Appendix E Greenfield Runoff Estimation

ICP SUDS Mean Annual Flood

```
                              Input

Return Period (years)      2  SAAR (mm)   850         Urban    0.000
           Area (ha)  1.000       Soil  0.450  Region Number  Region 7

                         Results    l/s

                        QBAR Rural   5.5
                        QBAR Urban   5.5

                          Q2 years   4.9

                           Q1 year   4.7
                         Q30 years  12.5
                        Q100 years  17.6
```

AWP		Page 1
Kensington Court Woodwater Park Pynes Hill Exeter EX2 5TY	0456 Land south of Gillingham Greenfield Runoff Estimation ICP SuDS Method	
Date 03/11/2017 15:04 File	Designed by chris.yalden Checked by	
XP Solutions	Source Control 2017.1	

ICP SUDS Mean Annual Flood

```
                                   Input

Return Period (years)       2 SAAR (mm)   850         Urban    0.000
            Area (ha) 36.600      Soil 0.450 Region Number Region 7

                              Results    l/s

                              QBAR Rural 201.8
                              QBAR Urban 201.8

                                Q2 years 177.8

                                 Q1 year 171.5
                               Q30 years 457.3
                              Q100 years 643.7
```

Appendix F Long Term Storage Calculation

Long Term Storage (LTS) Volume Calculation

Project No.	*0456*
Project Title	*Ham Farm, Gillingham*
Client	*Welbeck Strategic Land*
Sheet Ref	*Total Application*

Calcs by	*CPY*
Reviewed by	*RPW*
Date	*18.12.2018*
Revision	*B*

LTS calculation method based on equation 24.10 from CIRIA C753 - The SuDS Manual (2015);

$$Vol_{xs} = RD \text{ x } A \text{ x } 10 \, [PIMP/100 \text{ x } (\alpha \text{ x } Cv) + (1-PIMP/100) \text{ x } (\beta \text{ x } SPR) - SPR]$$

Where;	Vol_{xs}	Extra runoff volume from a dev. site compared to the greenfield equivalent during the 100 yr 6 hr storm			
	RD	Rainfall Depth	63	mm	*(for 100 year 6 hour storm)*
	A	*Site Area*	1.67	ha	*(Exc. large undeveloped areas)*
		Impermeable Catchment	1	ha	
	PIMP	*Percentage Impermeable*	60.0	%	
	α	Proportion Impermeable to Network	1.0		
	Cv	Impermeable Runoff Coefficient	0.84		*(0.84 Modified Rational Method)*
		Permeable Catchment	0.67	ha	
		Permeable Catchment to Network	0.00	ha	
	β	Proportion Perm. to Network	0.00		
	SPR	Soil Proportion Runoff	0.45		*(Ref. to WRAP map)*

	RD		*A*		*PIMP*			**α**		*Cv*		*PIMP*		**β**		*SPR*		*SPR*	
Vol_{xs} =	63	x	1.7	x 10 x ((	60	/ 100)	x (	1.00	x	0.84	) + (1 -	60	/ 100) x (	0.00	x	0.45	) -	0.45	)

$Volume_{xs}$ <u>57</u> *m3 per hectare of impermeable catchment*

LTS Discharge Rate <u>3.33</u> *(2 l/s/ha)*

As above, assuming all permeable surfaces <u>do not</u> enter the drainage system

Vol_{xs} = <u>*56.70*</u>

As above, assuming all permeable surfaces enter the drainage system

Vol_{xs} = <u>*245.70*</u>

Appendix G MicroDrainage Modelling Outputs

Summary of Results for 2 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Overflow (l/s)	Σ Max Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	98.613	0.113	2.0	0.0	2.0	90.6	O K
30 min Summer	98.649	0.149	2.2	0.0	2.2	119.1	O K
60 min Summer	98.687	0.187	2.4	0.0	2.4	149.8	O K
120 min Summer	98.728	0.228	2.6	0.0	2.6	182.7	O K
180 min Summer	98.753	0.253	2.7	0.0	2.7	202.3	O K
240 min Summer	98.770	0.270	2.8	0.0	2.8	215.8	O K
360 min Summer	98.791	0.291	2.9	0.0	2.9	232.9	O K
480 min Summer	98.803	0.303	3.0	0.0	3.0	242.2	O K
600 min Summer	98.809	0.309	3.0	0.0	3.0	247.6	O K
720 min Summer	98.814	0.314	3.0	0.0	3.0	251.5	O K
960 min Summer	98.821	0.321	3.1	0.0	3.1	257.0	O K
1440 min Summer	98.827	0.327	3.1	0.0	3.1	261.9	O K
2160 min Summer	98.826	0.326	3.1	0.0	3.1	260.7	O K
2880 min Summer	98.818	0.318	3.1	0.0	3.1	254.7	O K
4320 min Summer	98.798	0.298	3.0	0.0	3.0	238.2	O K
5760 min Summer	98.777	0.277	2.9	0.0	2.9	221.2	O K
7200 min Summer	98.756	0.256	2.8	0.0	2.8	205.0	O K
8640 min Summer	98.738	0.238	2.7	0.0	2.7	190.1	O K
10080 min Summer	98.720	0.220	2.6	0.0	2.6	176.3	O K
15 min Winter	98.627	0.127	2.0	0.0	2.0	101.6	O K
30 min Winter	98.667	0.167	2.3	0.0	2.3	133.5	O K
60 min Winter	98.710	0.210	2.5	0.0	2.5	168.1	O K
120 min Winter	98.757	0.257	2.8	0.0	2.8	205.3	O K
180 min Winter	98.785	0.285	2.9	0.0	2.9	227.6	O K
240 min Winter	98.804	0.304	3.0	0.0	3.0	243.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
15 min Summer	48.984	0.0	76.1	0.0	19
30 min Summer	32.391	0.0	102.4	0.0	34
60 min Summer	20.665	0.0	145.6	0.0	64
120 min Summer	12.926	0.0	183.2	0.0	122
180 min Summer	9.776	0.0	208.2	0.0	182
240 min Summer	8.008	0.0	227.5	0.0	242
360 min Summer	6.032	0.0	256.9	0.0	362
480 min Summer	4.921	0.0	278.9	0.0	480
600 min Summer	4.201	0.0	296.8	0.0	572
720 min Summer	3.692	0.0	311.7	0.0	622
960 min Summer	3.011	0.0	335.4	0.0	748
1440 min Summer	2.259	0.0	362.4	0.0	1010
2160 min Summer	1.694	0.0	449.6	0.0	1428
2880 min Summer	1.381	0.0	487.5	0.0	1844
4320 min Summer	1.036	0.0	542.7	0.0	2640
5760 min Summer	0.845	0.0	604.7	0.0	3456
7200 min Summer	0.721	0.0	644.9	0.0	4248
8640 min Summer	0.634	0.0	679.0	0.0	5008
10080 min Summer	0.568	0.0	706.1	0.0	5752
15 min Winter	48.984	0.0	86.0	0.0	19
30 min Winter	32.391	0.0	115.1	0.0	33
60 min Winter	20.665	0.0	163.6	0.0	62
120 min Winter	12.926	0.0	205.7	0.0	120
180 min Winter	9.776	0.0	233.6	0.0	180
240 min Winter	8.008	0.0	255.1	0.0	238

Summary of Results for 2 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
360 min Winter	98.829	0.329	3.1	0.0	3.1	263.5	O K
480 min Winter	98.844	0.344	3.2	0.0	3.2	275.2	O K
600 min Winter	98.853	0.353	3.2	0.0	3.2	282.5	O K
720 min Winter	98.859	0.359	3.2	0.0	3.2	286.8	O K
960 min Winter	98.864	0.364	3.3	0.0	3.3	290.9	O K
1440 min Winter	98.868	0.368	3.3	0.0	3.3	294.6	O K
2160 min Winter	98.861	0.361	3.2	0.0	3.2	289.0	O K
2880 min Winter	98.847	0.347	3.2	0.0	3.2	277.4	O K
4320 min Winter	98.813	0.313	3.0	0.0	3.0	250.1	O K
5760 min Winter	98.779	0.279	2.9	0.0	2.9	223.4	O K
7200 min Winter	98.749	0.249	2.7	0.0	2.7	199.5	O K
8640 min Winter	98.723	0.223	2.6	0.0	2.6	178.3	O K
10080 min Winter	98.699	0.199	2.5	0.0	2.5	159.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
360 min Winter	6.032	0.0	287.7	0.0	352
480 min Winter	4.921	0.0	312.0	0.0	464
600 min Winter	4.201	0.0	331.6	0.0	574
720 min Winter	3.692	0.0	347.7	0.0	680
960 min Winter	3.011	0.0	372.4	0.0	780
1440 min Winter	2.259	0.0	394.2	0.0	1082
2160 min Winter	1.694	0.0	503.8	0.0	1540
2880 min Winter	1.381	0.0	546.2	0.0	1988
4320 min Winter	1.036	0.0	606.9	0.0	2852
5760 min Winter	0.845	0.0	677.6	0.0	3688
7200 min Winter	0.721	0.0	722.7	0.0	4472
8640 min Winter	0.634	0.0	761.2	0.0	5272
10080 min Winter	0.568	0.0	792.3	0.0	6048

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	2	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	18.300	Shortest Storm (mins)	15
Ratio R	0.350	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.000

Time From:	**(mins) To:**	**Area (ha)**
0	4	1.000

Model Details

Storage is Online Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.500

Depth (m)	Area (m^2)	Depth (m)	Area (m^2)
0.000	800.0	1.500	800.0

Complex Outflow Control

Hydro-Brake® Optimum

Unit Reference	MD-SHE-0079-2000-0125-2000
Design Head (m)	0.125
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	79
Invert Level (m)	98.500
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.125	2.0	Kick-Flo®	0.121	2.0
Flush-Flo™	0.101	2.0	Mean Flow over Head Range	-	1.2

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

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)
0.100	2.0	0.800	4.7	2.000	7.2	4.000	10.3	7.000	13.6
0.200	2.5	1.000	5.2	2.200	7.6	4.500	10.9	7.500	14.1
0.300	3.0	1.200	5.7	2.400	7.9	5.000	11.5	8.000	14.6
0.400	3.4	1.400	6.0	2.600	8.3	5.500	12.1	8.500	15.0
0.500	3.8	1.600	6.5	3.000	8.9	6.000	12.6	9.000	15.5
0.600	4.1	1.800	6.9	3.500	9.6	6.500	13.1	9.500	15.9

Pipe

Diameter (m)	0.100	Roughness k (mm)	0.600	Upstream Invert Level (m)	98.860
Slope (1:X)	100.0	Entry Loss Coefficient	0.500		
Length (m)	10.000	Coefficient of Contraction	0.600		

Pipe Overflow Control

Diameter (m)	0.100	Roughness k (mm)	0.600	Upstream Invert Level (m)	99.160
Slope (1:X)	100.0	Entry Loss Coefficient	0.500		
Length (m)	10.000	Coefficient of Contraction	0.600		

