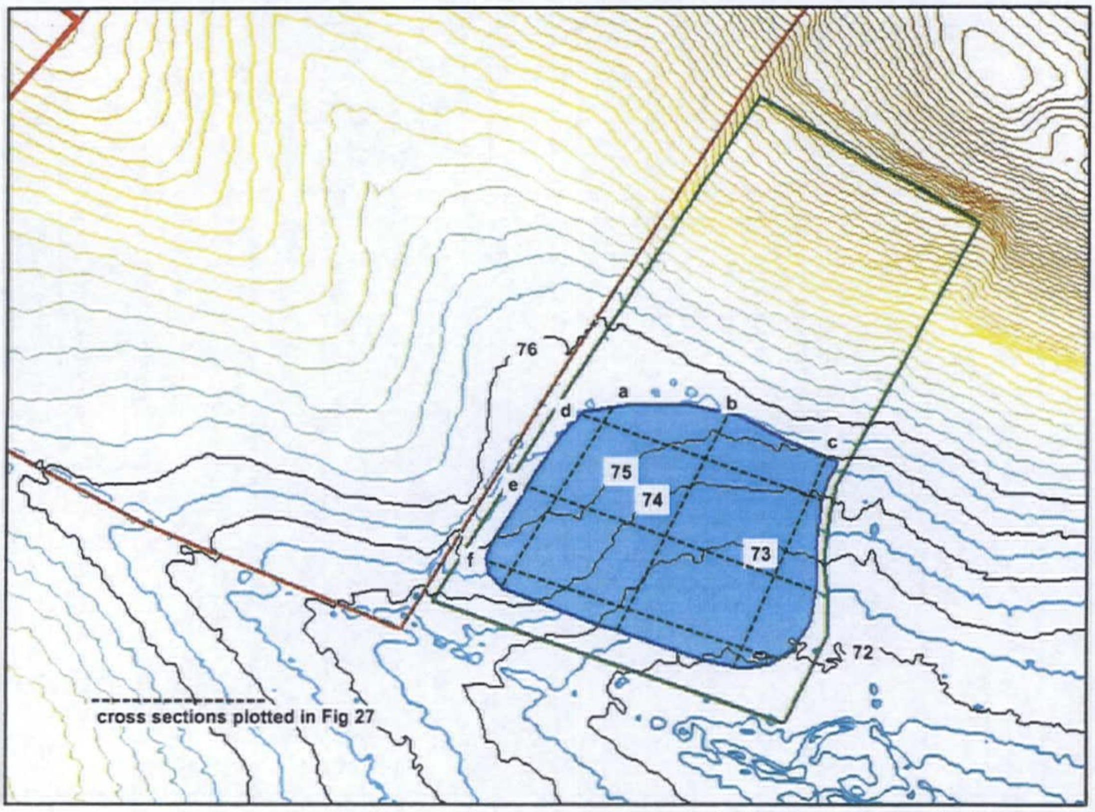
Figure 24



Trumpet Phase 2 Concept **Figure 25**



Baeza Pond **Figure 26**

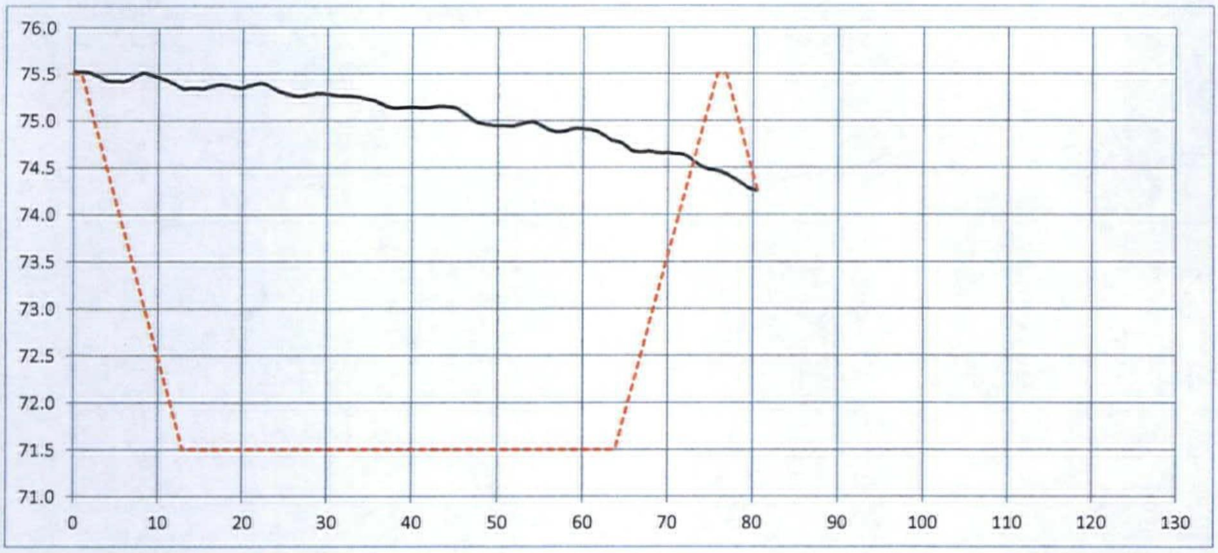


pond outline represents external embankment toe

