Figure 2
Local Hydrology

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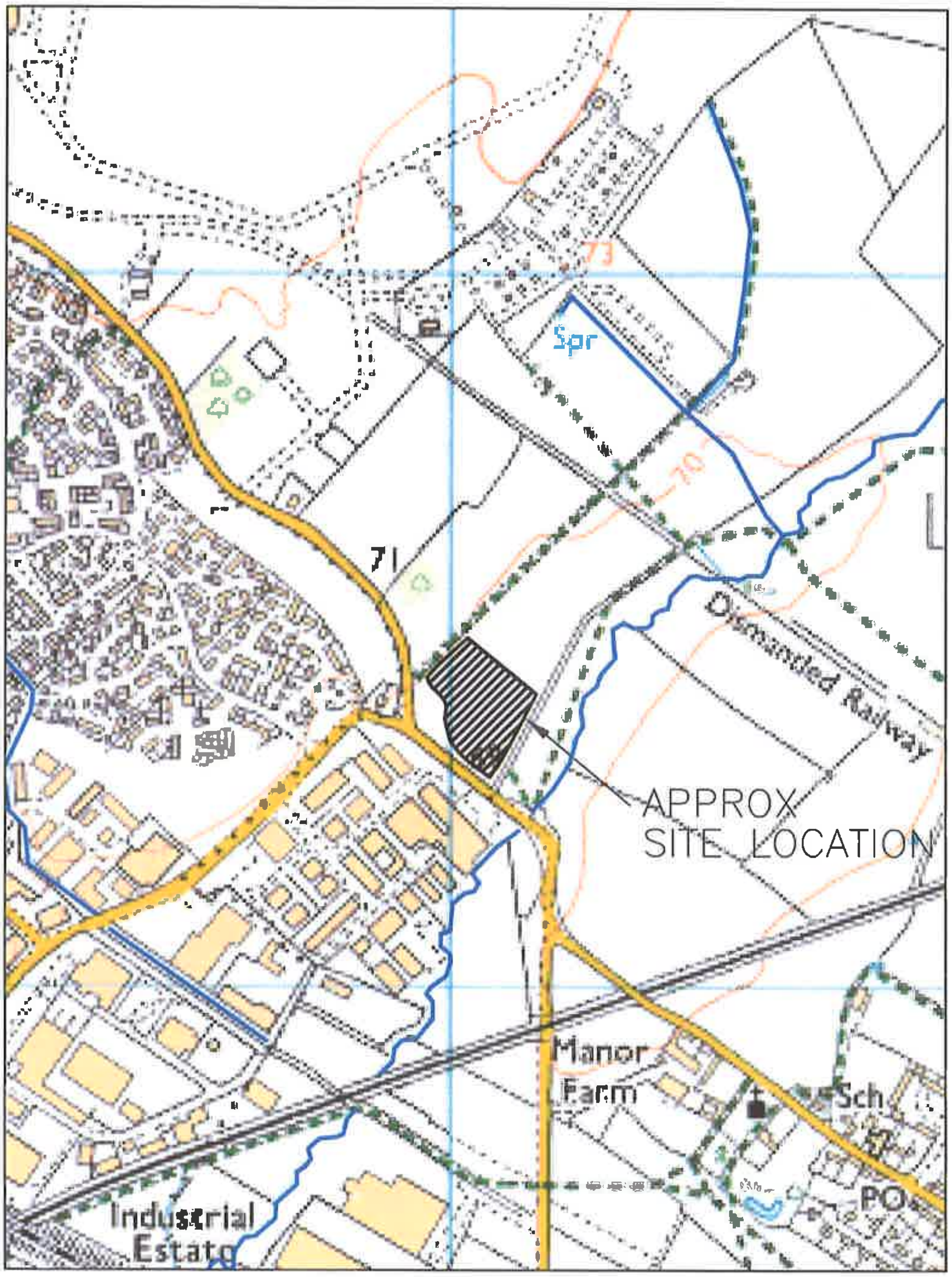

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Drawing Title:	LOCAL HYDROLOGY
Job Title:	LAND AT SKIMMINGDISH LANE BICESTER
Client:	OXFORD DIOCESAN BOARD OF FINANCE
Scale:	NOT TO SCALE
Drawn:	CBW
Date:	MARCH 2006
Checked:	
Drawing No:	Figure 2
Rev:	

Figure 3

Drawing 2006.2281.SK04
Indicative Surface Water Drainage Proposals

NOTES:

1 DO NOT SCALE FROM THIS DRAWING

2. SURFACE WATER PROPOSALS INDICATIVE ONLY

KEY:

- AREA AVAILABLE FOR POROUS PAVEMENT
- AREA AVAILABLE FOR PONDS
- ROOF AREA

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Job Title:	LAND AT SKIMMINGDISH LANE BICESTER
Client:	OXFORD DIOCESAN BOARD OF FINANCE
Drawing Title:	INDICATIVE SURFACE WATER DRAINAGE PROPOSALS
Scale:	1:1000 @ A3
Drawn:	CBW
Date:	MAR 2006
Checked:	
Drawing No:	2006.2281.SK04
Rev:	

Appendix A

Environment Agency Correspondence

TELECON RECORD

Project:	Skimmingdish Lane, Bicester	**Job No:**	2281
From: **Company:** **Tel :**	Clare Williams SMA 01635 867711	**To:** **Company:** **Tel :**	Mike Robinson Environment Agency 01491 828483
Date:	25 October 2005	**Time:**	16.45
SUBJECT:	**Flood Risk Assessment**		

Distribution: File, CW, Susanna Bedford

Message:	**Action:**
CW telephoned to discuss the Environment Agency's continuing objection to the proposed development at Skimmingdish Lane. MR explained that the EA concern was that Flood Zones represent the natural Floodplain without the presence of structures such as road embankments and flood defences. Therefore for this site there may be an issue with the culvert under the main road creating a backwater effect and causing flooding to the proposed development. MR stated that an assessment of the 100 year and 100 year plus 20% flows should be undertaken. These flows could then be compared to the capacity of the culvert to see if the flows can pass through unimpeded. CW explained that topographical survey of the culvert and road section would be undertaken and a Flood Estimation handbook assessment of the flood flows used to determine the Flood level at the bridge structure. However there would not be a full hydraulic model undertaken for this site. MR agreed that the approach set out above would be acceptable to the Agency. If the culvert can convey the flows then it is likely that the floodplain does correspond with the 'Natural' outline represented by the Flood Zones, and submitted FRA could be accepted. MR stated that for the Surface water drainage they will require further work. They were looking for the developer to provide some form of water quality enhancement such as an above ground pond or some form of swale system to serve the car parking. CW explained that this is for an outline application and that the eventual developers of the site would have control over the systems in place, the FRA shows that there is sufficient space on site to store the surface water runoff generated. MR noted that it was the water quality aspect of SUDS that need considering within the report.	