Summary of Results for 30 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	98.715	0.215	2.6	0.0	2.6	172.0	O K
30 min Summer	98.785	0.285	2.9	0.0	2.9	227.9	O K
60 min Summer	98.860	0.360	3.2	0.0	3.2	288.0	O K
120 min Summer	98.934	0.434	5.8	0.0	5.8	347.2	O K
180 min Summer	98.968	0.468	7.9	0.0	7.9	374.1	O K
240 min Summer	98.983	0.483	8.9	0.0	8.9	386.5	O K
360 min Summer	98.995	0.495	9.6	0.0	9.6	396.2	O K
480 min Summer	99.005	0.505	9.8	0.0	9.8	404.3	O K
600 min Summer	99.013	0.513	10.0	0.0	10.0	410.1	O K
720 min Summer	99.017	0.517	10.1	0.0	10.1	413.8	O K
960 min Summer	99.021	0.521	10.2	0.0	10.2	416.8	O K
1440 min Summer	99.017	0.517	10.1	0.0	10.1	413.6	O K
2160 min Summer	99.001	0.501	9.7	0.0	9.7	401.1	O K
2880 min Summer	98.986	0.486	9.1	0.0	9.1	388.8	O K
4320 min Summer	98.962	0.462	7.5	0.0	7.5	369.2	O K
5760 min Summer	98.942	0.442	6.3	0.0	6.3	353.7	O K
7200 min Summer	98.925	0.425	5.4	0.0	5.4	339.8	O K
8640 min Summer	98.911	0.411	4.7	0.0	4.7	328.9	O K
10080 min Summer	98.898	0.398	4.1	0.0	4.1	318.0	O K
15 min Winter	98.741	0.241	2.7	0.0	2.7	192.7	O K
30 min Winter	98.819	0.319	3.1	0.0	3.1	255.4	O K
60 min Winter	98.903	0.403	4.3	0.0	4.3	322.5	O K
120 min Winter	98.981	0.481	8.7	0.0	8.7	385.1	O K
180 min Winter	99.017	0.517	10.1	0.0	10.1	413.7	O K
240 min Winter	99.037	0.537	10.5	0.0	10.5	429.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
15 min Summer	92.611	0.0	146.7	0.0	19
30 min Summer	61.675	0.0	188.0	0.0	34
60 min Summer	39.381	0.0	280.8	0.0	64
120 min Summer	24.438	0.0	348.5	0.0	122
180 min Summer	18.289	0.0	391.4	0.0	182
240 min Summer	14.820	0.0	423.0	0.0	240
360 min Summer	10.965	0.0	468.8	0.0	304
480 min Summer	8.854	0.0	503.4	0.0	362
600 min Summer	7.496	0.0	530.7	0.0	426
720 min Summer	6.540	0.0	552.6	0.0	492
960 min Summer	5.270	0.0	584.0	0.0	628
1440 min Summer	3.882	0.0	608.8	0.0	896
2160 min Summer	2.856	0.0	761.4	0.0	1296
2880 min Summer	2.295	0.0	813.3	0.0	1676
4320 min Summer	1.684	0.0	878.4	0.0	2468
5760 min Summer	1.351	0.0	968.6	0.0	3280
7200 min Summer	1.138	0.0	1019.4	0.0	4104
8640 min Summer	0.989	0.0	1061.7	0.0	4920
10080 min Summer	0.879	0.0	1097.0	0.0	5744
15 min Winter	92.611	0.0	163.0	0.0	19
30 min Winter	61.675	0.0	203.3	0.0	33
60 min Winter	39.381	0.0	314.6	0.0	62
120 min Winter	24.438	0.0	391.2	0.0	120
180 min Winter	18.289	0.0	439.5	0.0	176
240 min Winter	14.820	0.0	474.9	0.0	232

Summary of Results for 30 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m^3)	Status
360 min Winter	99.054	0.554	10.9	0.0	10.9	442.9	O K
480 min Winter	99.064	0.564	11.1	0.0	11.1	451.6	O K
600 min Winter	99.071	0.571	11.3	0.0	11.3	456.5	O K
720 min Winter	99.073	0.573	11.3	0.0	11.3	458.3	O K
960 min Winter	99.070	0.570	11.3	0.0	11.3	456.4	O K
1440 min Winter	99.053	0.553	10.9	0.0	10.9	442.6	O K
2160 min Winter	99.022	0.522	10.2	0.0	10.2	417.4	O K
2880 min Winter	98.995	0.495	9.6	0.0	9.6	396.2	O K
4320 min Winter	98.964	0.464	7.6	0.0	7.6	371.4	O K
5760 min Winter	98.942	0.442	6.2	0.0	6.2	353.3	O K
7200 min Winter	98.922	0.422	5.3	0.0	5.3	337.8	O K
8640 min Winter	98.907	0.407	4.4	0.0	4.4	325.9	O K
10080 min Winter	98.890	0.390	3.8	0.0	3.8	311.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m^3)	Discharge Volume (m^3)	Overflow Volume (m^3)	Time-Peak (mins)
360 min Winter	10.965	0.0	526.1	0.0	332
480 min Winter	8.854	0.0	564.6	0.0	374
600 min Winter	7.496	0.0	594.9	0.0	450
720 min Winter	6.540	0.0	619.3	0.0	526
960 min Winter	5.270	0.0	654.7	0.0	674
1440 min Winter	3.882	0.0	687.2	0.0	956
2160 min Winter	2.856	0.0	853.7	0.0	1364
2880 min Winter	2.295	0.0	911.9	0.0	1732
4320 min Winter	1.684	0.0	981.7	0.0	2548
5760 min Winter	1.351	0.0	1085.2	0.0	3392
7200 min Winter	1.138	0.0	1142.0	0.0	4184
8640 min Winter	0.989	0.0	1189.4	0.0	5104
10080 min Winter	0.879	0.0	1229.7	0.0	6048

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	30	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	18.300	Shortest Storm (mins)	15
Ratio R	0.350	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.000

Time From:	**(mins) To:**	**Area (ha)**
0	4	1.000

Model Details

Storage is Online Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.500

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	800.0	1.500	800.0

Complex Outflow Control

Hydro-Brake® Optimum

Unit Reference	MD-SHE-0079-2000-0125-2000
Design Head (m)	0.125
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	79
Invert Level (m)	98.500
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.125	2.0	Kick-Flo®	0.121	2.0
Flush-Flo™	0.101	2.0	Mean Flow over Head Range	-	1.2

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

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)
0.100	2.0	0.800	4.7	2.000	7.2	4.000	10.3	7.000	13.6
0.200	2.5	1.000	5.2	2.200	7.6	4.500	10.9	7.500	14.1
0.300	3.0	1.200	5.7	2.400	7.9	5.000	11.5	8.000	14.6
0.400	3.4	1.400	6.0	2.600	8.3	5.500	12.1	8.500	15.0
0.500	3.8	1.600	6.5	3.000	8.9	6.000	12.6	9.000	15.5
0.600	4.1	1.800	6.9	3.500	9.6	6.500	13.1	9.500	15.9

Pipe

Diameter (m)	0.100	Roughness k (mm)	0.600	Upstream Invert Level (m)	98.860
Slope (1:X)	100.0	Entry Loss Coefficient	0.500		
Length (m)	10.000	Coefficient of Contraction	0.600		

Pipe Overflow Control

Diameter (m)	0.100	Roughness k (mm)	0.600	Upstream Invert Level (m)	99.160
Slope (1:X)	100.0	Entry Loss Coefficient	0.500		
Length (m)	10.000	Coefficient of Contraction	0.600		

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	98.778	0.278	2.9	0.0	2.9	222.4	O K
30 min Summer	98.872	0.372	3.4	0.0	3.4	297.7	O K
60 min Summer	98.969	0.469	8.0	0.0	8.0	375.2	O K
120 min Summer	99.056	0.556	11.0	0.0	11.0	444.7	O K
180 min Summer	99.096	0.596	11.8	0.0	11.8	477.1	O K
240 min Summer	99.116	0.616	12.2	0.0	12.2	492.7	O K
360 min Summer	99.132	0.632	12.5	0.0	12.5	505.6	O K
480 min Summer	99.143	0.643	12.7	0.0	12.7	514.7	O K
600 min Summer	99.150	0.650	12.8	0.0	12.8	520.3	O K
720 min Summer	99.154	0.654	12.9	0.0	12.9	523.2	O K
960 min Summer	99.154	0.654	12.9	0.0	12.9	523.6	O K
1440 min Summer	99.141	0.641	12.6	0.0	12.6	513.2	O K
2160 min Summer	99.111	0.611	12.1	0.0	12.1	488.4	O K
2880 min Summer	99.078	0.578	11.4	0.0	11.4	462.7	O K
4320 min Summer	99.026	0.526	10.3	0.0	10.3	420.5	O K
5760 min Summer	98.991	0.491	9.4	0.0	9.4	392.9	O K
7200 min Summer	98.971	0.471	8.1	0.0	8.1	376.8	O K
8640 min Summer	98.955	0.455	7.1	0.0	7.1	364.1	O K
10080 min Summer	98.942	0.442	6.2	0.0	6.2	353.3	O K
15 min Winter	98.811	0.311	3.0	0.0	3.0	249.2	O K
30 min Winter	98.917	0.417	5.0	0.0	5.0	333.2	O K
60 min Winter	99.023	0.523	10.2	0.0	10.2	418.3	O K
120 min Winter	99.124	0.624	12.3	0.0	12.3	498.8	O K
180 min Winter	99.171	0.671	13.2	0.1	13.2	537.0	O K
240 min Winter	99.194	0.694	13.6	0.6	14.1	555.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
15 min Summer	119.600	0.0	184.0	0.0	19
30 min Summer	80.444	0.0	220.8	0.0	34
60 min Summer	51.705	0.0	369.6	0.0	64
120 min Summer	32.141	0.0	460.9	0.0	122
180 min Summer	24.001	0.0	516.6	0.0	182
240 min Summer	19.381	0.0	555.9	0.0	240
360 min Summer	14.239	0.0	611.2	0.0	302
480 min Summer	11.447	0.0	652.8	0.0	362
600 min Summer	9.656	0.0	685.0	0.0	426
720 min Summer	8.398	0.0	710.5	0.0	492
960 min Summer	6.732	0.0	746.6	0.0	628
1440 min Summer	4.920	0.0	783.3	0.0	898
2160 min Summer	3.588	0.0	958.7	0.0	1296
2880 min Summer	2.864	0.0	1017.5	0.0	1676
4320 min Summer	2.081	0.0	1085.9	0.0	2420
5760 min Summer	1.657	0.0	1189.2	0.0	3168
7200 min Summer	1.388	0.0	1243.9	0.0	3896
8640 min Summer	1.200	0.0	1289.1	0.0	4672
10080 min Summer	1.062	0.0	1327.4	0.0	5448
15 min Winter	119.600	0.0	199.9	0.0	19
30 min Winter	80.444	0.0	239.2	0.0	33
60 min Winter	51.705	0.0	414.9	0.0	62
120 min Winter	32.141	0.0	517.2	0.0	120
180 min Winter	24.001	0.0	579.4	0.1	176
240 min Winter	19.381	0.0	623.4	1.5	232

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
360 min Winter	99.208	0.708	13.8	1.0	14.8	566.1	O K
480 min Winter	99.217	0.717	13.9	1.6	15.5	574.0	O K
600 min Winter	99.221	0.721	14.0	1.7	15.8	576.7	O K
720 min Winter	99.220	0.720	14.0	1.7	15.7	576.2	O K
960 min Winter	99.212	0.712	13.9	1.3	15.2	569.9	O K
1440 min Winter	99.184	0.684	13.4	0.3	13.7	547.6	O K
2160 min Winter	99.132	0.632	12.5	0.0	12.5	505.3	O K
2880 min Winter	99.084	0.584	11.5	0.0	11.5	467.2	O K
4320 min Winter	99.014	0.514	10.0	0.0	10.0	411.3	O K
5760 min Winter	98.980	0.480	8.6	0.0	8.6	383.7	O K
7200 min Winter	98.959	0.459	7.3	0.0	7.3	367.1	O K
8640 min Winter	98.943	0.443	6.3	0.0	6.3	354.0	O K
10080 min Winter	98.928	0.428	5.5	0.0	5.5	342.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
360 min Winter	14.239	0.0	685.3	6.4	304
480 min Winter	11.447	0.0	731.8	10.5	368
600 min Winter	9.656	0.0	767.6	12.1	442
720 min Winter	8.398	0.0	795.8	11.9	518
960 min Winter	6.732	0.0	835.5	9.2	666
1440 min Winter	4.920	0.0	881.4	1.8	964
2160 min Winter	3.588	0.0	1074.8	0.0	1368
2880 min Winter	2.864	0.0	1140.8	0.0	1760
4320 min Winter	2.081	0.0	1213.9	0.0	2504
5760 min Winter	1.657	0.0	1332.2	0.0	3224
7200 min Winter	1.388	0.0	1393.6	0.0	3968
8640 min Winter	1.200	0.0	1444.0	0.0	4760
10080 min Winter	1.062	0.0	1487.0	0.0	5640

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	18.300	Shortest Storm (mins)	15
Ratio R	0.350	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.000