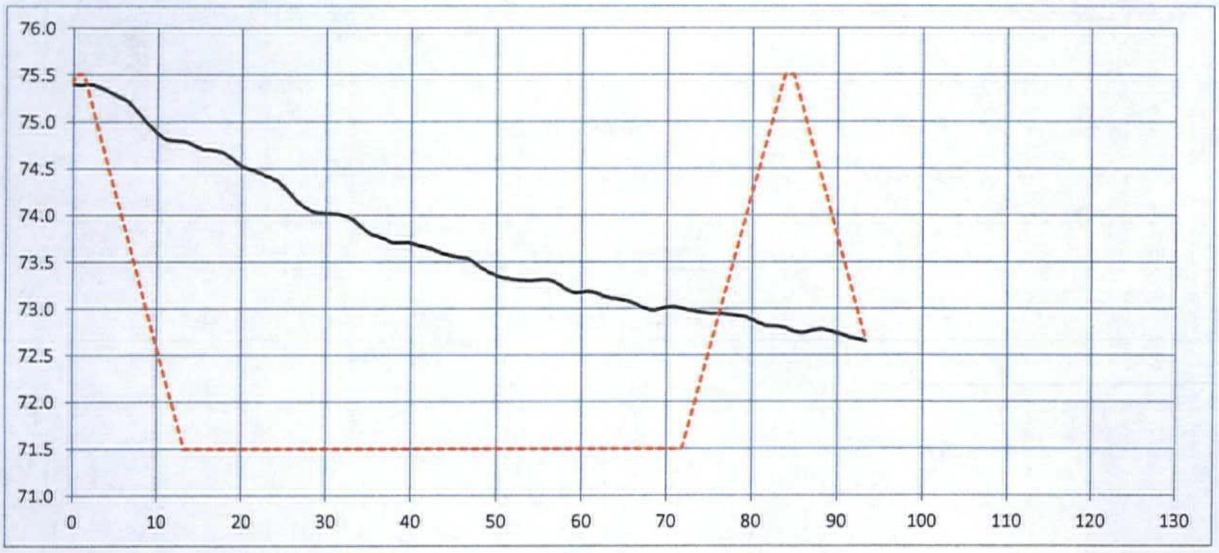
Baeza Pond Cross Sections

Figure 27

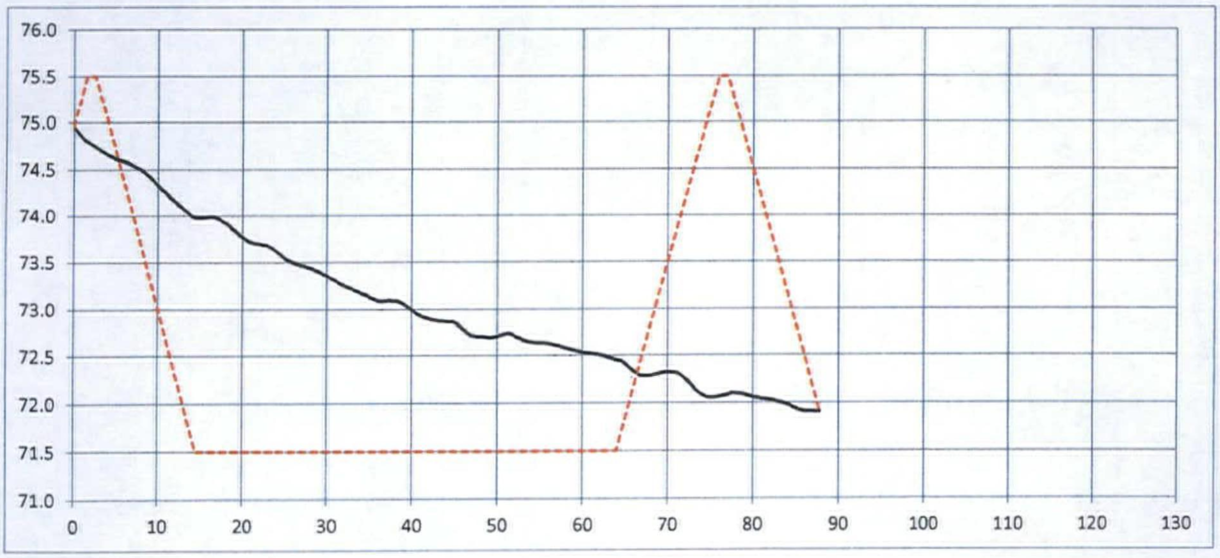
A



B



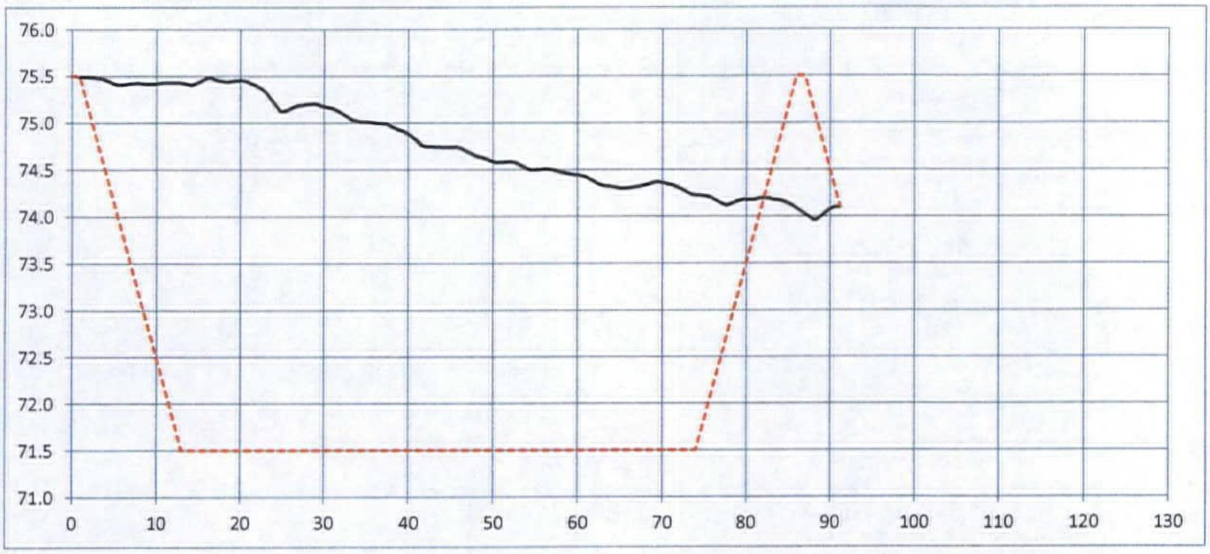
C



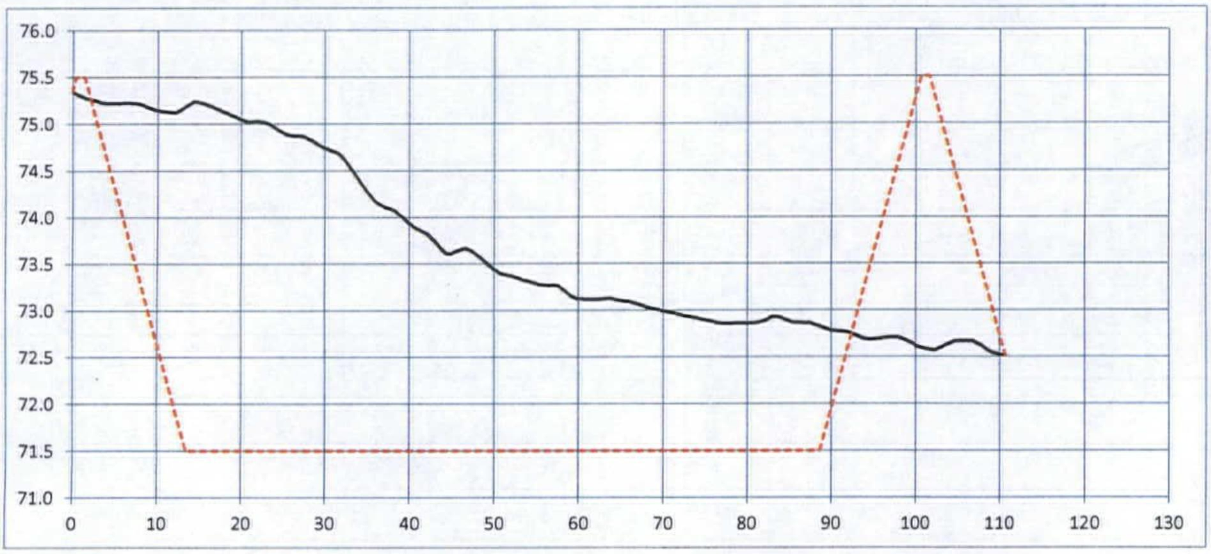
Baeza Pond Cross Sections