Our Ref : WA/2005/011612-3/1
Your Ref : 05/01563/OUT

ENVIRONMENT AGENCY

Date : 01 November 2005

Clare Williams
Stuart Michael Associates
Coombe House
Coombe Square
Thatcham
Berkshire
RG19 4JF

4 NOV 2005

Dear Sir/Madam

REVISED FLOOD RISK ASSESSMENT AND DRAINAGE STATEMENT FOR - B1 OFFICE DEVELOPMENT WITH ASSOCIATED PARKING, TURNING AND LANDSCAPING AREAS
LAND NORTH WEST OF LAUNTON ROAD, ROUNDABOUT ADJ SKIMMINGDISH LANE, CAVERSIFELD

FILE REF:SP62SW/16/4

Thank you for your letter dated 14 October 2005.

As stated in previous correspondence, the site lies partially within flood zone 3, the high risk 1 in 100 year floodplain.

Planning Policy Guidance Note 25 - "Development and Flood Risk" states that flood zones represent a natural floodplain and ignore the presence of structures and defences. There could be a significant backwater effect at the junction where the watercourse is culverted. An assessment should therefore be made on expected flows during a 1 in 100 year (and 1 in 100 year+20%) flood event and confirm the culvert does not cause a restriction to flood flows. If the culvert does cause a restriction the extent of flood zone 3 will increase.

With regards to surface water drainage, we need confirmation that land has been set aside based on the figures in your FRA for above ground storage. Oversized pipes and storage tanks are not sustainable and provide attenuation only. Sustainable drainage systems should be incorporated into the design to improve the quality of the water entering the local watercourse. Such measures can also offer other benefits in terms of promoting groundwater recharge and amenity enhancements. Approved Document Part H of the Building Regulations 2000 sets out a hierarchy for surface water disposal which encourages a SUDS approach.

Further information on SUDS can be found in PPG25 paragraphs 40-42, PPG25 appendix E, in the CIRIA C522 document Sustainable Urban Drainage Systems-design manual for England and Wales and the Interim Code of Practice for Sustainable Drainage Systems. The Interim Code of Practice provides advice on design, adoption and maintenance issues and a full overview of other technical guidance on SUDS. The Interim Code of Practice is available on both the Environment Agency's web site at: www.environment-agency.gov.uk and CIRIA's web site at www.ciria.org.uk

Therefore our objection on flood risk grounds remains, to discuss this issue in more detail please contact Sian Fairgrieve (Development Control Engineer) 01491 828678.

Yours faithfully

GParkhouse

GAIL PARKHOUSE
Planning Liaison Officer
gparkhouse@environment-agency.gov.uk

CC: Cherwell District Council

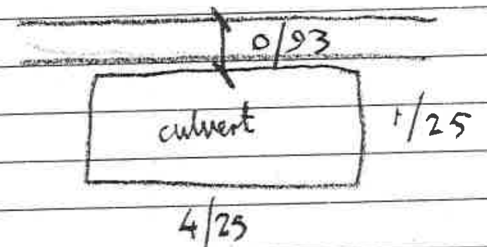
Appendix B
On-site Measurements

Date 11th February 2006 Levels taken for Skimmingdish Lane, Bicester

From MH (9350) C/L Telford Ind. Est. To Charbridge Lane

PATTERN B

BACK SIGHT	INTER-MEDIATE	FORE SIGHT	COLLIMATION OR H.P.C.	REDUCED LEVEL	DISTANCE	REMARKS
·72			71·412	69 540		MH 9350 C/L 69·540
	1·705			69·707	100 m	kb top (Lok) from c/L of culvert
	1·851			69·561	80 m	"
	1·939			69 473	60 m	"
	1·939			69·473	40 m	"
	1·874			69·538	20 m	"
	1·740			69·672	0 m	kb top c/L of culvert
	1·719			69·693		wall culvert D/S
	3·810			67·602		Bed " "
	1·640			69·772	0	kb " "
	1·610			69·802		wall " U/S
	3·585			67·827		bed " "
	1·530			69·882		island - top of kerb
	2·335			69·077	}	
	2·235			69·177	}	Indicative development low spot
	2·410			69·002	}	



Skimmingdish Lane Site Visit Photographs – 11 February 2006

Photograph 1: Looking downstream at Charbridge Lane Bridge

Photograph 2: Underside of Footbridge on downstream side of Charbridge Lane Bridge

Skimmingdish Lane Site Visit Photographs – 11 February 2006

Photograph 3: Looking upstream from Charbridge Lane Bridge

Photograph 4: Culvert under Charbridge Lane

Appendix C

HEC-RAS modelling Results

1.0 Introduction

1.1 The 100year flood flow for the Charbridge Brook was required so that a capacity check on a culvert under Skimmingdish Lane in Bicester could be undertaken.

1.2 As the catchment for this brook is small and rural with no gauging station upstream or downstream of the site, the rainfall runoff method was chosen to predict the 100year flood flow.

2.0 100 year flood estimation of the Charbridge Lane Brook using the Revitalised FSR/FEH Rainfall Runoff Method

2.1 The FSR/FEH rainfall runoff method is a widely used tool for design flood estimation in the United Kingdom. The method was first documented in the Flood Studies Report (FSR) in 1975 and since then numerous studies have updated and improved the method including the technical restatement of the method published in Volume 4 of the Flood Estimation Handbook (FEH) in 1999.