Time From:	**(mins) To:**	**Area (ha)**
0	4	1.000

Model Details

Storage is Online Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.500

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	800.0	1.500	800.0

Complex Outflow Control

Hydro-Brake® Optimum

Unit Reference	MD-SHE-0079-2000-0125-2000
Design Head (m)	0.125
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	79
Invert Level (m)	98.500
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.125	2.0	Kick-Flo®	0.121	2.0
Flush-Flo™	0.101	2.0	Mean Flow over Head Range	-	1.2

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

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)
0.100	2.0	0.800	4.7	2.000	7.2	4.000	10.3	7.000	13.6
0.200	2.5	1.000	5.2	2.200	7.6	4.500	10.9	7.500	14.1
0.300	3.0	1.200	5.7	2.400	7.9	5.000	11.5	8.000	14.6
0.400	3.4	1.400	6.0	2.600	8.3	5.500	12.1	8.500	15.0
0.500	3.8	1.600	6.5	3.000	8.9	6.000	12.6	9.000	15.5
0.600	4.1	1.800	6.9	3.500	9.6	6.500	13.1	9.500	15.9

Pipe

Diameter (m)	0.100	Roughness k (mm)	0.600	Upstream Invert Level (m)	98.860
Slope (1:X)	100.0	Entry Loss Coefficient	0.500		
Length (m)	10.000	Coefficient of Contraction	0.600		

Pipe Overflow Control

Diameter (m)	0.100	Roughness k (mm)	0.600	Upstream Invert Level (m)	99.160
Slope (1:X)	100.0	Entry Loss Coefficient	0.500		
Length (m)	10.000	Coefficient of Contraction	0.600		

Appendix H JBA Consulting Flood Modelling

Gillingham Modelling Extension

FINAL Report

29th October 2018

www.jbaconsulting.com

Welbeck Land
13 Woodstock Street
London
W1C 2 AG

Paul Redbourne

JBA Consulting
Kings Chambers
8 High Street
Newport
South Wales
NP20 1FQ

Revision history

Revision Ref/Date	Amendments	Issued to
29/10/2018	REV A - FINAL	Welbeck Land

Contract

This report describes work commissioned by Welbeck Strategic Land LLP by a letter dated 1st May 2018. Paul Redbourne of JBA Consulting carried out this work.

Prepared by .... Paul Redbourne BSc PGCert

Analyst

Reviewed by .... Adam Sinclair MA MSc CWem CSci CEnv

Senior Flood Risk Consultant

.... George Baker BEng AIEMA CEnv IEng MCIWEM C.WEM

Technical Director

Purpose

This document has been prepared as a Final Report. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to Welbeck Strategic Land LLP.

Copyright

Contents

List of Figures

List of Tables

Abbreviations

1D	One-Dimensional (modelling)
2D	Two-Dimensional (modelling)
ABD	Area Benefitting from Defence
AEP	Annual Exceedance Probability
DSM	Digital Surface Model
DTM	Digital Terrain Model
EA	Environment Agency
FEH	Flood Estimation Handbook
FEP	Flood Estimation Point
JBA	Jeremy Benn Associates Limited
LIDAR	Light Detection and Ranging
OS	Ordnance Survey
ReFH	Revitalised Flood Hydrograph
TUFLOW	Two-dimensional Unsteady FLOW (a hydraulic model)

1 Introduction

1.1 Project introduction

JBA Consulting have been commissioned by Awcock Ward Partnership on behalf of Welbeck Strategic Land and C G Fry & Son to review and update the existing Environment Agency (EA) 1D-2D ESTRY-TUFLOW model of the watercourses that flow through and adjacent to the town of Gillingham, Dorset. The updated model will be used to assess the existing EA flood zones and update them if the new model is shown to exhibit a change in flood risk.

The existing model was developed by Capita Symonds in 2006 for the Areas Benefitting from Defences (ABD) project[1] on behalf of the EA. That project used channel survey from a 1999 survey commission. The EA revisited the model in 2011 to assess the undefended model scenario for lower order design events but no subsequent updates have been applied to the model since.

The study objectives for this commission were:

- Review the schematisation of the existing model to ensure that it uses up-to-date modelling techniques and extend the model to improve the downstream boundary application and incorporate the Fern Brook and the Meadow watercourse into the model.
- Undertake a new hydrological assessment to provide updated model inflow hydrographs for the 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1%, 0.5%, 0.2%, 0.1% AEP events and the 1% AEP plus 40% and 85% climate change uplifts.
- Produce a technical modelling report outlining the construction of the model and the decisions, assumptions, and limitations of the flood model.

The principal area of interest is the predominantly rural area to the south of Gillingham. The extent of the updated model is shown in comparison to the existing 2006 ABD model in Figure 1-1.

---

[1] Gillingham ABD Final Report, Capita Symonds, November 2006

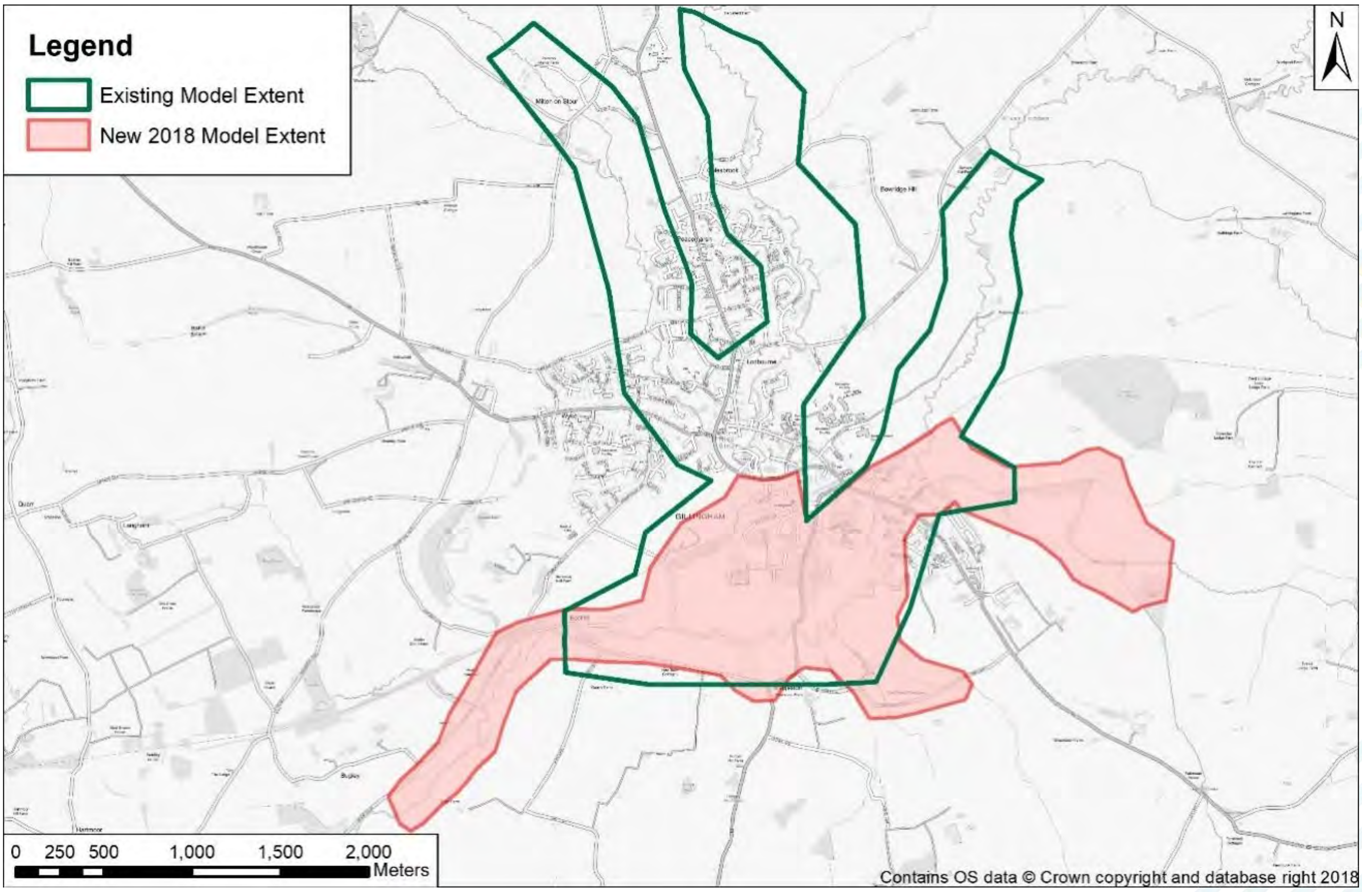


Figure 1-1: Updated modelled study area

1.2 Project reporting

This Model User Report includes detail on the configuration of the hydraulic model, model evaluation and contains details of how to rerun the model.

This document is split into nine sections:

2 Modelling Approach – Provides details of the available data and provides an overview of the modelling approach adopted.
3 Boundary Conditions – Summarises the schematisation of the model.
4 Structures – Provides details of the structures present within the study area and how they have been modelled if included within the hydraulic model.
5 Flood Defences – Provides details of any flood defences included in the defended model and the nature of the undefended model.
6 Topography Modification – Provides details of all topographic modifications that have been made to the basic LIDAR based model DTM.
7 Sensitivity Testing – Provides details of the sensitivity testing undertaken on the model.
8 Model Performance, limitations, assumptions, and uncertainty – Provides details any issues encountered.
9 Model runs – Provides details of the design runs carried out for this study.
10 Conclusion and deliverables – Provides and overview of the modelling updates and the project deliverables.

2 Modelling Approach

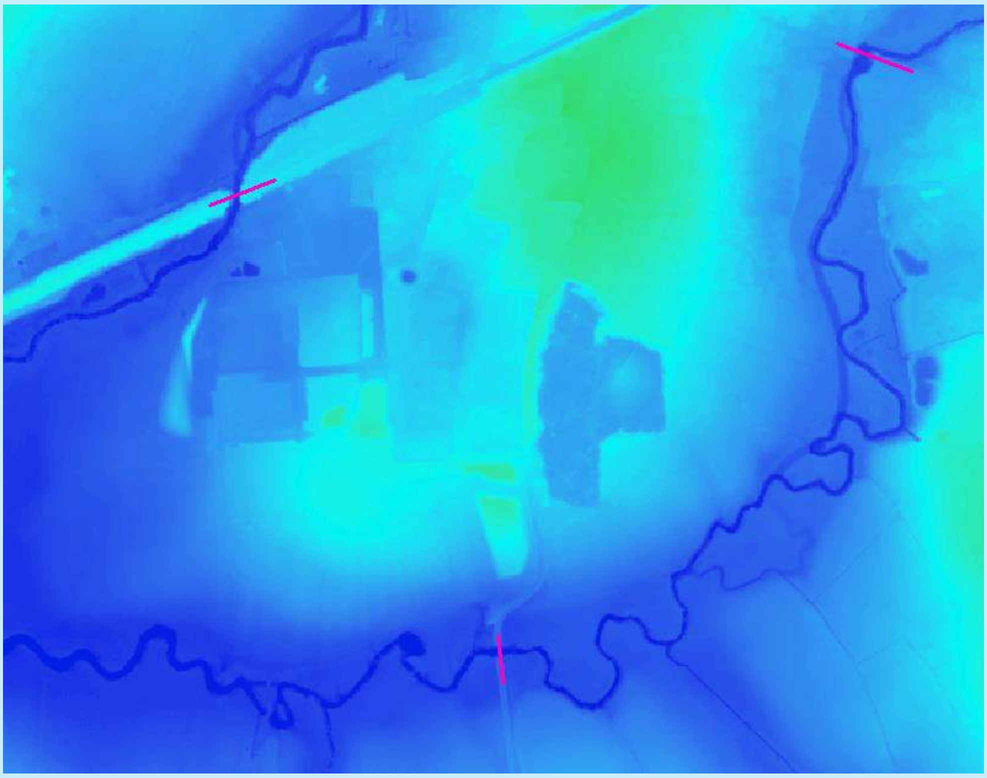
2.1 Model data

Item	Comments
Existing model	This study has utilised the existing EA Areas Benefitting from Defences (ABD) Model that was developed by Capita Symonds in 2006. The was revisited by the EA in 2011 to simulate additional undefended events. The model review undertaken by JBA found that the model is generally robust and extensive. application of downstream boundary conditions. The model was provided by the Environment Agency Product 7 data under the following open source license; Contains Environment Agency information © Environment Agency and/or database right. For this study, the model has been cut back to remove the areas to the north of the Gillingham town as this has no influence on the area of interest. Furthermore the model has been extended downstream to improve the
Cross section survey:	One of the main uncertainties associated with the existing EA model is the cross sectional survey used to represent the 1D channel. The 2006 Capita Symonds model report[2] outlined that the base channel survey was taken from a s105 survey which was collected in 1999 and used in the HEC-RAS modelling of Gillingham in 2000[3]. Given that the survey is now nineteen years old it was decided to collect check survey in order to understand if the existing channel data was still applicable. The EA were unable to provide the 1999 s105 survey that covers the Gillingham model extent, therefore all comparisons were undertaken using the existing model files. The check survey was collected by Maltby Land Surveys in June 2018 at five structures located within the updated model extent. The survey had to be collected at these structures to ensure that the survey was being collected at the exact same locations as previously surveyed. Two structures on the River Lodden and three structures on the River Stour were surveyed. The comparison of these structures showed that the general channel profile had changed very little and therefore it was decided that new channel survey for the full model reach was not required. Detailed analysis of this check survey can be found in Section 2.2. In order to improve the confidence in the model performance especially in relation to the area of interest for this study, the River Stour model extent needed to be extended downstream to ensure that the application of the downstream boundary conditions would not influence flood risk in the area immediately to the south of Gillingham. The Fern Brook watercourse and the Meadow watercourse appear on the current EA flood zone mapping but are not included in the ABD model so