Figure 27

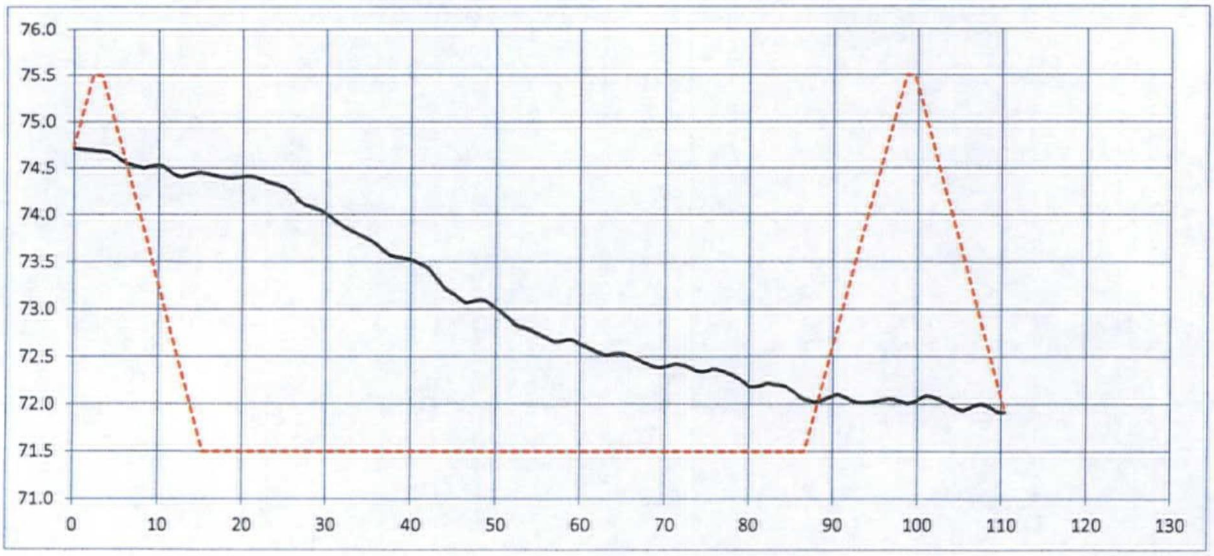
D



E

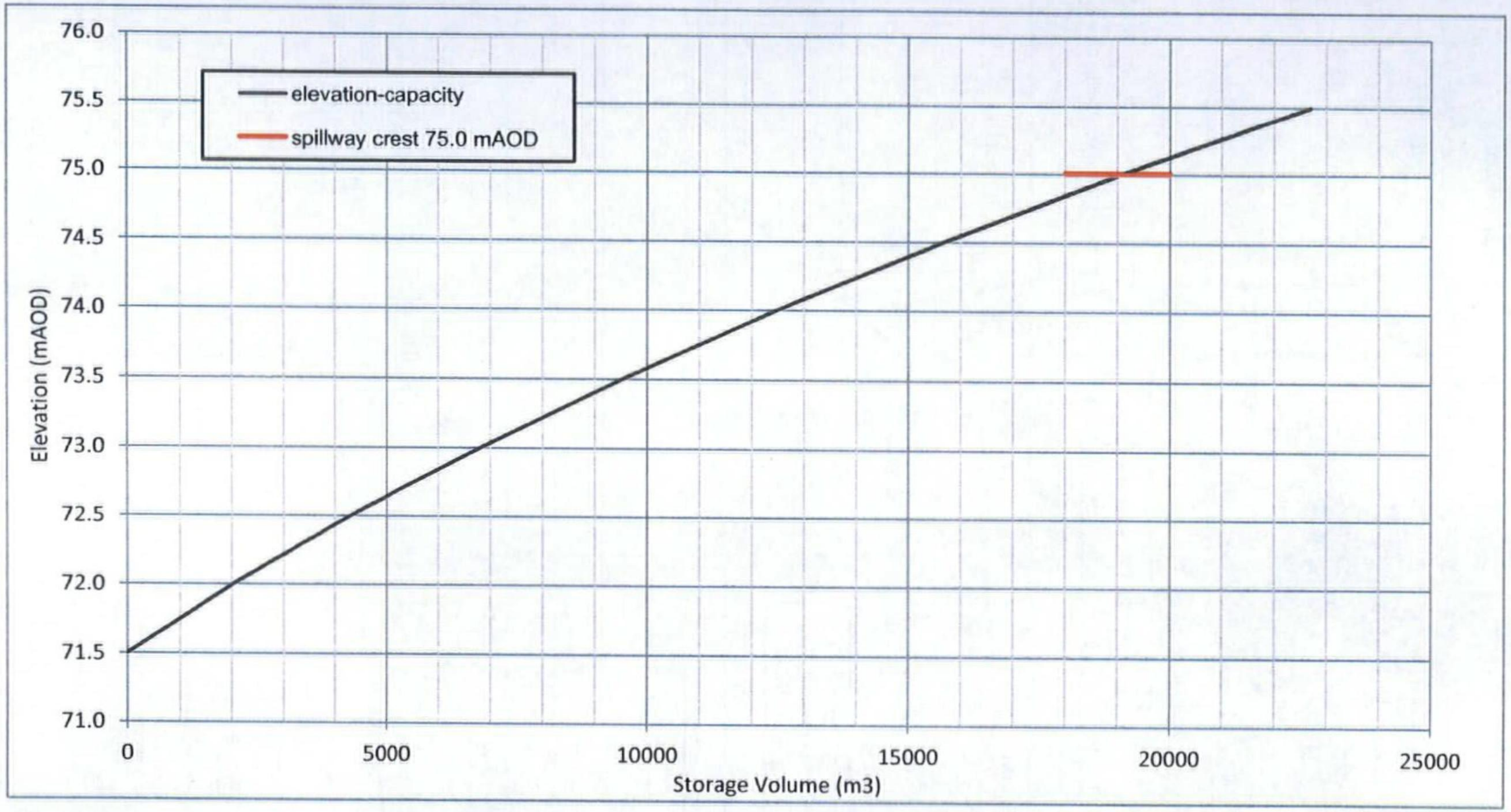


F



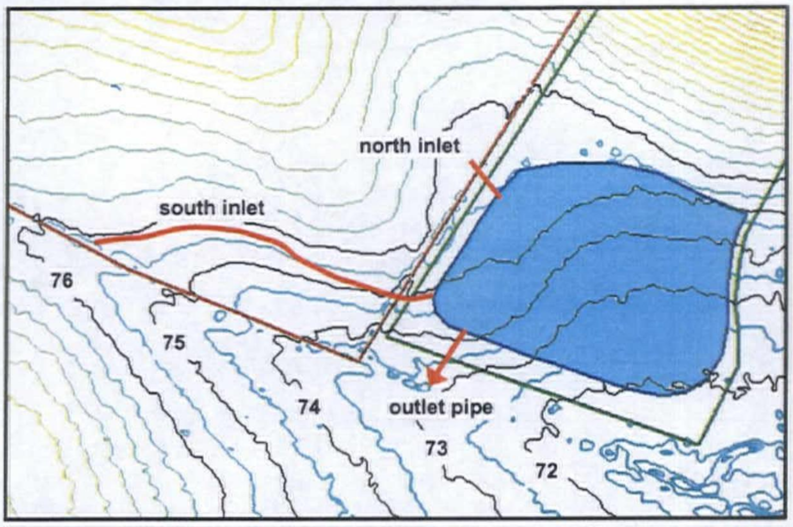
Baeza Pond Elevation-Capacity Curve

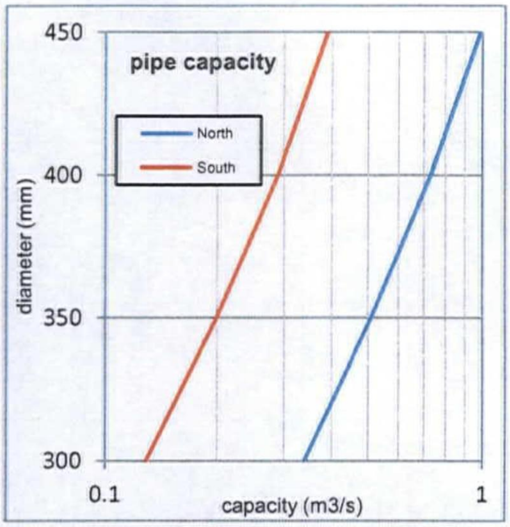