2.2 The latest revision, 'Revitalisation of the FSR/FEH rainfall runoff model', was undertaken to make updates and improvements to the key components of the FSR/FEH rainfall runoff model using new data, updated analytical techniques and advances in computation. These modifications include, improved rainfall runoff modelling techniques and a method for generating design flood events with added emphasis on quantifying the underlying physical flood generating mechanisms.

2.3 The development of the new physically based conceptual rainfall runoff model, the 'Revitalised Flood Hydrograph' (ReFH) model, is based on robust hydrological modelling techniques and is considered by DEFRA and the Environment Agency to be a significant improvement over the existing FSR/FEH model.

2.4 The ReFH model allows a more direct and transparent quantification of flood generating mechanisms, and the concept of seasonal variation in soil moisture content and design rainfall is introduced. Based on these results a set of equations was developed allowing users to estimate the model parameters for any catchment in the UK larger than $0.5km^2$.

2.5 Based on the ReFH model, a design method was developed which allows the development of design flood hydrographs through the specification of initial soil moisture content, design rainfall and required return period. Both soil moisture and rainfall are specified on a seasonal basis depending on the degree of urbanisation of the catchment (summer conditions for urbanised catchments and winter conditions for un-urbanised catchments).

2.7 Validation of this new design method confirmed that for most catchments is within ±10% of the peak flow obtained from statistical analysis of annual maxima peak flow data on the same catchment.

2.8 To support the dissemination of the results a spreadsheet implementation of the design method was developed by the Centre of Ecology and Hydrology and is attached at Appendix A.

3.0 100 year flood estimation of the Charbridge Lane Brook using Volume 4 of the Flood Estimation Handbook: Restatement and application of the FSR rainfall-runoff method

3.1 So that a comparison can be made between the differing results of the two hydrological models, the peak flow rate for the 100year event was also estimated using the methodology described in Volume 4 of the Flood Estimation Handbook.

Estimation of unit hydrograph and time to peak Tp (0)

3.1 The unit hydrograph time to peak is an index of catchment response time, i.e. the faster responding the catchment, the shorter the critical storm duration. Prediction of the unit hydrograph time to peak, (T_p), using catchment descriptors was calculated using the following equations given in FEH-Volume 4:

$$Tp\,(0) = 4.270 DPSBAR^{-0.35}\, PROPWET^{-0.80}\, DPLBAR^{0.54}\, (1+URBEXT)^{-5.77}$$

(Equation 2.10)

$$= 4.270(16.6)^{-0.35}\,(0.32)^{-0.80}\,(4.01)^{0.54}\,(1+0.010)^{-5.77}$$

$$= 8.0 \text{ hours}$$

10% to 20% of 8.0 hours is 0.8 - 1.6hours; therefore a 1.0 hour data interval is appropriate.

$$T_p\,(\Delta T) = T_p\,(0) + \Delta T/2$$

(Equation 2.40)

$$= 8.0 + ½$$

$$= 8.50 \text{ hours}$$

$$U_p = (2.2 / T_p)\ AREA$$

(Equation 2.60)

$$= (2.2 / 8.44) \times 14.23$$

$$= 3.7 m^3/s$$

$$T_B = 2.52\ T_p$$

(Equation 2.70)

$$= 2.52 \times 8.50$$

$$= 21.42 \text{ hours}$$

3.2 Using the computed values above the triangular unit hydrograph was constructed.

(Figure 1 refers)

Calculation of design storm duration, D

3.3 The design storm duration (D) is calculated from the unit hydrograph time to peak, T_p, and standard annual average rainfall, SAAR

D = T_p (1+SAAR/1000)

= 8.5 (1+ 635/1000) *(Equation 3.10)*

= 13.9 hours

ΔT = 1.0hr therefore the duration is rounded down to the nearest odd integer D = 13hrs

Calculation of design storm depth, P

3.4 The design storm depth, P, is the T-year D-hour rainfall for the catchment and is determined from the rainfall depth-duration-frequency relationships.

3.5 Determination of the appropriate rainfall return period depends on the urban extent of the catchment and the required return period of the flood. If URBEXT < 0.125 i.e. rural or moderately urbanised catchments, T_R is determined from the design flood return period, T_F. Table 2 refers:

Therefore, the rainfall return period, T_R = 140 years

3.6 The point MT-Dh rainfall was then abstracted from the rainfall depth-duration-frequency data presented on the FEH CD-ROM.

MT-Dh (point) = M140-14h (point) = 87.4mm

3.7 The design storm depth P is the T-year D-hour catchment rainfall, calculated by scaling the MT-Dh (point) by an areal reduction factor ARF. The ARF appropriate to the catchment area and storm duration was also extracted from the FEH CD-ROM.

$ARF_{13.0}$ = 0.963

P = MT-Dh (catchment) = ARF_DMT-Dh (point) = 0.963 x 87.4 = 84.1mm

(Equation 3.2)

Derivation of design storm profile and antecedent catchment wetness index

3.8 URBEXT < 0.125, therefore the appropriate storm profile is the 75% winter profile. The design storm depth, P, was distributed within the design storm profile.

3.9 D = 13.0 hours and ΔT = 1 hour, therefore each rainfall data block of interval 1-hour shall have a duration equivalent to 1/13 or 7.7% of D. The storm is centred on the 1-hour period occurring between 8 and 9 hours after the commencement of the storm. This peak period represents 1/14 or 7.7% of D and the winter profile specifies that this contains 20% of the design storm depth P. The central 3 hours of the storm represents 3/13 or 23.1% of the storm duration. This contains 49.5% of P. Of this 20.0% occurs within the central 1 hour period, therefore the remaining 29.5% of the depth is divided between the two outer 1-hour periods with 14.75% of P in each. The rest of the profile was constructed in similar fashion and the results are represented in Table3.

3.10 The state of the catchment prior to the storm is referred to as the antecedent catchment wetness and is indexed by the catchment wetness index CWI.