---

[2] Gillingham ABD Final Report, Capita Symonds, November 2006

[3] Hydraulic Study of Gillingham Watercourse, Lewin Fryer and partners, December 2000.

	were likely produced through broadscale JFLOW modelling. Therefore they needed to be modelled in greater detail so new topographical was commissioned to extent the River Stour downstream and represent the Fern Brook and the Meadow watercourses. The topographical survey collected in 2018 has been provided in Appendix A.
LIDAR and other topographical data	The existing model used a 2D zpt layer to represent the base topography within the hydraulic model. This has been updated to use the EA's latest opensource 2m Light Detection and Ranging (LIDAR) Digital Terrain Model (DTM) dataset. The combined LIDAR dataset used within the model was collected in March 2005 and January 2008. Checks on the DTM were undertaken to identify any filtering issues i.e. had any embankments been incorrectly filtered from the LIDAR. This check is essential to ensure that the model grid accurately represents ground conditions. There are some instances where bridge decks have been filtered and required addition to the TUFLOW model. An example of these issues is highlighted in pink in Figure 2-1. The LIDAR has been provided by the Environment Agency as open source data; © Environment Agency copyright and/or database right 2018. All rights reserved.  Figure 2-1: LIDAR filtering issues
Map Data	Ordance Survey (OS) Open Data has been used to produce model figures in this report and the latest OS MasterMap data was purchased from emapsite.com. This has been used within the hydraulic model to update the 2D Manning's n roughness.
Hydrological assessment	A hydrological assessment for the watercourses in the study area was completed by JBA Consulting in this study area for the purpose of this

<table>
<tr>
<td></td>
<td>

project. Detailed description of the process undertaken to generate model inflow hydrographs can be found in the Flood Estimation Handbook (FEH) Calculation record in Appendix B.

Hydrographs have been generated for all watercourses for the following annual exceedance probability (AEP) events: 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1%, 0.5%, 0.2% and 0.1%. Climate change has been applied to the 1% AEP event, based on the latest guidance[4]. For the purpose of this study the Higher Central climate change factor of 40% for the '2080s' epoch has been simulated along with the Upper End '2080s' epoch climate change factor of 85%.

Storm duration testing has been undertaken using the 1% AEP event looking at two ciritcal storm durations. The River Stour and River Lodden exhibited very similar storm durations and therefore a storm duration of 9 hours was adopted for testing. For the smaller catchments of the Fern Brook and the Meadow watercourse the hydrological analysis indicated a critical storm duration of 5.5 hours. Both of these have been tested with analysis of the results outlined in Section 7.3. The tetsing showed that the 9 hour duration could be used for the final design runs.

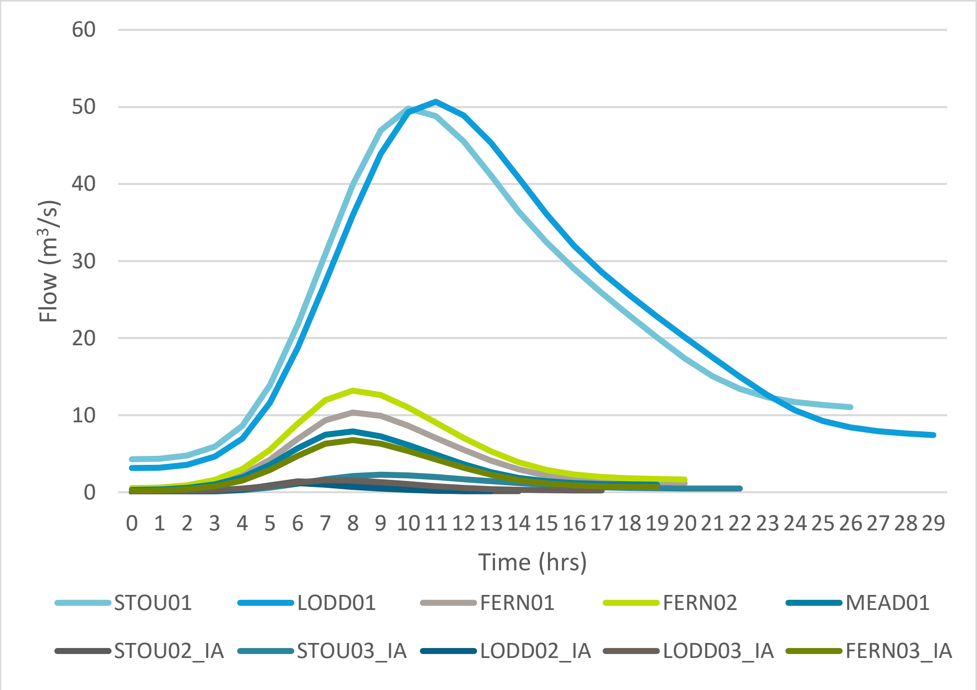


Figure 2-2: 1% AEP fluvial event hydrographs for the 9hr duration

</td>
</tr>
<tr>
<td>Gauge data / Calibration data</td>
<td>

There are no flow gauges located within the updated flood extent so calibration is not feasibale for this study. The EA historical flood map does not show any evidence of flood risk to the area of interest. However this is not surprising considering the area of interest is rural and flood events are rarely recorded for such areas

Gauge data from Colesbrook on the Shreen Water watercourse which is located to the north of Gillingham town has been used in the

</td>
</tr>
</table>

---

[4] https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

	hydrological assessment to improve QMED estimation which is detailed in the FEH Calculation Record in Appendix B.

2.2 Application of check survey

The comparison between the existing model files and the 2018 check survey focused on comparing invert levels, soffit levels, and opening areas. As mentioned in Section 2.1 the s105 survey that is used in the 2006 ABD model was not provided so the model files were used. This section details the differences at each of the structures surveyed.

2.2.1 B3081 (LODD32)

The B3081 bridge on the River Lodden has a total of three openings, the comparison has focused on the two arch bridge structures which are the main online openings rather than the raised bypass culvert. Table 2-1 shows the differences between the 2006 ABD model for the B3081 (LODD32) bridge geometry compared to the 2018 survey. The invert and soffit levels are similar as would be expected, the difference in opening area can be attributed the simplified HW table used in the ABD model.

Table 2-1: B3081 River Lodden structure geometry comparison

	Existing Model		**2018 Survey**		**Difference**	
	Right	Left	Right	Left	Right	Left
Invert Level (mAOD)	67.91	67.91	68.03	68.04	0.12m	0.13m
Soffit Level (mAOD)	70.31	70.31	70.28	70.28	-0.03m	-0.03m
Opening Area (m^2)	7.57	7.57	6.57	6.62	-13.2%	-12.5%

Figure 2-3 shows a comparison between the open channel section at the upstream face of the B3081 River Lodden bridge. This shows a similar channel profile in relation to the width and height. There are minor difference to the bed, however this has no impact when assessing large design events.

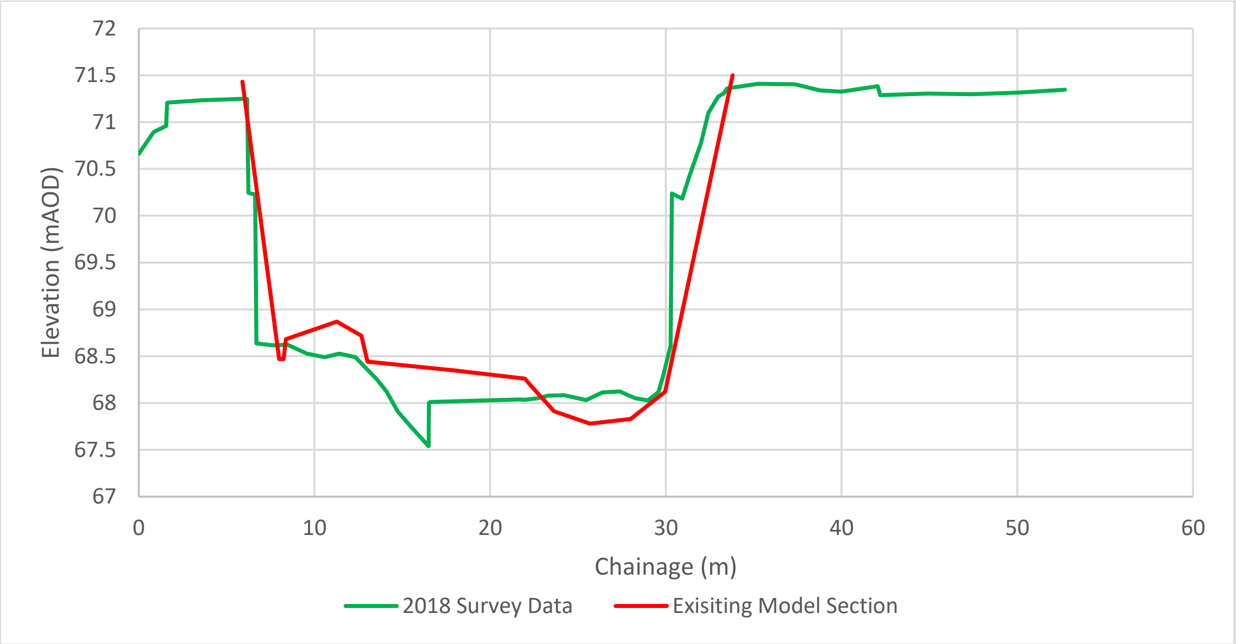


Figure 2-3: B3081 open channel cross section comparison plot

2.2.2 B3092 (LOOD7)

The B3092 bridge on the River Lodden has a total of two online arch bridge openings. Table 2-2 shows the difference in structure geometry between the 2006 ABD model and the 2018 survey for the B3092 bridge. As can be seen the differences between the two are negligible and would have no influence on model results. It is apparent that the previous model simplified the representation of structures by keeping both openings the same geometry even though they are different as shown in the 2018 survey.

Table 2-2: B3092 River Lodden structure geometry comparison

	Existing Model		**2018 Survey**		**Difference**	
	Right	Left	Right	Left	Right	Left
Invert Level (mAOD)	66.19	66.19	66.19	65.97	0	-0.22
Soffit Level (mAOD)	68.59	68.59	68.54	68.47	-0.05	-0.12
Opening Area (m^2)	6.34	6.34	6.37	6.49	-0.5%	-2.4%

Figure 2-4 shows the difference between the open channel section taken from upstream of the B3092 Road Bridge. The bed levels are similar but the general channel profile is considerably different. Unfortunately, due to the EA not having the s105 survey on file this could not be checked in detail. It is assumed that the channel section shown in the existing model isn't taken directly at the upstream face of bridge but is taken further upstream. This correlates to the 2006 channel profile as it does not appear to include the bridge abutments. This is commonplace if the bridge face is particularly constrained due to access and/or health and safety.