Figure 28

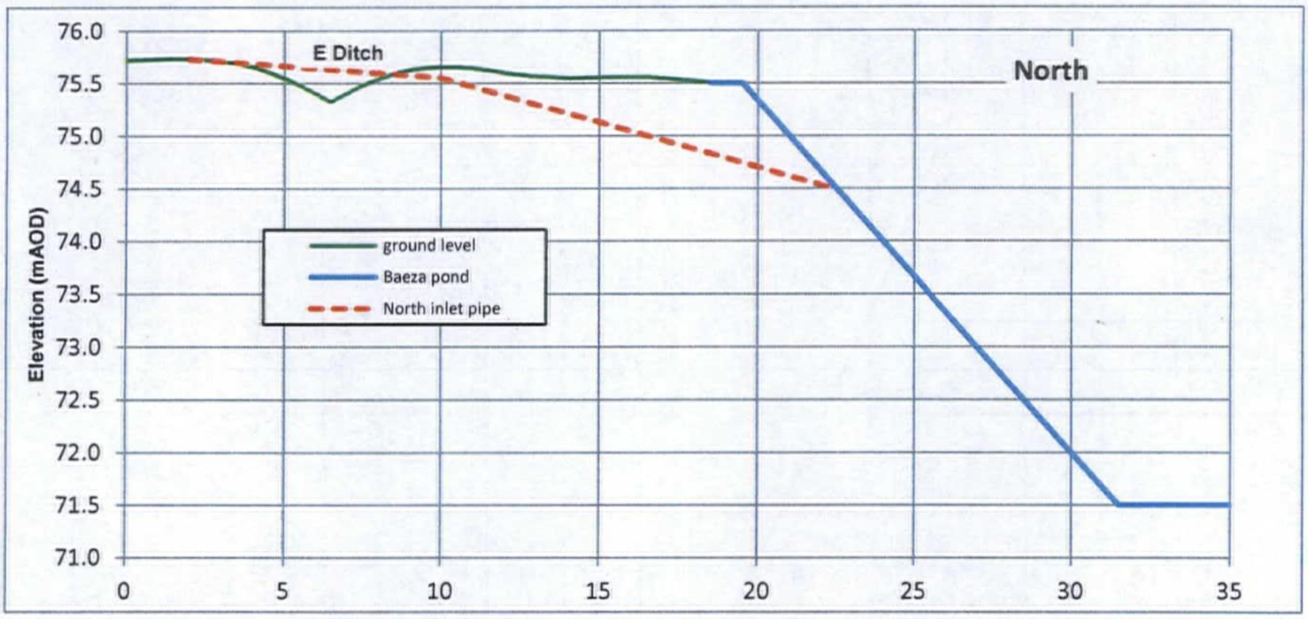


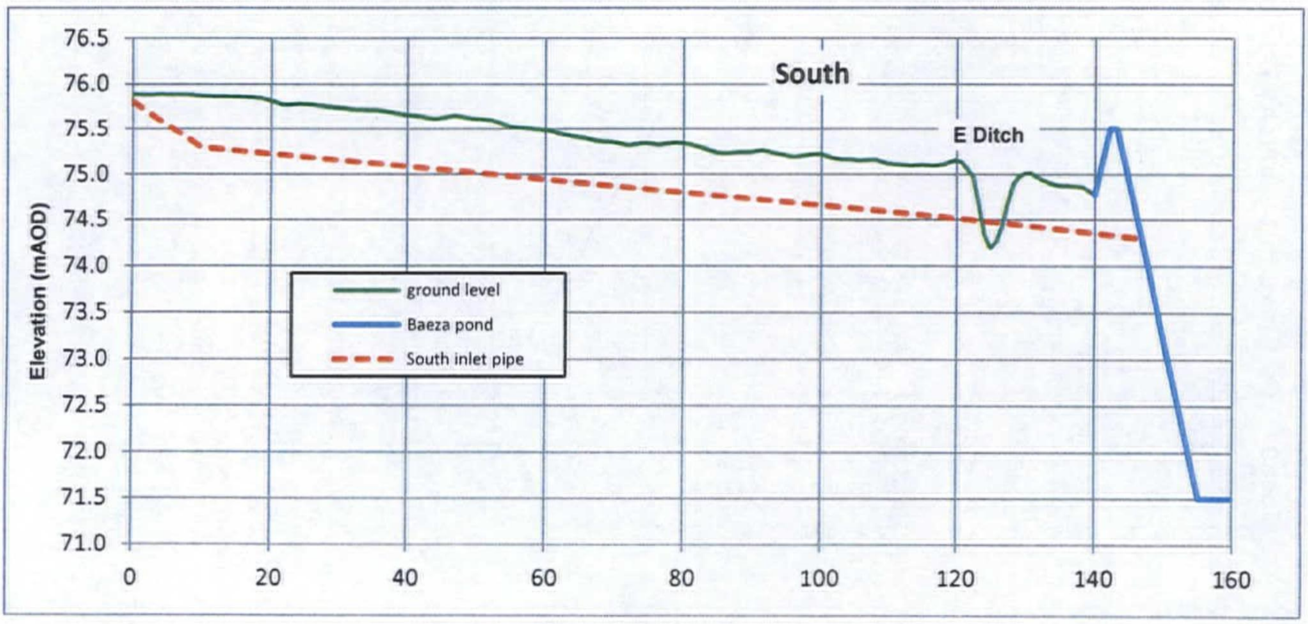
Baeza Pond Inlet Arrangements

Figure 29

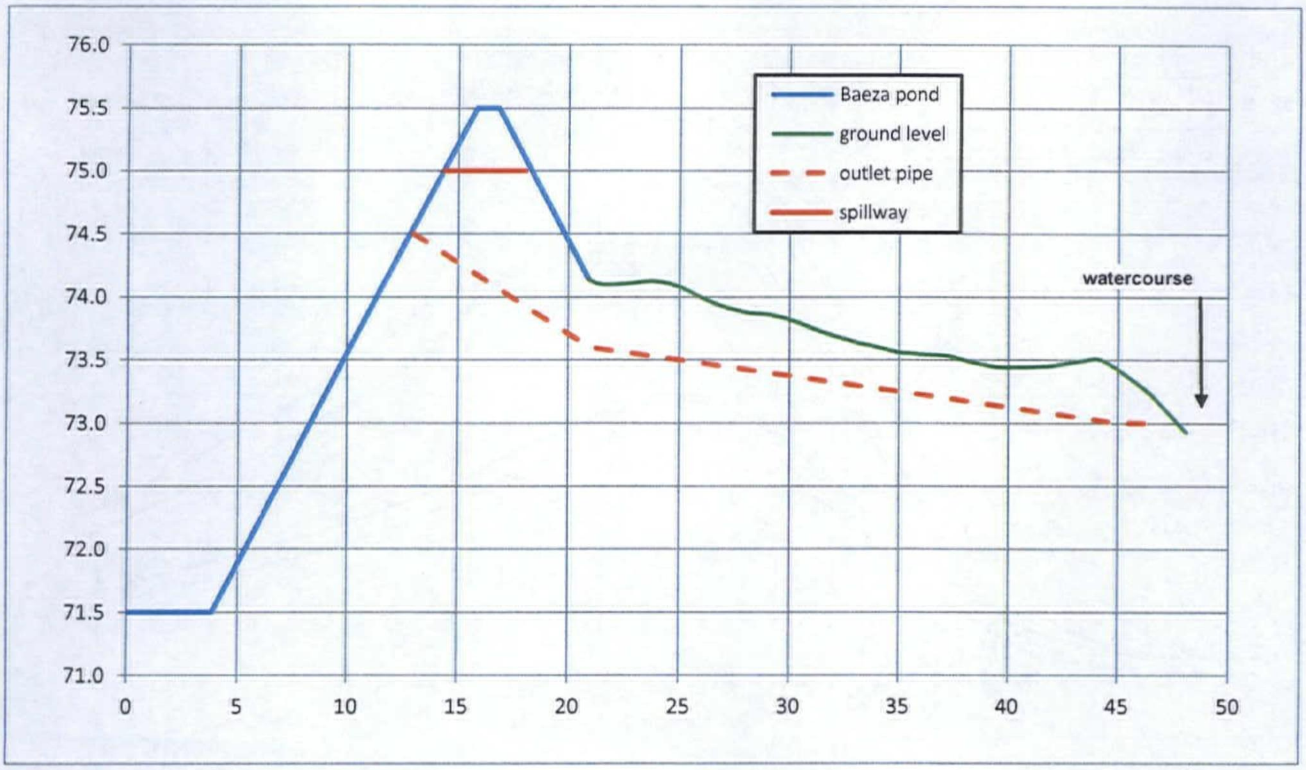




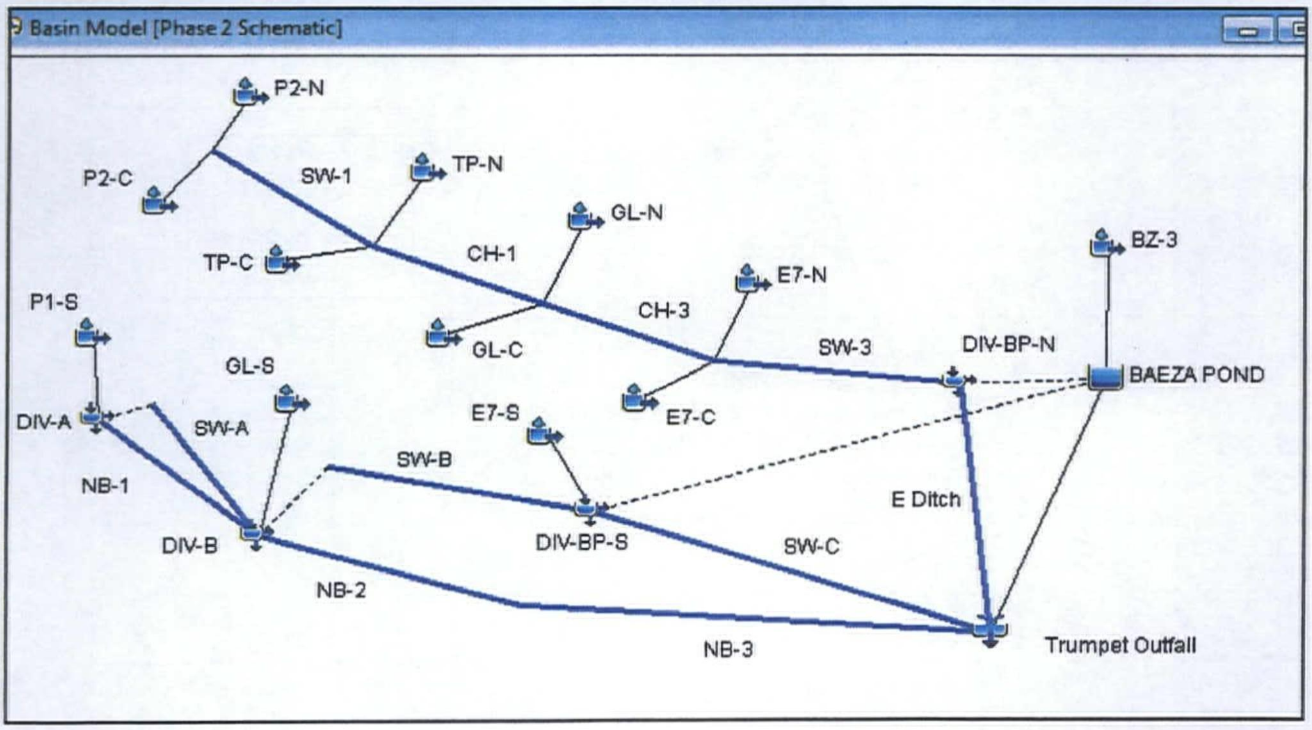