SAAR = 635mm

CWI = 90.07mm

(Figure 2 refers)

Calculation of percentage runoff

3.11 The standard percentage run-off SPR was derived from catchment descriptors.

SPR = SPRHOST = 25.9%

(Equation 2.17)

3.12 The percentage runoff PR appropriate for the design event was calculated using the following equations:

$DPR_{CWI} = 0.25\ (CWI-125)$

$DPR_{CWI} = 0.25\ (93.53-125) = -8.7\%$

(Equation 2.12)

$DPR_{RAIN} = 0.45\ (P - 40)^{0.7}$

$DPR_{RAIN} = 0.45\ (84.1 - 40)^{0.7} = 6.4\%$

(Equation 2.13)

$PR_{RURAL} = SPR + DPR_{CWI} + DPR_{RAIN}$

$PR_{RURAL} = 25.9 + (-7.9) + 6.4 = 23.6\%$

(Equation 2.14)

$PR = PR_{RURAL}\ (1.0 - 0.615\ URBEXT) + 70\ (0.615\ URBEXT)$

$PR = 24.4\ (1.0 - 0.615(0.01)) + 70\ (0.615\ (0.01)) = 23.9\%$

(Equation 2.15)

Derivation of net event hyetograph

3.13 The net rainfall hyetograph was derived by applying the percentage runoff, PR, to each block of the design storm hyetograph.

Calculation of baseflow, BF

3.14 $BF = \{33(CWI - 125) + 3.0\ SAAR + 5.5\} \times 10^{-5}\ AREA$

(Equation 2.15)

$= \{33(93.53 - 125) + (3.0 \times 635) + 5.5\} \times 10^{-5} \times 14.23$

$= 0.12 m^3/s$

Derivation of total runoff hydrograph

3.15 The total runoff hydrograph was derived by adding the baseflow (BF) to each ordinate of the rapid response runoff hydrograph.

(Figures 3and 4 refer)

3.16 The 100-year flood for the River Hart at Fleet is estimated as **Q100=7.13m^3/s**.

4.0 Culvert hydraulic capacity check

Introduction

4.1 A simple one dimensional hydraulic model of the Charbridge Lane Brook was created using the HEC-RAS River Analysis Software to establish the backwater effects of the existing culvert under Skimmingdish Lane.

Physical geometry and cross-sectional data

4.2 The model was created with four cross-sections using limited level and offset information recorded during a site visit. The locations of these cross-sections relative to the culvert are listed below, *Figure 1 refers*:

- XS-4 10m upstream of the culvert
- XS-3 Immediately (0.5m) upstream of the culvert
- XS-2 Immediately (0.5m) downstream of the culvert
- XS-1 10m downstream of the culvert

4.3 The bed level and top of bank levels for cross-sections 2 and 3 were recorded during the site visit together with the dimensions of the culvert. Due to the minimal amount of information, an identical trapezoidal channel was created for all four sections with a bed width equal to that of the culvert and 1 in 2 side slopes.

4.4 With the lack of more detailed information, the longitudinal fall of the Charbridge Brook was assumed to be constant and at the same grade as the culvert.

4.5 Manning's roughness coefficients (n) for the watercourse, its banks and the culvert were estimated following a visual inspection of the watercourse and by referring to Chow, 1959.

4.6 The main channel of the watercourse can be described as a clean straight minor stream with some stones and weeds. Therefore, the channel roughness coefficient (n_{ch}) was estimated to be in the region of $0.030 \geq nch \geq 0.040$, *Photo 1 refers.* An n_{ch} value of 0.035 was used.

4.7 The culvert can be described as concrete, straight and free from debris. Therefore, the culvert roughness coefficient (n_{cu}) was estimated to be in the region of $0.010 \geq nch \geq 0.013$, *Photo 2 refers.* An n_{cu} value of 0.011 was used. As the bed of the culvert is silted with some vegetation and stones, an n_{cu} value of 0.02 was used for the bed of the culvert.

Flow and boundary conditions

4.8 Flow data for the 1 in 100 year return period flood was estimated using the Revitalised FSR/FEH Rainfall Runoff Method.

$Q100 = 4.50m^3/s$

4.9 Boundary conditions are necessary to establish the starting water surface at upstream and downstream stations of the watercourse. For mixed flow regime (sub-critical and super-critical) calculations, boundary conditions must be applied at both ends of the modelled reach.

4.10 Critical depth was used as boundary conditions for the upstream and downstream extents of the model.

4.11 A Steady Flow analysis using a mixed flow regime was implemented as this allows both sub-critical and supercritical flow calculations at each cross-section.

4.12 The model was then re-analysed with different depth of blockages within the culvert to assess the impact on water level upstream of the culvert.

Results

4.13 The results from the HEC-RAS calculations are reproduced at Tables.

CEH SPREADSHEET

FIGURES

TABLES

Δt hrs	1	2	3	4	5	6	7	8	9	10	11
u_t m^3/s	0.44	0.87	1.31	1.74	2.18	2.61	3.05	3.48	3.56	3.27	2.98
Δt hrs	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
u_t m^3/s	2.70	2.41	2.12	1.84	1.55	1.27	0.98	0.69	0.41	0.12	0.00

Table 1: Unit hydrograph ordinates

Flood peak return period (yrs)	2.33	10	30	50	100	1000
Rainfall return period (yrs)	2	17	50	81	140	1000

Table 2: Recommended storm return period to yield flood peak of return period by design event method

%D	7.7	23.1	38.5	53.9	69.2	84.6	100.0
%P	20.0	49.5	69.0	82.0	90.5	96.2	100.0
Δ%	20.0	29.5	19.5	13.0	8.5	5.7	3.8
Δ (mm)	16.8	24.8	16.4	10.9	7.1	4.8	3.2

Table 3: Distribution of design storm depth P for the 75% winter profile

Time Interval (hrs)	1	2	3	4	5	6	7	8	9	10	11	12	13
Total Rain (mm)	1.60	2.40	3.55	5.45	8.20	12.4	16.8	12.4	8.20	5.45	3.55	2.40	1.60
Net Rain (mm)	0.4	0.6	0.9	1.4	2.0	3.1	4.2	3.1	2.0	1.4	0.9	0.6	0.4

Table 4: Net rain per unit interval

Plan ID	Description	Flow (m3/s)	Water Level (m)
2281-01	CULVERT CAPACITY CHECK	4.50	68.62
2281-02	25% BLOCKED CULVERT	4.50	68.97
2281-03	50% BLOCKED CULVERT	4.50	69.29
2281-04	75% BLOCKED CULVERT	4.50	69.85