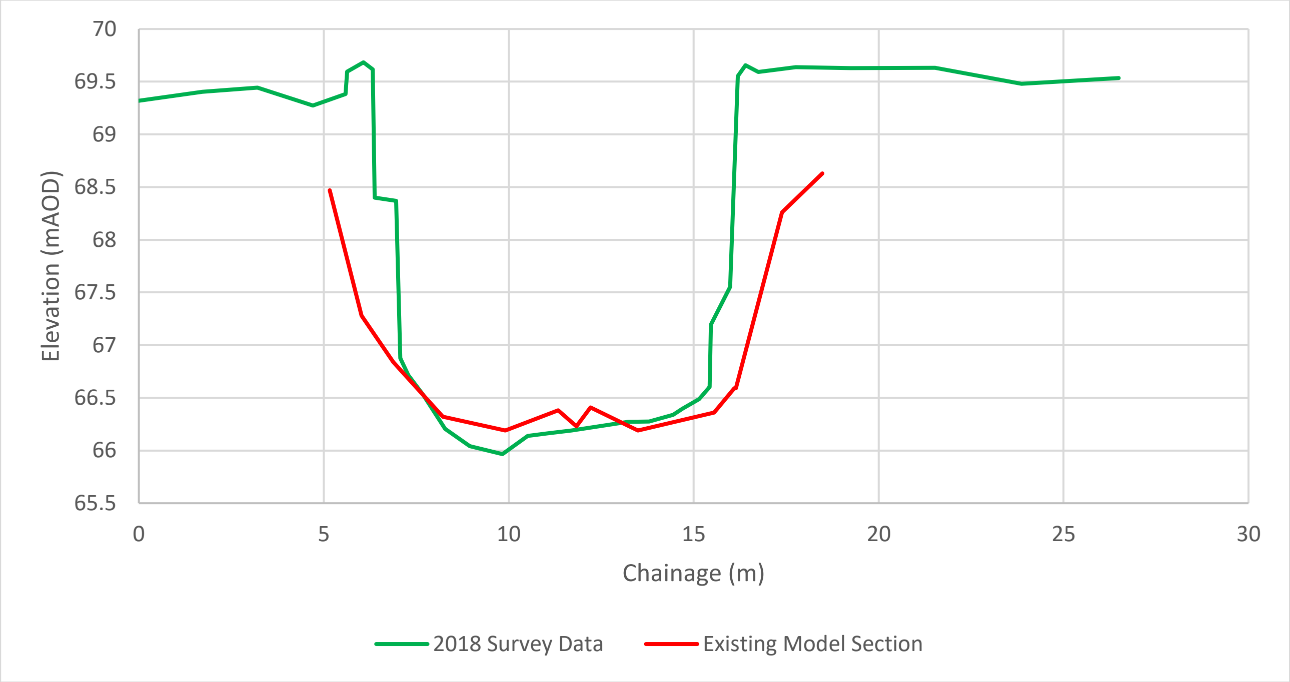


Figure 2-4: B3092 open channel cross section comparison plot

2.2.3 River Stour railway bridge (STOU9.3)

The railway bridge on the River Stour consists of two online arch openings. The 2006 ABD model simplified the representation of these arch structures, and were included in the model as two rectangular culvert units. As shown in Table 2-3, the soffit levels are therefore not comparable as a nominal elevation seems to have been set to provide a representative opening area. The opening area from the 2018 study is smaller than the previous modelling. This is likely to be a result of the application of skew angles. The 2018 survey has highlighted a 36° skew to the upstream face, this skew angle was not detailed in the 2006 modelling report or any of the model layers so was quite likely overlooked in the original modelling.

Table 2-3: River Stour railway bridge

	Existing Model		**2018 Survey**		**Difference**	
	Right	Left	Right	Left	Right	Left
Invert Level (mAOD)	68.63	67.32	68.69	67.65	0.06	0.33
Soffit Level (mAOD)	71.41	71.32	72.25	72.29	0.84	0.93
Opening Area (m^2)	20.85	29.2	17.82	25.45	14.5%	12.8%

The application of the 36° skew angle is most clearly shown when comparing the 2006 modelled open channel section taken from upstream of the railway bridge to the 2018 survey with the skew applied. This shows a clear reduction in opening width with the skew applied as would be expected.

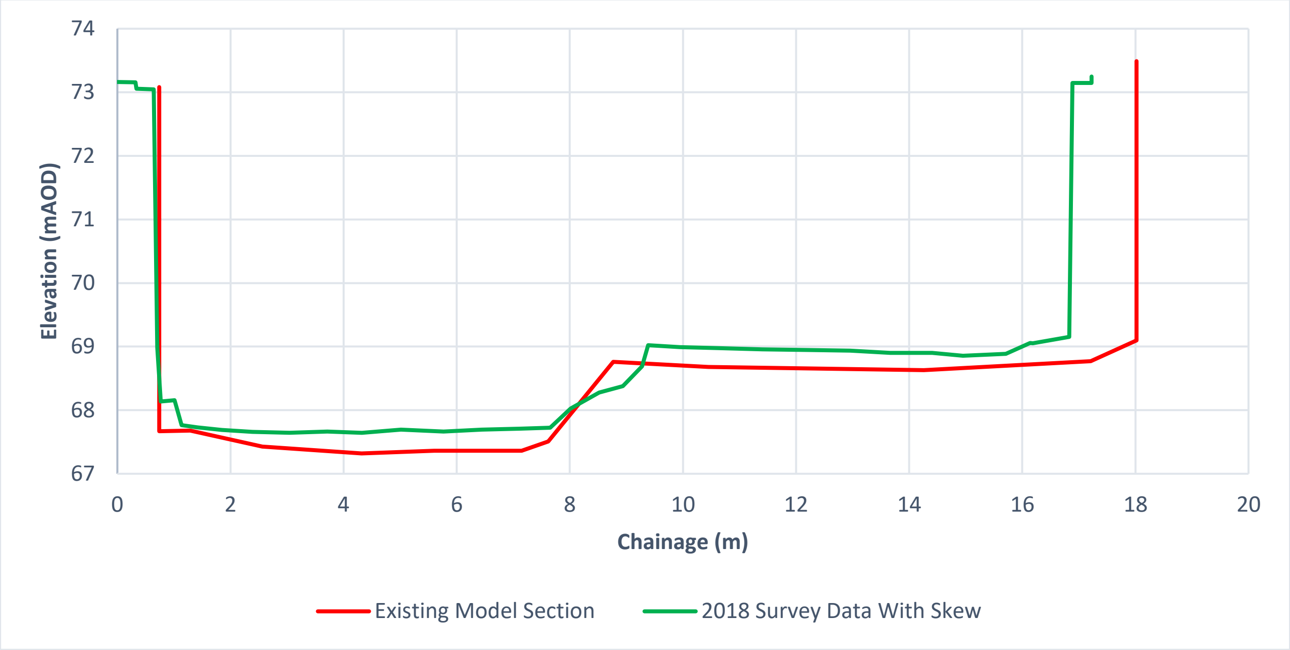


Figure 2-5: River Stour railway bridge open channel comparison

2.2.4 River Stour Nations Rd arch bridges (StouSt_0.2)

The two arch structures are separate bridge units but have been grouped together for this analysis given their close proximity to one another. The 2006 ABD model adopted a simplified approach to their representation using two rectangular culvert units. This approach meant that the invert levels, soffit levels, and opening area provide a particular close match as can be seen in Table 2-4. The soffit levels are incorrect in the previous model but that is to be expected given the simplified approach used.

Table 2-4: River Stour Nations Rd Bridge

	Existing Model		**2018 Survey**		**Difference**	
	Right	Left	Right	Left	Right	Left
Invert Level (mAOD)	64	65.2	64	64.87	0	-0.33
Soffit Level (mAOD)	65.75	67.25	66.42	67.37	0.67	0.12
Opening Area (m^2)	7	10.25	10.77	8.2	-53.9%	20.0%