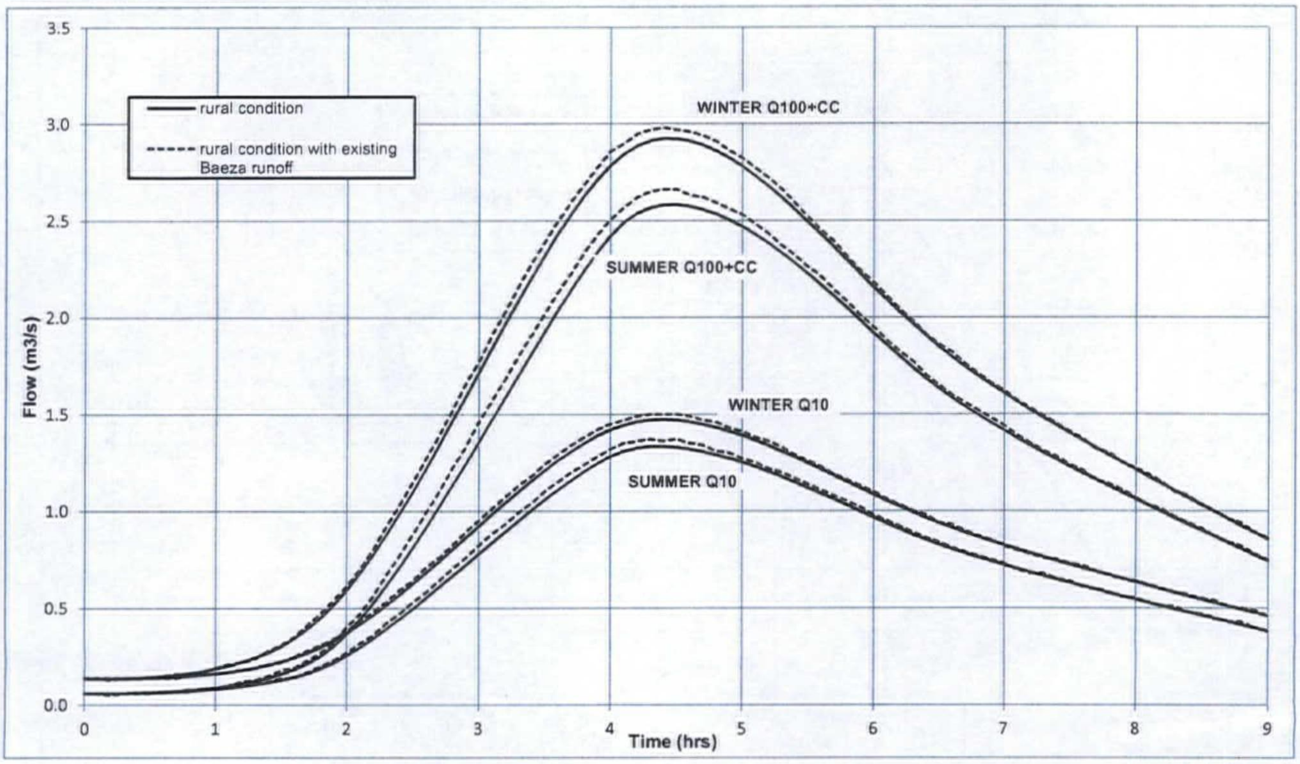
Baeza Pond Outlet Works **Figure 30**



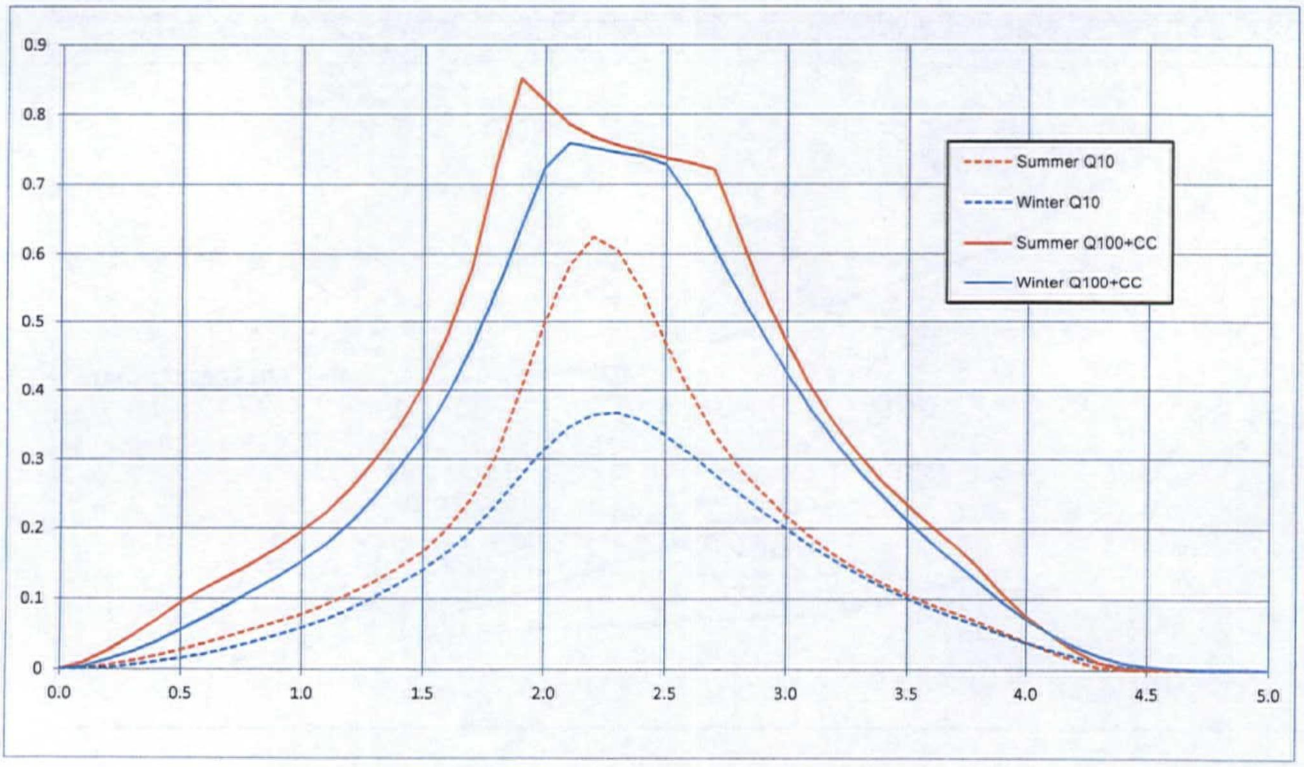
HEC-HMS Model Schematic : Phase 2 **Figure 31**