Table 5: HEC-RAS results

PHOTOGRAPHS

Photo 1 – Skimmingdish Lane Brook – Typical cross-section

Photo 1 – Skimmingdish Lane Brook – Culvert

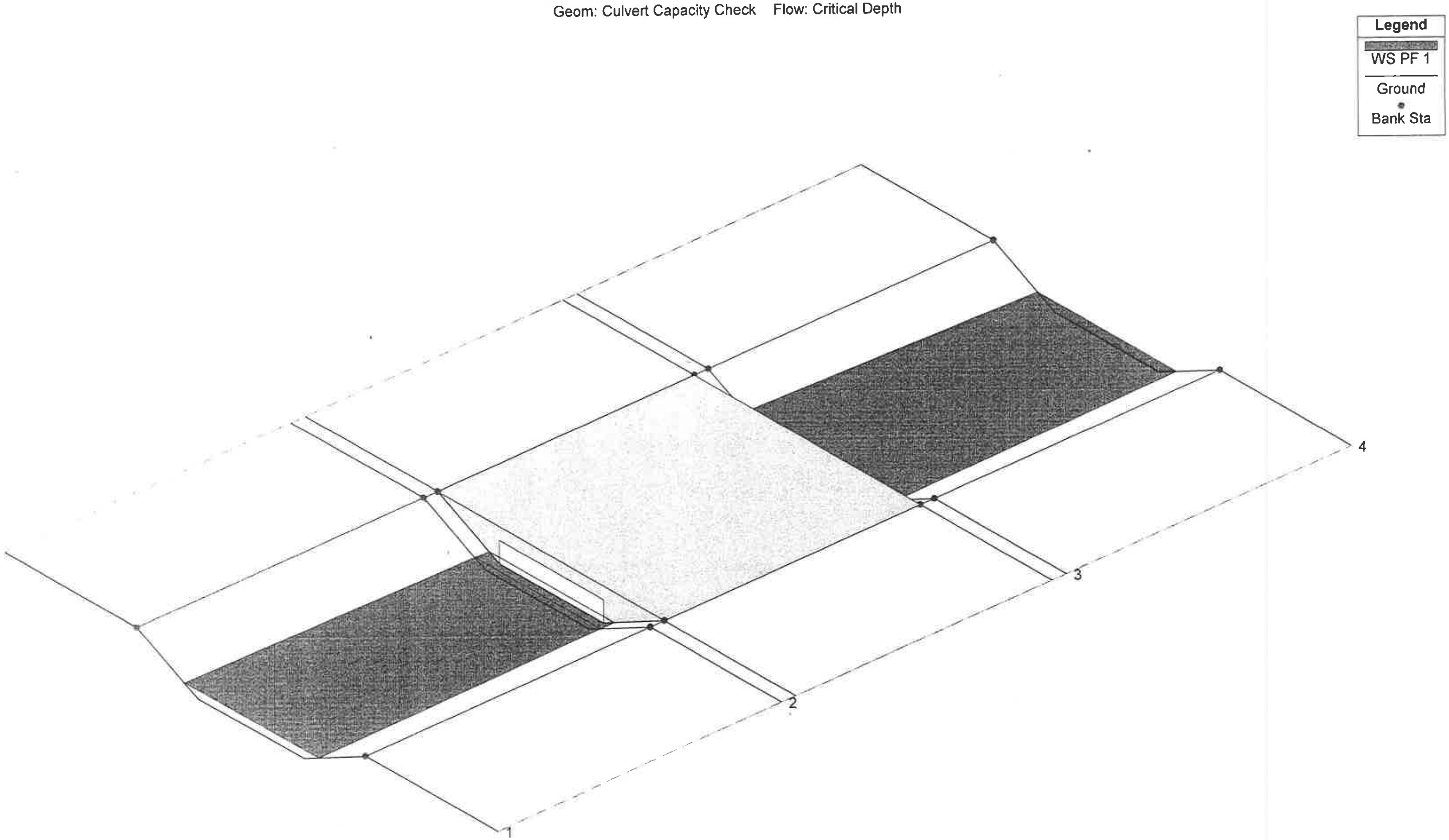
HEC-RAS OUTPUT

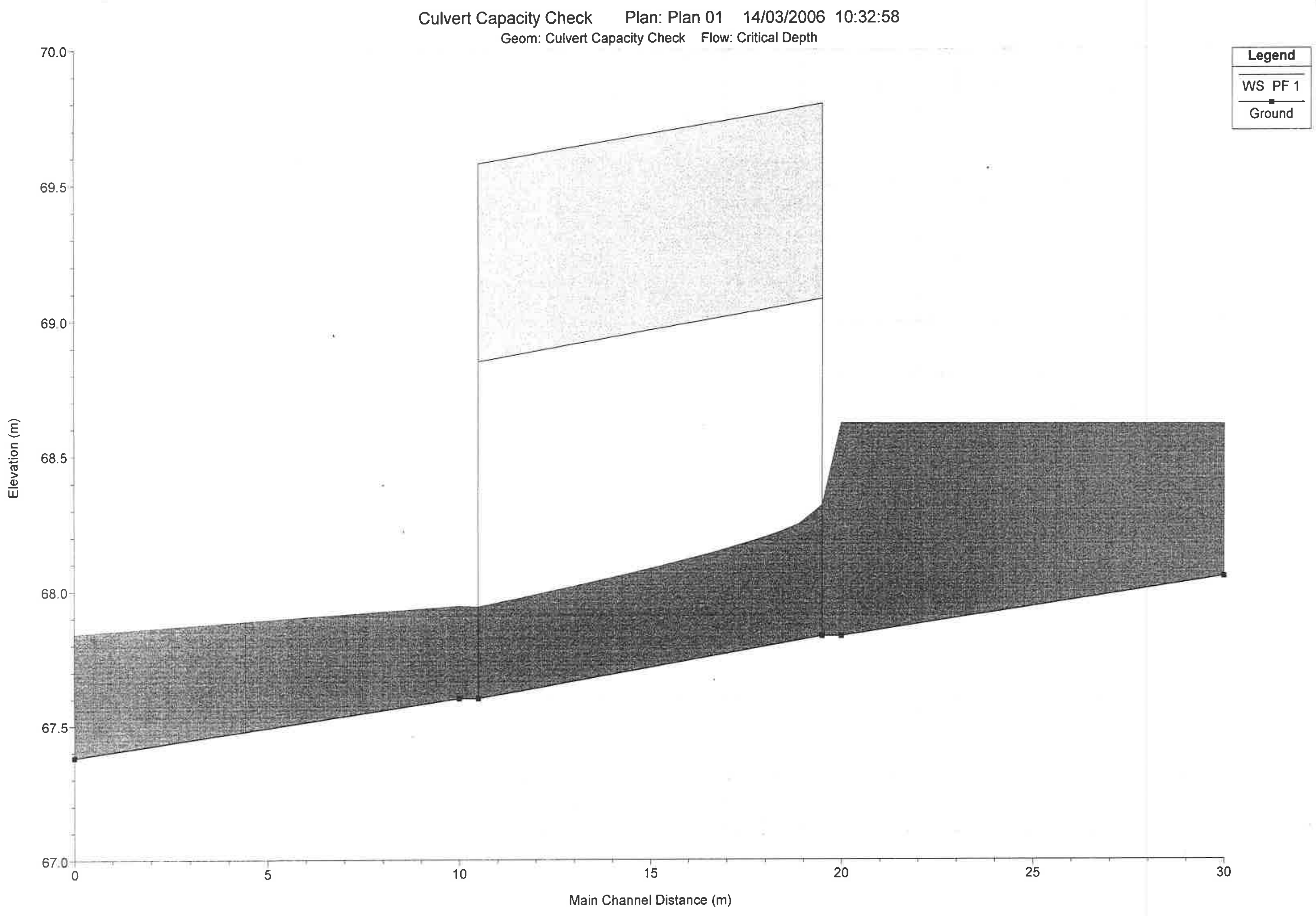
HEC-RAS Plan: 2281-01 River: Skimmingdish Bro Reach: Culvert Profile: PF 1