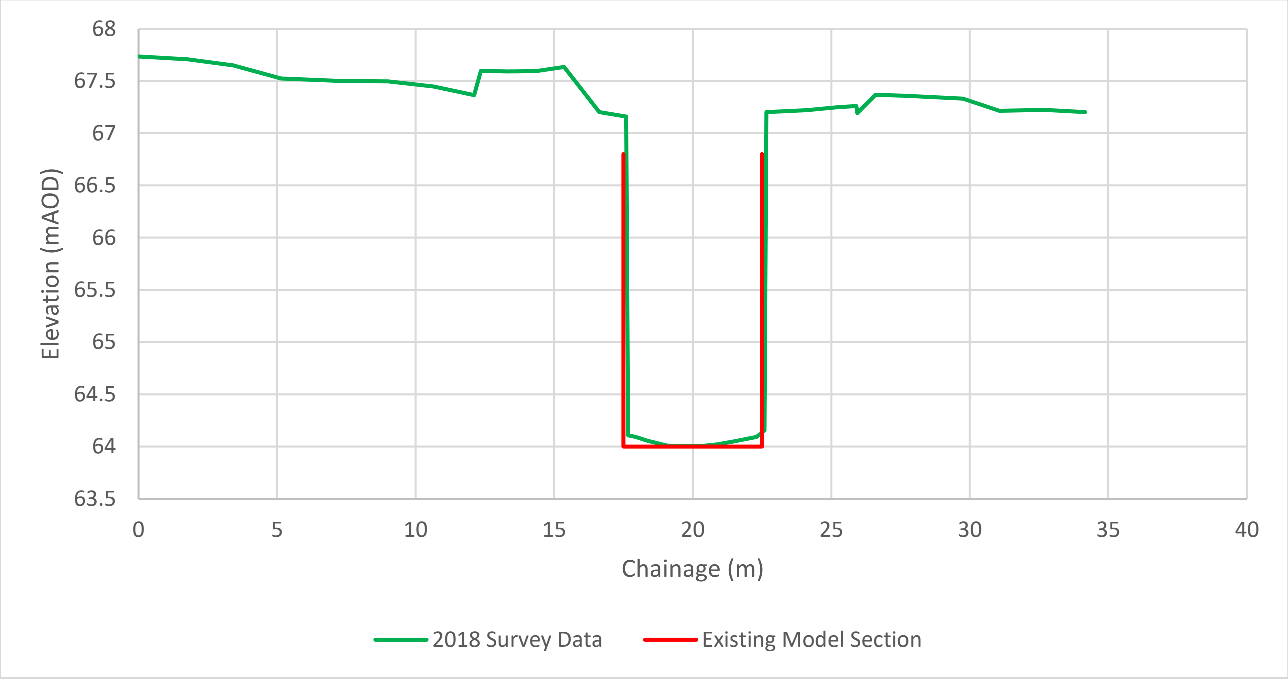


Figure 2-6: River Stour Nations Rd Bridge right hand open channel comparison

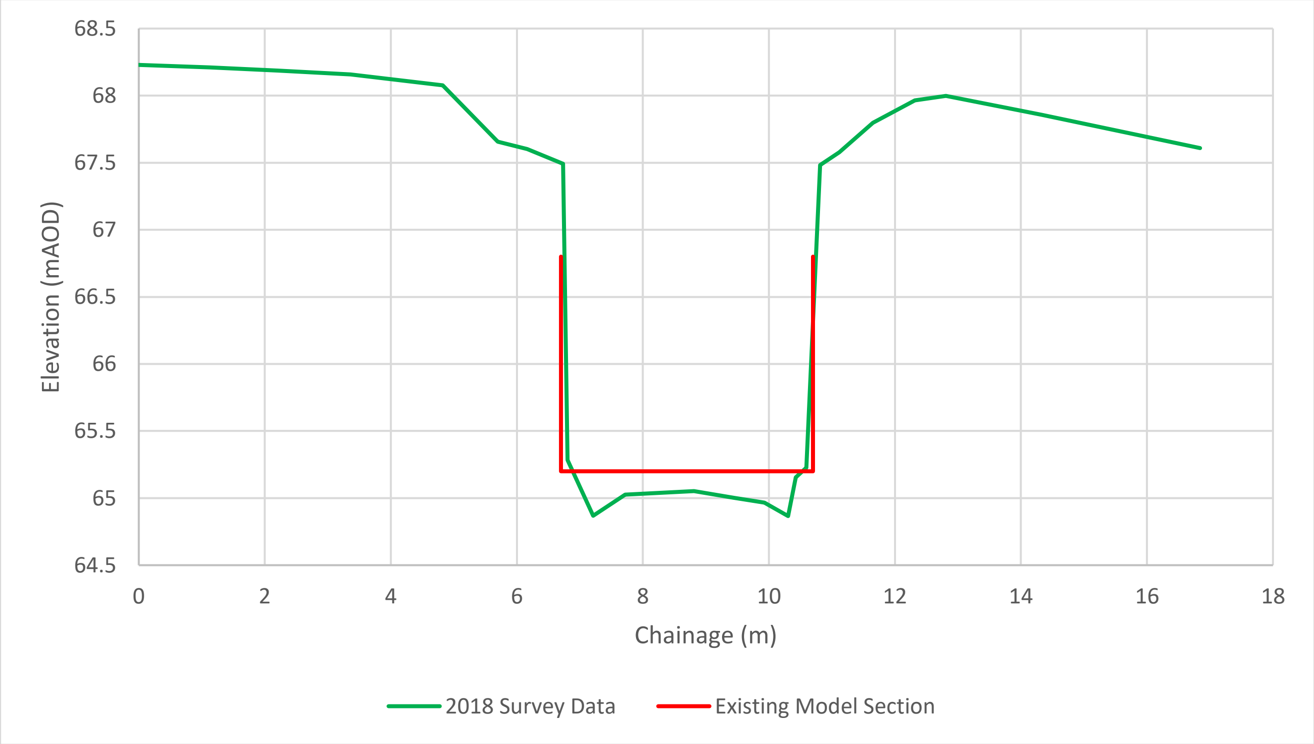


Figure 2-7: River Stour Nations Rd Bridge left hand open channel comparison

Even though the modelled opening areas differ the general shape of the sections provide a good match as can be seen in Figure 2-6 and Figure 2-7 which emphasises that the channel widths haven't changed significantly. The existing model sections shown in Figure 2-6 and Figure 2-7 are square, indicating their simplification for the modelling. Their comparison to the 2018 survey needs to be taken with that in mind.

2.2.5 Conclusion

Overall the comparison between the 2018 check survey and the existing model sections have confirmed that little has changed in terms of channel bed levels geometry. Based on this analysis a full replacement survey for the River Stour and River Lodden concluded to be unnecessary and therefore the data used within the 2006 ABD model has been retained.

2.3 Model schematisation

Item	Notes	Comments
What software & reason for choice:	ESTRY-TUFLOW Linked 1D/2D Model. Version 2018-03-AB.	An ESTRY-TUFLOW model has been developed to represent the River Stour, River Lodden, Fern Brook and Meadow watercourse. The existing model was an ESTRY-TUFLOW model so there was no reason to rebuild the model in a different modelling package.
Grid size selection:	Outline the reasons behind selection of grid sizes for the 2D domain.	A 2m grid resolution has been adopted for this model. The 2006 ABD model used a 5m grid resolution, this was acceptable given its focus on the main rivers of the River Stour, River Lodden, and Shreen Water. With the inclusion of the Fern Brook and the Meadow watercourse the grid size needed to be reduced in order to correctly represent the 1D-2D linkage for these smaller watercourses.
Coefficients:	State documentary sources.	The Manning's n roughness coefficients for culvert structures have been based on CIRIA's 'Culvert Design and Operational guide[5]'. Specific details regarding the roughness coefficients used for each culvert structure can be found in Section 4.
Model Proving:	Outline the test to be applied with the reason, the target accuracy and method of calculation.	Sensitivity analysis has been undertaken for the following model parameters for the 1% AEP undefended model scenario: - Manning's n roughness (1D and 2D) ±20%; - 1D & 2D Downstream boundary±20%; and - Storm duration testing (9hr & 5.5hr).
Any limitations in the method of modelling used:	Existing model data.	The main limitation with this model is the age of the open channel survey used to produce the 1D sections of the River Stour and River Lodden. As detailed in section 2.2 this has been extensively checked and was deemed to be acceptable.

---

[5] CIRIA, (2010). CIRIA C689 – Culvert design and operation guide.

3 Boundary conditions

Item	Model Name	Comments
Inflow Boundaries	STOU01	Upstream model extent of the River Stour. Represented within the model using a single point inflow within the 1d_bc layer at section Stou13.1.
	STOU02_IA	Intervening area hydrograph between STOU01 and STOU02. Inflow hydrograph applied to model using a 1d_bc polygon that splits the inflow over the model sections between StouSt_12 and StouSt_6.
	STOU03_IA	Intervening area hydrograph between STOU02, LODD03, and STOU03. Inflow hydrograph applied to model using a 1d_bc polygon that splits the inflow over the model sections between StouSt_3 and STOU_1176M.
	LODD01	Upstream model extent of the River Lodden. Represented within the model using a single point inflow within the 1d_bc layer at section Lodd_42.1.
	LODD02_IA	Intervening area hydrograph between LODD01, LODD02, and FERN03. Inflow hydrograph applied to model using a 1d_bc polygon that splits the inflow over the model sections between Lodd_35 and Lodd_13.
	LODD03_IA	Intervening area hydrograph between LODD02, MEAD01, and LODD03. Inflow hydrograph applied to model using a 1d_bc polygon that splits the inflow over the model sections between Lodd_10 and Lodd_2.
	FERN01	Upstream model extent of the Fern Brook south channel. Represented within the model using a single point inflow within the 1d_bc layer at section FER2_0284U.
	FERN02	Upstream model extent of the Fern Brook south channel. Represented within the model using a single point inflow within the 1d_bc layer at section FER1_1997U.
	FERN03_IA	Intervening area hydrograph between FERN01, FERN02, and FERN03. Inflow hydrograph applied to model using a 1d_bc polygon that splits the inflow over the model sections between FER1_1772D and FER1_0302D.
	MEAD01	Upstream model extent of the Meadow watercourse east channel. Represented within the model using a single point inflow within the 1d_bc layer at section MEA1_0977U. A single flow

		estimation point was used for the Meadow watercourse at its downstream extent. The flow was then split between the east and west watercourses based on the FEH web service catchment areas. The MEAD01 uses 46% of the total flow.
	MEAD02	Upstream model extent of the Meadow watercourse west channel. Represented within the model using a single point inflow within the 1d_bc layer at section MEA2_0294. A single flow estimation point was used for the Meadow watercourse at its downstream extent. The flow was then split between the east and west watercourses based on the FEH web service catchment areas. The MEAD02 uses 54% of the total flow.
	DSBDY	A stage-discharge boundary (HQ) boundary has been applied at the downstream extent of the model. This has been derived used the ISIS Utility tool – approximate QH boundary based on the geometry of the downstream cross section.
Length of 1D model (km):	The total 1D modelled reach is approximately 10.4km.	
Total number of nodes and structures:	There are 136 1D nodes included in the model. This includes the direct representation of key structures such as bridges and culverts, based on collected survey data.	
Model domain:	There is a single 2D domain used for the Gillingham model. This has a grid resolution of 2m, the previous 2006 ABD model used a 5m grid resolution but given the channel widths of the Meadow watercours and the Fern brook a smaller grid resolution was deemed approapriate. The 2D model domain is approximately 2.99km^2.	
Labelling / numbering system used:	There are two separate model cross section labelling approaches adopted in the Gillingham model. The previous model used numbered sections whereas the newly collected surcey sections have their respective labelling based on chainage. Due to the EA being unable to provide the base survey used within the 2006 model it was decided not to update the labelling for the exitsing channel sections. A suffix of W represents weirs, B represents bridges, C represents circular conduits, I represents irregular culverts, R represents rectangular conduits and X represents connections. Upper case suffixes of U (Upstream), M (Middle) and D (Downstream) indicate where a channel section was split longitudinally. On multiple opening structures, suffixes of L (Left), M (Middle) and R (Right) have also been used where section was split horizontally.	
Hydraulic roughness values used:	Manning's n roughness coefficients are required for 1D channels and culverts, and the 2D model domain. Hydraulic roughness coefficient	

	values for the model channels were sourced from Chow [6](1973). The application of Manning's n roughness coefficients to the 1D and 2D model domains has differing effects and, as such, Manning's n values are not directly transferable between domains. 1D water levels are typically more sensitive to Manning's n. Consideration should also be given to 2D grid sizes prior to selection of Manning's n for 2D domain areas. 1D channel Manning's n coefficients were based on survey photographs. Based on this evidence, the channel bed and banks were split into a number of different classifications each with different roughness characteristics. These roughness classifications and the Manning's n coefficient values selected for each of these zones are outlined in Table 3-1. For each 1D channel section the hard bed geometry has been adopted when soft and hard bed levels have been provided on the survey. Descriptions of the channel in the table were compared to photographs collected during the Maltby Land Survey survey collection to obtain the Manning's n coefficients.

---

[6] Chow V T. 'Open Channel Hydraulics'. International Edition, 1973. McGraw-Hill.

Table 3-1: **1D Manning's n values**

Material Code	**1D Manning's n**	Comment / Example
100	0.040	Channel bed: Natural stream with a bed of rocks and cobbles
101	0.040	Channel banks: Vegetated bank, materials based on grass and light brush
102	0.025	Channel banks: Masonry
103	0.014	Channel banks: Concrete
104	0.018	Channel banks: Brick
105	0.020	Channel banks: Road or footpath
106	0.033	Channel bed: Natural stream that has some boulders
107	0.020	Channel bank: bank of very light vegetation
108	0.070	Channel banks: Dense shrubs and trees
109	0.030	Channel bed: Natural stream with a channel bed typified by a silt with some rocks and a little vegetation
110	0.020	Channel banks: maintained grass
111	0.035	Channel bed: Natural stream with a channel bed typified by rocks and a little vegetation
112	0.040	Channel banks: Trees with minimal understorey
113	0.050	Channel banks: unmaintained grass with some scrub and reeds

Within the 2D domain a generalised value of 0.05 was applied for the Manning's n coefficient value across the entire area. Key floodplain features were then identified using OS MasterMap data to provide a more physically reasonable representation of these features, such as roads, vegetation, gardens and pathways. OS MasterMap data was compared to aerial imagery to ensure the accuracy of the data was fit for purpose across the study region. Manning's n values were assigned based on the feature code attribute of OS MasterMap, for example, where the feature code was 10172, this value represented roads in OS MasterMap and was given a Manning's n value of 0.020. 2D Manning's n roughness coefficients were selected based on previous modelling experience and internal JBA guidance and are provided in Table 3-2.

Table 3-2: 2D Manning's n values

Material Code	**1D Manning's n**	Comment / Example
999	0.050	Typical value for 2D domain
10183	0.025	Roads, tracks and paths
10096	0.040	Man made landform
10099	0.050	Natural Landform
10093	0.045	Combined manmade/natural landforms
10193	0.060	Structures
10119	0.025	Roads tracks and paths
10021	0.300	Buildings
10053	0.050	Gardens
10054	0.050	General Surface
10056	0.050	Grassed areas (including fields and agricultural land)
10057	0.050	Grassed areas (including fields and agricultural land)
10062	0.100	Glasshouse
10089	0.040	Inland Water
10111	0.080	Scrub and rough grassland
10123	0.020	Paths from Master Map
10167	0.040	Railway line
10172	0.020	Roads, Tracks and Pavements
10185	0.030	Structures
10203	0.040	Tidal Waters Foreshore
10210	0.040	Tidal Waters
10217	0.040	Land for Development
1000	0.1	Model stability

Manning's n coefficients for culverts within the model were determined for each structure individually, as described in Section 4

	of this report. The structure roughness values were sourced from Table A1.2 from CIRIA's 'Culvert Design and Operational guide'.
Manning's n: 0.040 Material code: 101 Channel banks: Vegetated bank, materials based on grass and light brush.	
Manning's n: 0.025 Material code: 102 Channel banks: Masonry.	
Manning's n: 0.070 Material code: 108 Channel banks: Dense shrubs and trees.	

Manning's n: 0.030 Material code: 109 Channel bed: Natural stream with a channel bed typified by a silt with some rocks and a little vegetation.	
Manning's n: 0.020 Material code: 110 Channel banks: maintained grass.	
Manning's n: 0.035 Material code: 111 Channel bed: Natural stream with a channel bed typified by rocks and a little vegetation.	