Priors Court Baseline Hydrographs for Phase 2 Model **Figure 32**



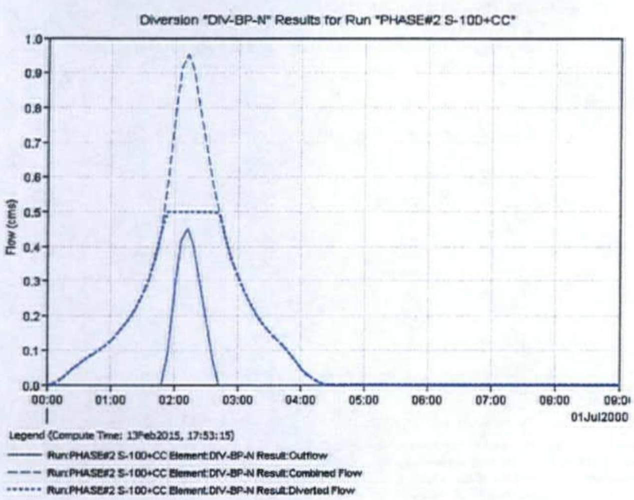
Baeza Pond Design Inflows **Figure 33**



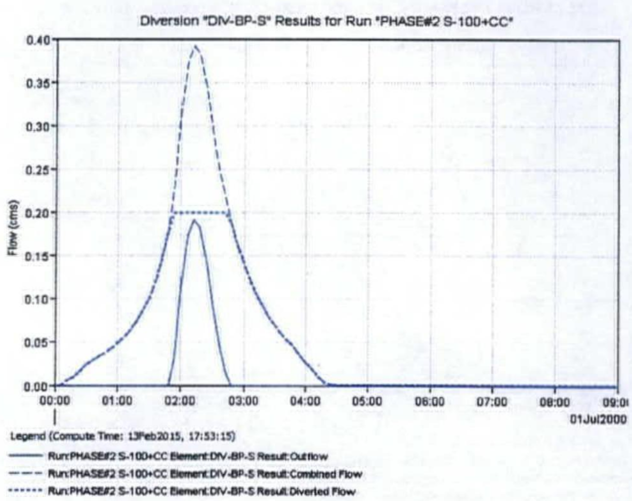
Elements of HEC-HMS Model : Future Condition Phase 2