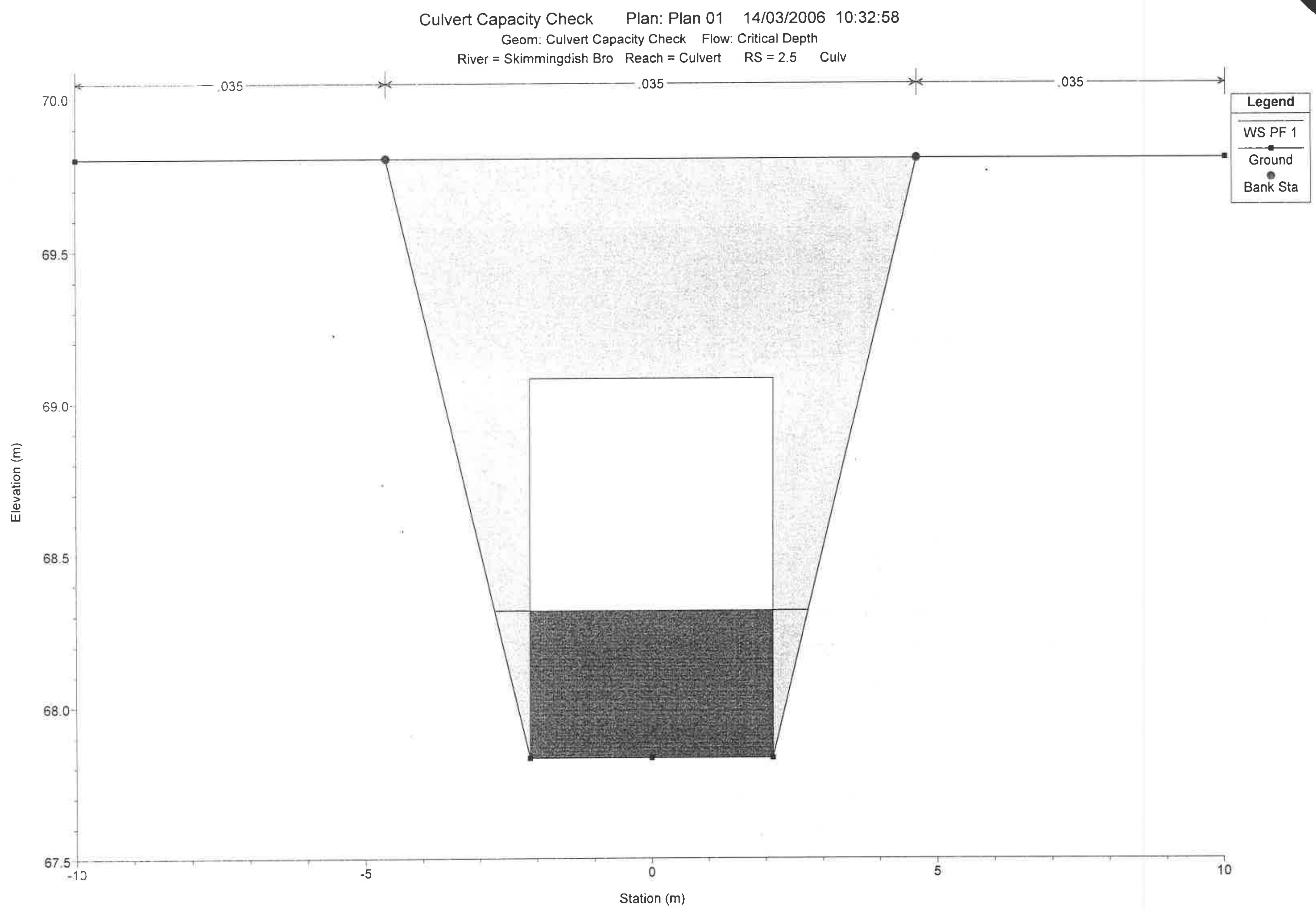
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Culvert	4	PF 1	4.50	68.05	68.62	68.51	68.75	0.008821	1.60	2.81	5.68	0.73
Culvert	3	PF 1	4.50	67.83	68.62	68.29	68.68	0.002798	1.09	4.14	6.25	0.43
Culvert	2.5		Culvert									
Culvert	2	PF 1	4.50	67.60	67.94	68.06	68.34	0.047428	2.79	1.61	5.12	0.43
Culvert	1	PF 1	4.50	67.38	67.84	67.84	68.05	0.017639	2.02	2.23	5.42	1.01