Manning's n: 0.050 Material code: 113 Channel banks: unmaintained grass with some scrub and reeds.	
Material codes 100, 103, 104, 105, 106, 107, and 112 have not been used within this study.	

4 Structures

This section deals with all structures within the model extent. A table is provided for all structures (bridge, culvert, and weirs). Any assumptions made in the modelling of structures are recorded on the following pages. All structures within the model have been updated using 2018 survey.

4.1 River Stour structures

Name of Structure:	River Stour railway bridge
Location (NGR):	380658, 125961
Included in model (state reason if not):	Yes
Model Label:	Stou9.3BL & Stou9.3BR
Type:	'BB' Bridge units
How has the structure been modelled?	Twin ESTRY Bridge unit with HW tables to specify structure geometry. Invert Level = 67.65mAOD (L) 68.69mAOD (R) Soffit Level = 72.29mAOD (L) 72.25mAOD (R) Deck Level = 73.62mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure. The bridge is 20.70m in length with overtopping represented within the 2D domain. It was decided to represent each opening using a separate 1d_nwke due to the differences in invert level. The right hand opening is perched and therefore dry during baseflow conditions. A skew angle of 36° has been applied to the 1d_cs and 1d_xs tables. Due to the influence of piers a form loss value of 0.2 has been applied to each of the bridge openings.
Source of the survey data:	2018 check survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

Name of Structure:	Nations Road Left hand bridge
Location (NGR):	379819, 125296
Included in model (state reason if not):	Yes
Model Label:	Stou_0.2BL
Type:	'BB' Bridge unit
How has the structure been modelled?	ESTRY Bridge unit with HW table to specify structure geometry. Invert Level = 64.87mAOD Soffit Level = 67.37mAOD Deck Level = 67.82mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure using the TUFLOW recommended form loss value of 0.001. The bridge is 7m in length with overtopping represented within the 2D domain.
Source of the survey data:	2018 check survey, Maltby Land Surveys.
 Upstream Face	No photograph of Downstream Face

Name of Structure:	Nations Road Right hand bridge
Location (NGR):	379819, 125313
Included in model (state reason if not):	Yes
Model Label:	Stou_0.2BR
Type:	'BB' Bridge unit
How has the structure been modelled?	ESTRY Bridge unit with HW table to specify structure geometry. Invert Level = 64.00mAOD Soffit Level = 66.42mAOD Deck Level = 67.29mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure using the TUFLOW recommended form loss value of 0.001. The bridge is 5.3m in length with overtopping represented within the 2D domain. Typically the bridge overtopping would be represented within the 1D domain given the 2m grid size. However it was felt that given the close proximity to Stou_0.2BL, it was important to keep the overtopping approach consistent.
Source of the survey data:	2018 check survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

Name of Structure:	Nations Road access bridge
Location (NGR):	379592, 125357
Included in model (state reason if not):	Yes
Model Label:	STOU_1554BL & STOU_1554BR
Type:	'BB' Bridge unit with separate overtopping weir 'WW' unit.
How has the structure been modelled?	Twin ESTRY Bridge unit with HW tables to specify structure geometry. Invert Level = 63.32mAOD (L) 62.86mAOD (R) Soffit Level = 66.18mAOD (L) 66.23mAOD (R) Deck Level = 67.04mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure. Due to the influence of piers a form loss value of 0.25 has been applied to each of the bridge openings. The bridge is 3.59m in length with overtopping represented within the 1D domain by using a separate weir unit. As the overtopping geometry is uneven a separate 1d_nwke weir unit is required. The overtopping geometry has been defined using an XZ table. An ESTRY weir coefficient of 0.69 has been applied to represent the inefficient nature of metal railings on the bridge parapet.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

4.2 River Lodden structures

Name of Structure:	B3081 Road bridge
Location (NGR):	381476, 126123
Included in model (state reason if not):	Yes
Model Label:	Lodd_32BL, Lodd_32BM, & Lodd_32BR
Type:	'BB' Bridge units
How has the structure been modelled?	Twin arch ESTRY Bridge unit with HW tables to specify structure geometry and a separate BB bridge unit to represent the rectangular left hand side opening. Invert Level = 68.35mAOD (Bridge Left) 68.04mAOD (Bridge Middle) 68.03mAOD (Bridge Right) Soffit Level = 70.32mAOD (Bridge Left) 70.28mAOD (Bridge Right and Middle openings) Deck Level = 71.39mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure. Due to the influence of piers a form loss value of 0.25 has been applied to each of the bridge openings. This form loss value is based on the opening area ratio and the pier area. The bridge is 11.3m in length with overtopping represented within the 2D domain.
Source of the survey data:	2018 check survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

Name of Structure:	Wren Place Footbridge
Location (NGR):	381434, 125902
Included in model (state reason if not):	Yes
Model Label:	LODD_0027B & LODD_0027W
Type:	‘BB’ Bridge unit with a separate 1D overtopping ‘WW’ weir unit.
How has the structure been modelled?	It appears this structure has been built since the development of the previous modelling as it was not included but has been surveyed and added for this study. ESTRY Bridge unit with HW table to specify structure geometry and a Weir unit using an XZ table to represent the bridge overtopping. Invert Level = 67.43mAOD Soffit Level = 71.98mAOD Deck Level = 71.83mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure using the TUFLOW recommended form loss value of 0.001. The bridge is 2.64m in length with overtopping represented within the 1D domain using a separate 1d_nwke weir unit. This was required due to the arch shape bridge deck. An ESTRY weir factor of 0.69 was applied to represent the inefficient nature of the bridge railings. The survey highlights two small openings either side of the main bridge opening. It was decided not to represent these in the floodplain as they would have little to no impact on the modelled flood risk.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

Name of Structure:	B3092 Road bridge
Location (NGR):	380993, 125349
Included in model (state reason if not):	Yes
Model Label:	Lodd_7BL & Lodd_7B
Type:	'BB' Bridge units.
How has the structure been modelled?	Twin ESTRY Bridge unit with HW tables to specify structure geometry. Invert Level = 65.97mAOD (L) 66.19mAOD (R) Soffit Level = 68.47mAOD (L) 68.54mAOD (R) Deck Level = 69.66mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure using the TUFLOW recommended form loss value of 0.001. The bridge is 7.9m in length with overtopping represented within the 2D domain. Due to the influence of piers a form loss value of 0.15 has been applied to each of the bridge openings.
Source of the survey data:	2018 check survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

4.3 Fern Brook structures

Name of Structure:	Fern Brook access bridge 1
Location (NGR):	382699, 125745
Included in model (state reason if not):	Yes
Model Label:	FER1_1580B & FER1_1580B
Type:	'BB' Bridge unit with separate overtopping 'WW' weir unit.
How has the structure been modelled?	ESTRY Bridge unit with HW table to specify structure geometry with a separate 1d_nwke weir using an XZ table to represent its geometry. Invert Level = 73.70mAOD Soffit Level = 75.72mAOD Deck Level = 75.95mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure using the TUFLOW recommended form loss value of 0.001. This has been modelled as bridge due to the height length ratio of 1:1.64. The bridge is 3.61m in length with overtopping represented within the 1D domain using a separate 1d_nwke overtopping unit. This was required due to the sloped geometry of the bridge deck. An ESTRY weir factor of 0.88 has been used to represent the inefficient nature of a bridge deck.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

Name of Structure:	Fern Brook access bridge 2
Location (NGR):	382264, 126161
Included in model (state reason if not):	Yes
Model Label:	FER1_0561B & FER1_0561W
Type:	'BB' Bridge unit with separate overtopping 'WW' weir unit.
How has the structure been modelled?	ESTRY Bridge unit with HW table to specify structure geometry with a separate 1d_nwke weir using an XZ table to represent its geometry. Invert Level = 70.56mAOD Soffit Level = 72.09mAOD Deck Level = 72.07mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure using the TUFLOW recommended form loss value of 0.001. This has been modelled as bridge due to the height length ratio of 1:2.4. The bridge is 3.72m in length with overtopping represented within the 1D domain using a separate 1d_nwke overtopping unit. This was required due to the uneven geometry of the bridge deck. An ESTRY weir factor of 0.88 has been used to represent the inefficient nature of a bridge deck.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

Name of Structure:	Fern Brook access bridge 3
Location (NGR):	381747, 126212
Included in model (state reason if not):	Yes
Model Label:	FER1_0024B & FER1_0024W
Type:	'BB' Bridge unit with separate overtopping 'WW' weir unit.
How has the structure been modelled?	ESTRY Bridge unit with HW table to specify structure geometry with a separate 1d_nwke weir using an XZ table to represent its geometry. Invert Level = 69.10mAOD Soffit Level = 71.01mAOD Deck Level = 71.34mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure using the TUFLOW recommended form loss value of 0.001. This has been modelled as bridge due to the height length ratio of 1:2.7. The bridge is 5.10m in length with overtopping represented within the 1D domain using a separate 1d_nwke overtopping unit. This was required due to the sloped geometry of the bridge deck. An ESTRY weir factor of 0.69 has been used to represent the inefficient nature of a bridge deck and metal railings. A skew angle of 25° has been applied to the 1d_cs and 1d_xs tables.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

Name of Structure:	Fern Brook access bridge 4
Location (NGR):	382830, 125693
Included in model (state reason if not):	Yes
Model Label:	FER2_0064B & FER2_0064W
Type:	'BB' Bridge unit with separate overtopping 'WW' weir unit.
How has the structure been modelled?	ESTRY Bridge unit with HW table to specify structure geometry with a separate 1d_nwke weir using an XZ table to represent its geometry. Invert Level = 75.22mAOD Soffit Level = 76.04mAOD Deck Level = 76.06mAOD The 2016 TUFLOW automated structure losses approach has been adopted for this structure using the TUFLOW recommended form loss value of 0.001. This has been modelled as bridge due to the height length ratio of 1:4.6. The bridge is 3.75m in length with overtopping represented within the 1D domain using a separate 1d_nwke overtopping unit. This was required due to the uneven geometry of the bridge deck. An ESTRY weir factor of 0.88 has been used to represent the inefficient nature of a bridge deck.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

4.4 Meadow watercourse structures

Name of Structure:	Cole Street Lane Culvert 1
Location (NGR):	381851, 125039
Included in model (state reason if not):	Yes
Model Label:	MEA1_0811C
Type:	'C' Circular culvert unit.
How has the structure been modelled?	ESTRY Circular culvert unit. Invert Level = 72.29mAOD Soffit Level = 73.04mAOD Deck Level = 73.42mAOD Culvert diameter = 0.75m The culvert is 11.34m in length with overtopping represented within the 2D domain. A Manning's n roughness value of 0.018 has been chosen. This has been based on Table A1.2 from CIRIA's 'Culvert Design and Operational guide' which specifies that a brickwork culvert should have a manning roughness value between 0.012 and 0.018. The default entry and exit losses for a circular culvert have been used with the width contraction coefficient set to 1. The entry loss coefficient set to 0.5 and the exit loss coefficient set to 1.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

Name of Structure:	Meadow Access Culvert 1
Location (NGR):	381490, 125183
Included in model (state reason if not):	Yes
Model Label:	MEA1_0412C
Type:	'C' Circular culvert unit.
How has the structure been modelled?	ESTRY Circular culvert unit. Invert Level = 69.34mAOD Soffit Level = 70.04mAOD Deck Level = 70.55mAOD Culvert diameter = 0.7m The culvert is 5.32m in length with overtopping represented within the 2D domain. A Manning's n roughness value of 0.018 has been chosen. This has been based on Table A1.2 from CIRIA's 'Culvert Design and Operational guide' which specifies that a brickwork culvert should have a Manning's roughness value between 0.012 and 0.018. The default entry and exit losses for a circular culvert have been used with the width contraction coefficient set to 1. The entry loss coefficient set to 0.5 and the exit loss coefficient set to 1.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

Name of Structure:	Meadow Access Culvert 2 & Sluice
Location (NGR):	381384, 125174
Included in model (state reason if not):	Yes
Model Label:	MEA1_0272C, MEA1_0272W, & MEA1_0273W
Type:	'C' Circular culvert unit with a separate overtopping 'WW' weir unit and additional 'WW' unit to represent the upstream sluice.
How has the structure been modelled?	ESTRY Circular culvert unit with a separate 1d_nwke weir using an XZ table to represent its geometry. The upstream sluice structure is represented as a weir unit with and XZ table used to represent the uneven flume shape. Based on photographs the sluice does not appear to operational with broken wooden boards at the culvert inlet, therefore it is not acting as a sluice and a weir unit was identified to be the most appropriate way to represent the structure. Invert Level = 68.69mAOD Soffit Level = 69.29mAOD Deck Level = 69.48mAOD Sluice (weir) Invert Level = 69.07mAOD Culvert diameter = 0.6m The culvert is 6.32m in length with overtopping represented within the 1D domain. Given its length, the overtopping was originally modelled in the 2D domain but this led to instabilities are therefore was changed to the 1D. A Manning's n roughness value of 0.015 has been chosen. This has been based on Table A1.2 from CIRIA's 'Culvert Design and Operational guide' which specifies that a smooth inner wall plastic culvert should have a Manning's roughness value between 0.009 and 0.015. The default entry and exit losses for a circular culvert have been used with the width contraction coefficient set to 1. The entry loss coefficient set to 0.5 and the exit loss coefficient set to 1.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.