Figure 34

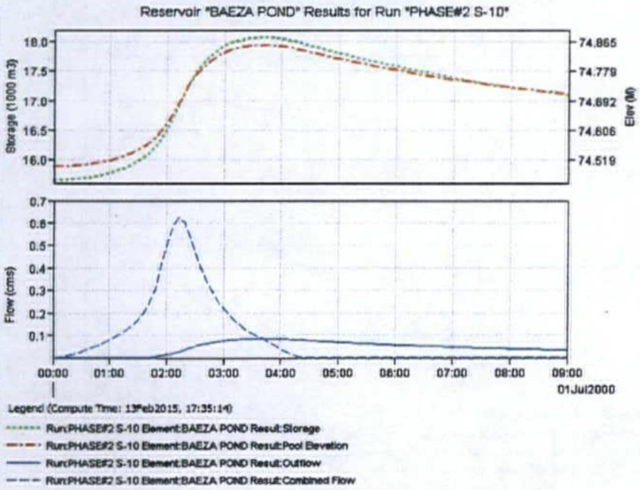
A BAEZA POND NORTHERN INLET DIVERSION : SUMMER Q100+CC



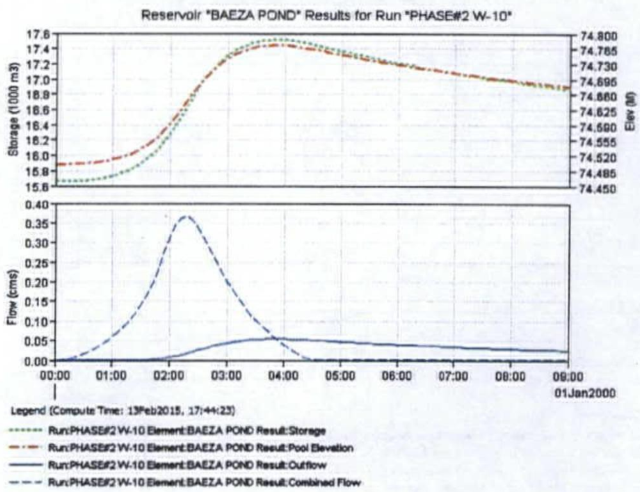
B BAEZA POND SOUTHERN INLET DIVERSION : SUMMER Q100+CC



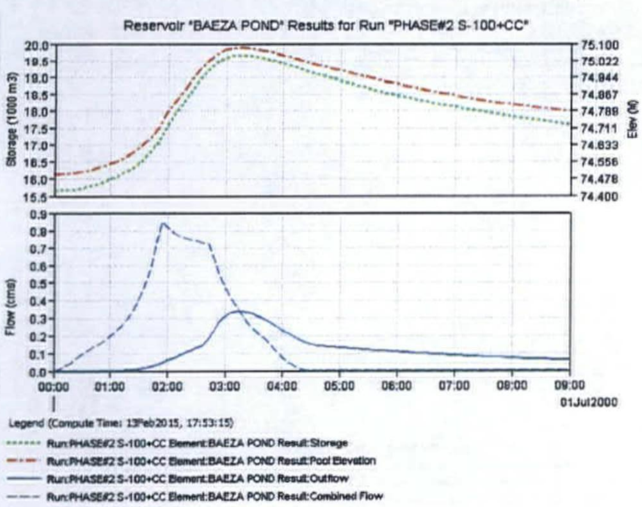
C BAEZA POND OPERATION : SUMMER Q10



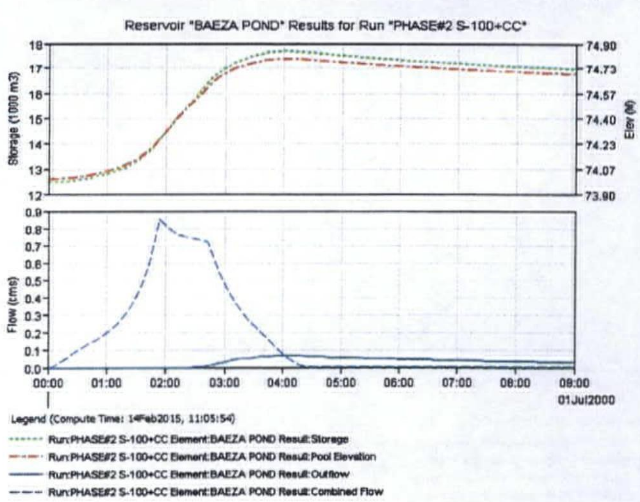
D BAEZA POND OPERATION : WINTER Q10



E BAEZA POND OPERATION : SUMMER Q100+CC (zero drawdown)



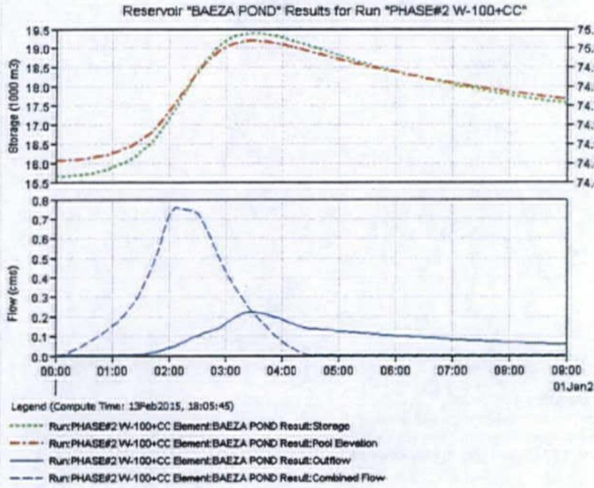
F BAEZA POND OPERATION : SUMMER Q100+CC (drawdown 0.5 m)



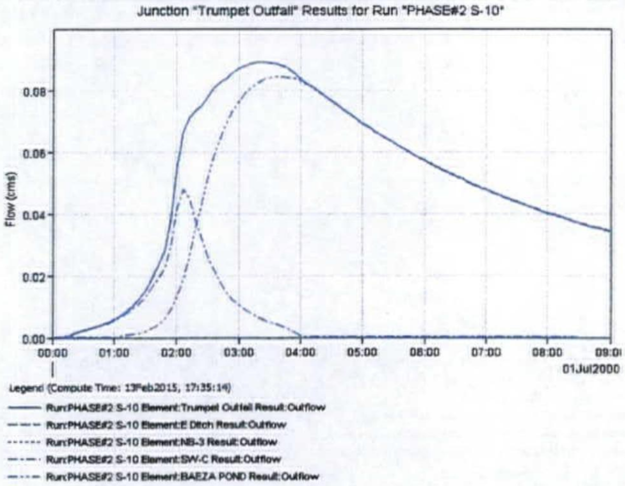
Elements of HEC-HMS Model : Future Condition Phase 2

Figure 34

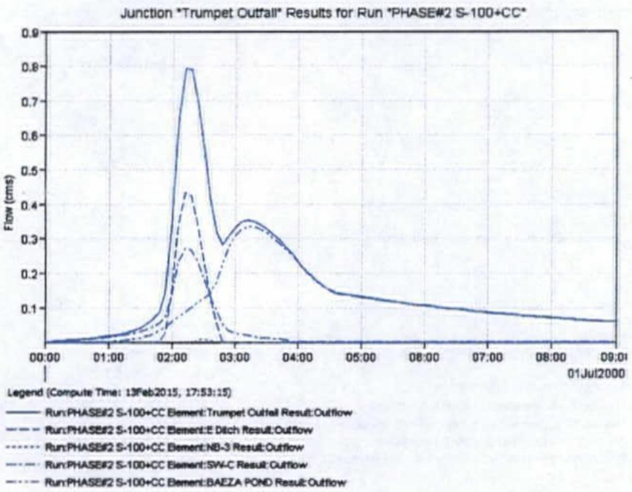
G **BAEZA POND OPERATION : WINTER Q100+CC (zero drawdown)**