Culvert Capacity Check   Plan: Plan 01   14/03/2006 10:32:58

Geom: Culvert Capacity Check   Flow: Critical Depth



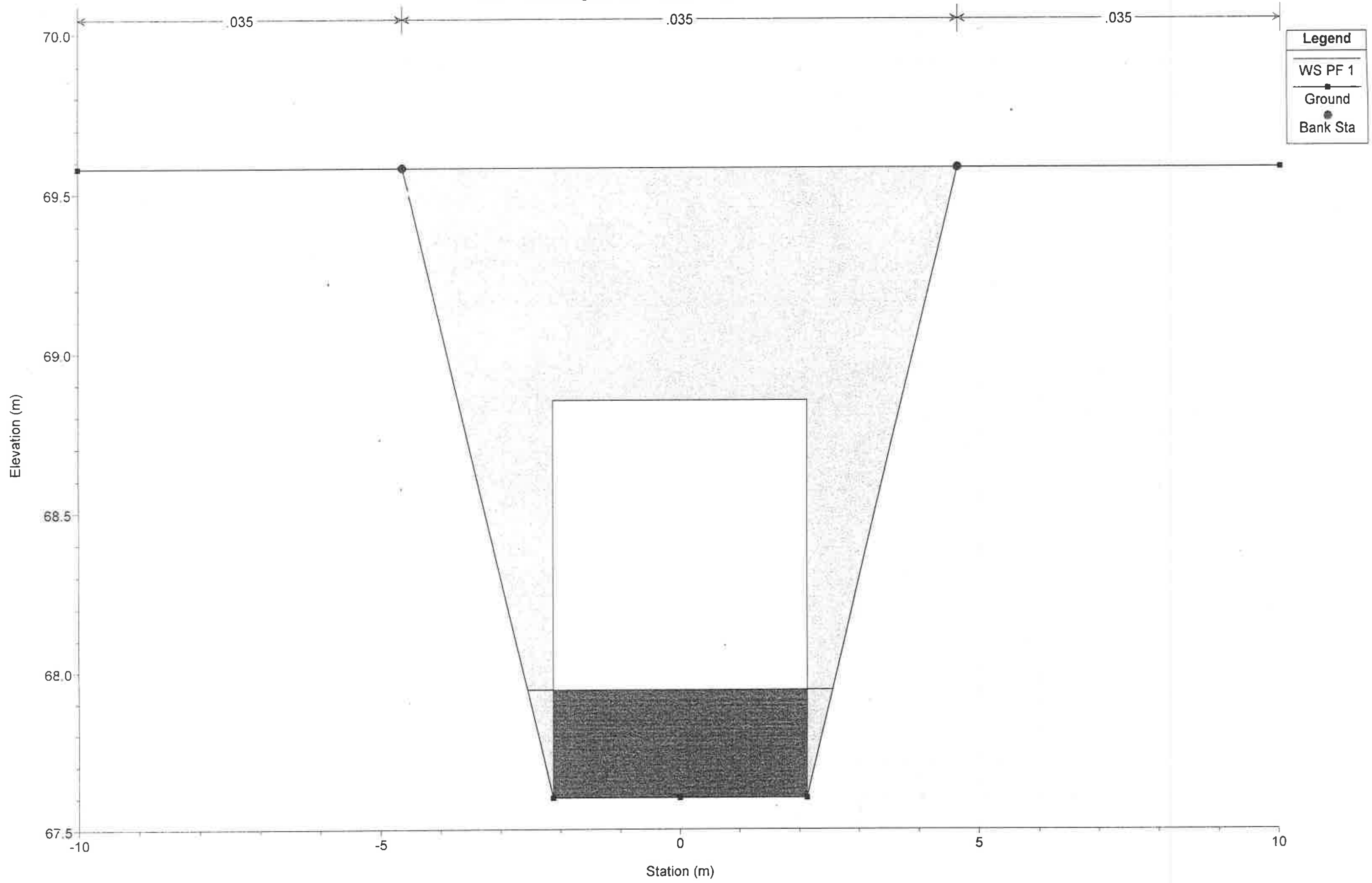

Culvert Capacity Check Plan: Plan 01 14/03/2006 10:32:58
Geom: Culvert Capacity Check Flow: Critical Depth
Legend
WS PF 1
Ground
Elevation (m)
70.0
69.5
69.0
68.5
68.0
67.5
67.0
0
5
10
15
20
25
30
Main Channel Distance (m)


Culvert Capacity Check   Plan: Plan 01   14/03/2006 10:32:58
Geom: Culvert Capacity Check   Flow: Critical Depth
River = Skimmingdish Bro   Reach = Culvert   RS = 2.5   Culv
.035
.035
.035
Legend
WS PF 1
Ground
Bank Sta
70.0
69.5
69.0
68.5
68.0
67.5
Elevation (m)
-10
-5
0
5
10
Station (m)