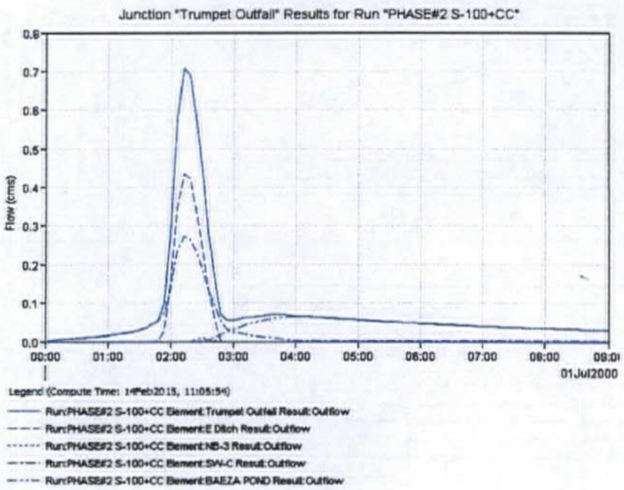
H **TRUMPET OUTFALL : SUMMER Q10**



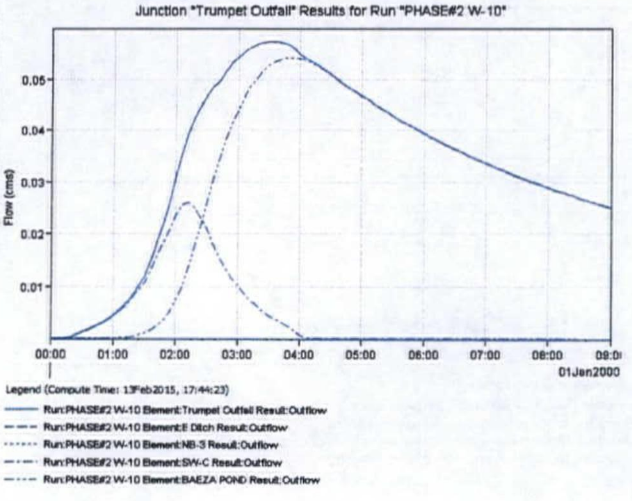
I **TRUMPET OUTFALL : SUMMER Q100+CC (zero drawdown)**



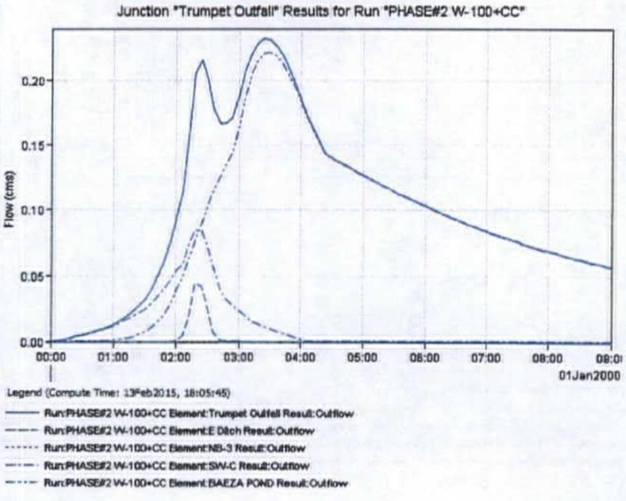
J **TRUMPET OUTFALL : SUMMER Q100+CC (drawdown 0.5 m)**



K **TRUMPET OUTFALL : WINTER Q10**

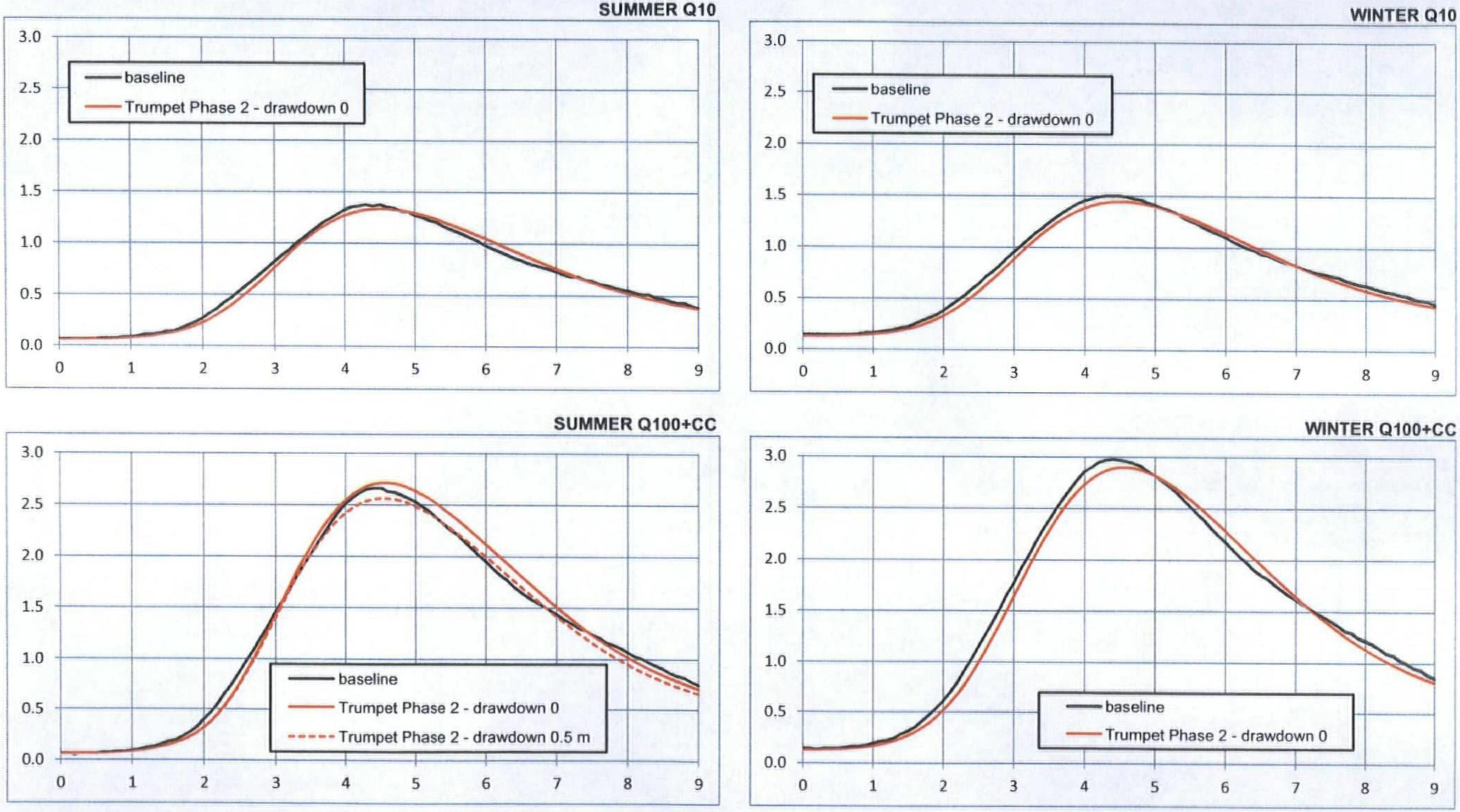


L **TRUMPET OUTFALL : WINTER Q100+CC**



Impact of Trumpet Phase 2 Development at Priors Court

Figure 35



Trumpet northern boundary **Plate 1**

view south down T3 **Plate 2**

view north up T3 **Plate 3**

Trumpet eastern boundary **Plate 4**

adjacent North Branch watercourse **Plate 5**

North Branch watercourse margin **Plate 6**

typical sleeper track **Plate 7**