Culvert Capacity Check Plan: Plan 01 14/03/2006 10:32:58

Geom: Culvert Capacity Check Flow: Critical Depth

River = Skimmingdish Bro Reach = Culvert RS = 2.5 Culv



Appendix D

Additional Surface Water Drainage Information

Summary of Results for 100 year Return Period

Half Drain Time : 327 minutes

Storm Duration (mins)	Maximum Control (l/s)	Maximum Filtration (l/s)	Maximum Outflow (l/s)	Maximum Water Level (m OD)	Maximum Depth (m)	Maximum Volume (m³)	Status
15 Summer	9.7	0.0	9.7	10.1998	0.1997	149.4	O K
30 Summer	9.7	0.0	9.7	10.2328	0.2327	203.1	O K
60 Summer	9.7	0.0	9.7	10.2598	0.2597	253.5	O K
120 Summer	9.7	0.0	9.7	10.2803	0.2802	294.1	O K
180 Summer	9.7	0.0	9.7	10.2868	0.2867	308.2	O K
240 Summer	9.7	0.0	9.7	10.2883	0.2882	311.1	O K
360 Summer	9.7	0.0	9.7	10.2868	0.2867	308.0	O K
480 Summer	9.7	0.0	9.7	10.2843	0.2842	303.2	O K
600 Summer	9.7	0.0	9.7	10.2813	0.2812	296.6	O K
720 Summer	9.7	0.0	9.7	10.2773	0.2772	288.7	O K
960 Summer	9.7	0.0	9.7	10.2688	0.2687	271.0	O K
1440 Summer	9.7	0.0	9.7	10.2498	0.2497	234.3	O K
2160 Summer	9.7	0.0	9.7	10.2223	0.2222	185.4	O K
2880 Summer	9.7	0.0	9.7	10.1998	0.1997	149.5	O K
4320 Summer	8.2	0.0	8.2	10.1698	0.1698	108.3	O K
5760 Summer	7.1	0.0	7.1	10.1478	0.1478	81.8	O K
7200 Summer	6.3	0.0	6.3	10.1308	0.1308	64.0	O K
8640 Summer	5.6	0.0	5.6	10.1173	0.1173	51.4	O K
10080 Summer	5.0	0.0	5.0	10.1058	0.1058	42.0	O K
15 Winter	9.7	0.0	9.7	10.2143	0.2142	172.2	O K
30 Winter	9.7	0.0	9.7	10.2493	0.2492	232.9	O K
60 Winter	9.7	0.0	9.7	10.2783	0.2782	290.4	O K
120 Winter	9.7	0.0	9.7	10.3003	0.3002	338.0	O K

Storm Duration (mins)	Rain (mm/hr)	Time-Peak (mins)
15 Summer	99.54	18
30 Summer	65.08	33
60 Summer	40.51	62
120 Summer	24.36	122
180 Summer	17.85	180
240 Summer	14.24	240
360 Summer	10.32	296
480 Summer	8.21	356
600 Summer	6.87	422
720 Summer	5.94	490
960 Summer	4.71	626
1440 Summer	3.40	892
2160 Summer	2.45	1260
2880 Summer	1.94	1616
4320 Summer	1.39	2336
5760 Summer	1.10	3056
7200 Summer	0.91	3752
8640 Summer	0.79	4496
10080 Summer	0.69	5240
15 Winter	99.54	18
30 Winter	65.08	33
60 Winter	40.51	62
120 Winter	24.36	120

Summary of Results for 100 year Return Period

Storm Duration (mins)	Maximum Control (l/s)	Maximum Filtration (l/s)	Maximum Outflow (l/s)	Maximum Water Level (m OD)	Maximum Depth (m)	Maximum Volume (m³)	Status
180 Winter	9.7	0.0	9.7	10.3083	0.3082	356.1	O K
240 Winter	9.7	0.0	9.7	10.3108	0.3107	361.7	O K
360 Winter	9.7	0.0	9.7	10.3093	0.3092	357.9	O K
480 Winter	9.7	0.0	9.7	10.3053	0.3052	348.8	O K
600 Winter	9.7	0.0	9.7	10.3008	0.3007	339.1	O K
720 Winter	9.7	0.0	9.7	10.2953	0.2952	327.3	O K
960 Winter	9.7	0.0	9.7	10.2833	0.2832	300.6	O K
1440 Winter	9.7	0.0	9.7	10.2558	0.2557	245.1	O K
2160 Winter	9.7	0.0	9.7	10.2153	0.2152	174.0	O K
2880 Winter	9.1	0.0	9.1	10.1873	0.1873	131.7	O K
4320 Winter	7.2	0.0	7.2	10.1503	0.1503	84.6	O K
5760 Winter	6.0	0.0	6.0	10.1248	0.1248	58.3	O K
7200 Winter	5.1	0.0	5.1	10.1063	0.1063	42.4	O K
8640 Winter	4.4	0.0	4.4	10.0923	0.0923	32.0	O K
10080 Winter	3.9	0.0	3.9	10.0818	0.0818	24.9	O K

Storm Duration (mins)	Rain (mm/hr)	Time-Peak (mins)
180 Winter	17.85	176
240 Winter	14.24	232
360 Winter	10.32	338
480 Winter	8.21	382
600 Winter	6.87	458
720 Winter	5.94	534
960 Winter	4.71	682
1440 Winter	3.40	964
2160 Winter	2.45	1324
2880 Winter	1.94	1672
4320 Winter	1.39	2380
5760 Winter	1.10	3112
7200 Winter	0.91	3816
8640 Winter	0.79	4504
10080 Winter	0.69	5240

Rainfall Details

Region	ENG+WAL	Shortest Storm (mins)	15
Return Period (years)	100	Longest Storm (mins)	10080
M5-60 (mm)	20.000	Summer Storms	Yes
Ratio-R	0.410	Winter Storms	Yes
Cv (Summer)	0.750	Climate Change %	+0
Cv (Winter)	0.840		

Time / Area Diagram

Total Area (ha) = 1.030

Time from:	(mins) to:	Area (ha)
0	4	1.030

Porous Car Park Details

Infil Coef - Base (m/hr)	0.000000	Invert Level (m)	10.000
Membrane Percolation (mm/hr)	1000	Cover Level (m)	11.000
Safety Factor	2.0	Slope (1:x)	250.0
Porosity	0.30	Max Percolation (l/s)	2055.6
Length (m)	74.0	Depression Storage (mm)	5
Width (m)	100.0	Evaporation (mm/day)	3

Hydro-Brake Outflow Control

Design Head (m)	0.500	Hydro-Brake Type	MD2	Invert Level (m)	10.000
Design Flow (l/s)	11.0	Diameter (mm)	123		

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.10	4.8	0.80	14.0	2.00	22.2	4.00	31.4	7.00	41.5
0.20	9.7	1.00	15.7	2.20	23.3	4.50	33.3	7.50	43.0
0.30	9.4	1.20	17.2	2.40	24.3	5.00	35.1	8.00	44.4
0.40	10.0	1.40	18.6	2.60	25.3	5.50	36.8	8.50	45.7
0.50	11.1	1.60	19.8	3.00	27.2	6.00	38.4	9.00	47.1
0.60	12.1	1.80	21.0	3.50	29.3	6.50	40.0	9.50	48.3