Upstream Face

Downstream Face

Dilapidated Upstream Sluice

Name of Structure:	Cole Street Lane Culvert 2
Location (NGR):	381499, 124935
Included in model (state reason if not):	Yes
Model Label:	MEA2_0231C
Type:	'C' Circular culvert unit.
How has the structure been modelled?	ESTRY Circular culvert unit. Invert Level = 69.62mAOD Soffit Level = 70.17mAOD Deck Level = 71.05mAOD Culvert diameter = 0.55m The culvert is 9.21m in length with overtopping represented within the 2D domain. A Manning's n roughness value of 0.013 has been chosen. This has been based on Table A1.2 from CIRIA's 'Culvert Design and Operational guide' which specifies that a metal pipe culvert should have a Manning roughness value between 0.012 and 0.013. The default entry and exit losses for a circular culvert have been used with the width contraction coefficient set to 1. The entry loss coefficient set to 0.5 and the exit loss coefficient set to 1.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.

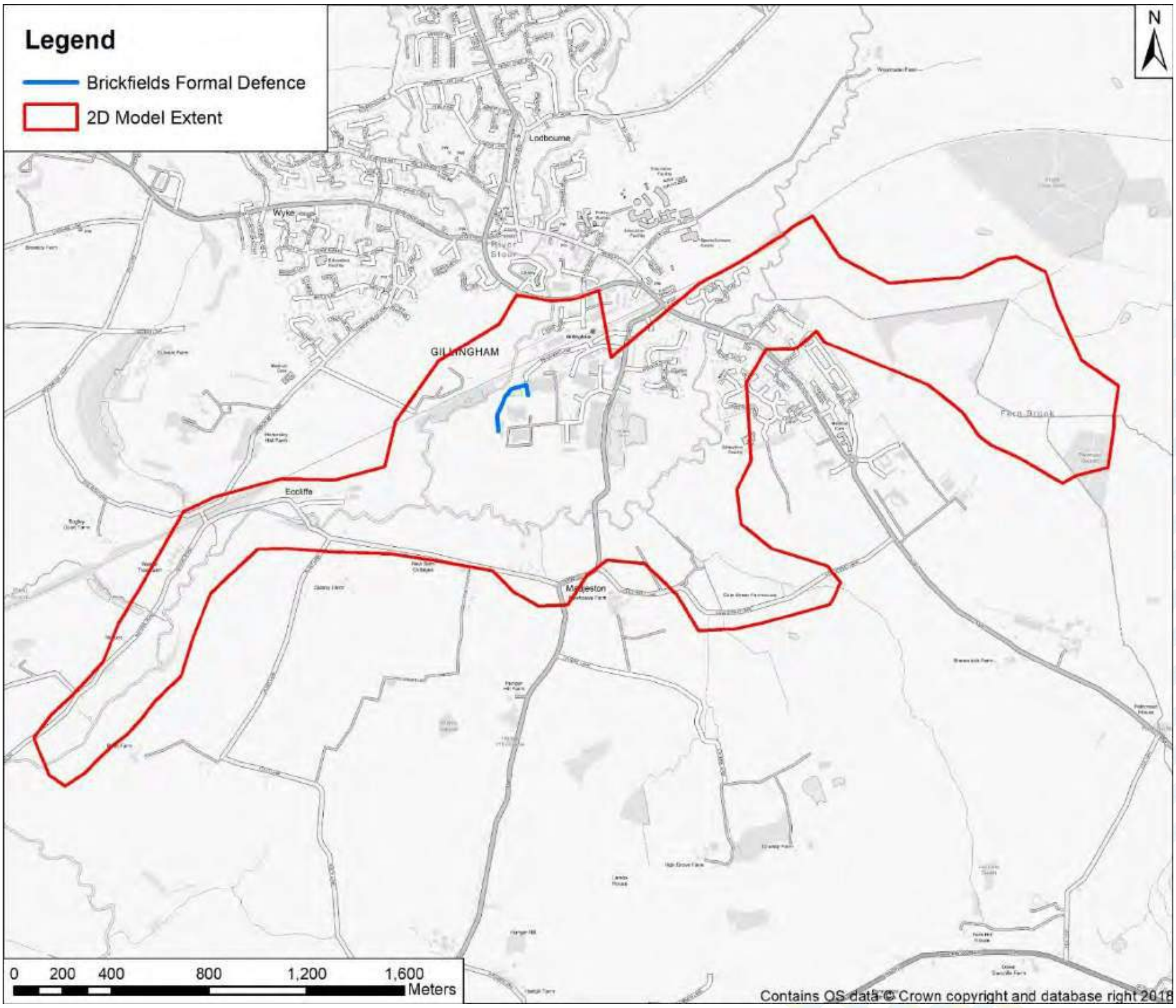
Upstream Face

Downstream Face

Name of Structure:	Meadow Access Culvert 3
Location (NGR):	381429, 125126
Included in model (state reason if not):	Yes
Model Label:	MEA2_0027C
Type:	'C' Circular culvert unit.
How has the structure been modelled?	ESTRY Circular culvert unit. Invert Level = 68.76mAOD Soffit Level = 69.46mAOD Deck Level = 69.73mAOD Culvert diameter = 0.7m The culvert is 6.53m in length with overtopping represented within the 2D domain. A manning's n roughness value of 0.011 has been chosen for the upstream culvert. This has been based on Table A1.2 from CIRIA's 'Culvert Design and Operational guide' which specifies that a precast concrete pipe culvert should have a manning roughness value between 0.010 and 0.011. The default entry and exit losses for a circular culvert have been used with the width contraction coefficient set to 1. The entry loss coefficient set to 0.5 and the exit loss coefficient set to 1.
Source of the survey data:	2018 additional survey, Maltby Land Surveys.
 Upstream Face	 Downstream Face

5 Flood defences

The formal flood defences that the EA has located within the modelled study area have been retained from the previous Gillingham ABD study developed in 2006. A number of these have been removed due to the reduced model extent for the purpose of this study. The only formal flood defence that remains in the updated model extent is the Brickfields Business Park Embankment which is located immediately to the west of the business park. This defence is a 3m high embankment that is approximately 270m in length. The EA did not provide any indication that new flood defence works have been constructed since the additional EA modelling work undertaken in 2011 that focused on simulating the undefended model scenario for the full suite of events. The location of the Brickfields formal flood defence is shown in Figure 5-1.



Contains OS data © Crown copyright and database right 2018

Figure 5-1: Brickfields formal flood defence location

6 Topographic Modifications

A series of topography modifications were applied to the model to modify the grid to enable a more detailed representation of floodplain features. Table 6-1 provides details of these modifications.

Table 6-1: Topographic modifications

Topography modification command	Purpose of command
2d_zpt_Gil_Brickfields_072	2D Zpt layer that has been retained from the previous 2006 ABD model for the undefended model scenario that lowers any remnants of the Brickfields formal defence embankment from the base LIDAR. The elevations have been retained from the previous modelling, though the layer was widended to ensure that any remnants of the defence in the updated base LIDAR were correctly removed.
2d_zln_Gil_068	This 2D Z-line layer that has been retained from the previous 2006 ABD model. This layer has been cut back to cover the new model extent. The Z-line is used to define bank crest elevations for the existing model reaches of the River Stour and River Lodden. The elevations have been retained from the previous modelling.
2d_zsh_bridge_decks_066	This 2D Z-shape layer has been developed for the updated 2018 model. Due to the model grid size being reduced from 5m to 2m the structure overtopping schematisation has been reviewed for all structures and updated where necessary. Where deemed appropriate this Z-shape layer specifies the elevations of bridge decks represented in the 2D domain. Elevations have been taken from the 2018 topographical survey.
2d_zln_GIll_Bridge_Parapets_072	This 2D Z-line layer has been used to represent the elevations of solid bridge parapets to ensure the 2D overtopping spill level is corrcet as based on the 2018 survey.
2d_zln_Gil_Weir_070	This Z-line has been retained from the 2006 ABD study but cut back to the new model extent. This Z-line represents ground raising adjacent to the B3092 Road Bridge on the River Lodden and the Nations Road Bridge on the River Stour.
2d_zln_Gil_Brickfields_001	This Z-line is being read into the defended model to represent the formal flood defence embankments at the Brickfields business park.
2d_zln_Bellway_072	This Z-line has been retained from the 2006 ABD study. The 2006 ABD report does not indicate that this relates to a formal defence structure but instead reflects detailed topographical survey that was collected at this location. Considering this is located on the River Stour upstream of the railway embankment it should have no influence on the flood risk to the area of interest.

7 Sensitivity testing

Sensitivity testing allows for greater understanding of the impact of various assumptions made during model development. The model proving involved sensitivity analysis on the parameters and inputs detailed in Table 7-1.

Table 7-1: Sensitivity analysis parameters

Parameter	Variance
Manning's n roughness coefficient	±20% change in roughness applied to 1D and 2D model domains.
Downstream boundary	±20% change 1D and 2D downstream boundary conditions.
Storm duration	Simulated the 1% AEP event using the two critical storm durations identified form the hydrological analysis (9 hour & 5.5 hour).

The sensitivity of the model to changes in these parameters was assessed in terms of both maximum flood extent and water level. The 1% AEP fluvial event (undefended scenario) was used as the baseline for all sensitivity tests. The run reference is the same for the Manning’s n roughness and storm duration testing (run version 069). During the model development the downstream boundary conditions were updated and therefore that sensitivity analysis was re-simulated and based on run version 073. The sensitivity and significance of a parameter change was considered in terms of the change in water depth, flood extent and impact on potential receptors.

7.1 Manning’s n roughness

Hydraulic roughness values of the channel and floodplain are represented by specifying a Manning's n roughness coefficient within the model. The value of the Manning's n roughness coefficient varies throughout the model and is based on established reference texts and channel survey. Roughness characteristics in the 2D domain representing the floodplain were defined using OS MasterMap data. Each key land use type was then assigned a Manning's n roughness coefficient, to test the roughness the coefficients have been increased in both the 1D and 2D domains. The 1D roughness sensitivity test has only been applied to the 1D open channel sections. It was decided not to change the 1D roughness for the modelled culvert units as this commonly leads to model instabilities in particular the reduction in Manning’s n.

The results of the model sensitivity testing showed that:

- In general, an increase in Manning's n roughness coefficient results in increased water levels as the conveyance of the channel is decreased, the increase in water level ranges from 5mm to 170mm with the decrease in water level ranging from -3mm to -194mm. This results in greater out of bank flooding compared to the baseline model. Conversely, decreasing the Manning’s n value increases the conveyance of the channel and results in decreased water levels in comparison to the baseline scenario.
- Figure 7-1 shows the difference in flood outlines between the two Manning’s n roughness sensitivity tests and the baseline model for the 1% AEP event. As can be seen the modelled outlines only exhibit minor differences which indicates that model outputs are not sensitive the Manning’s n roughness parametrisation.

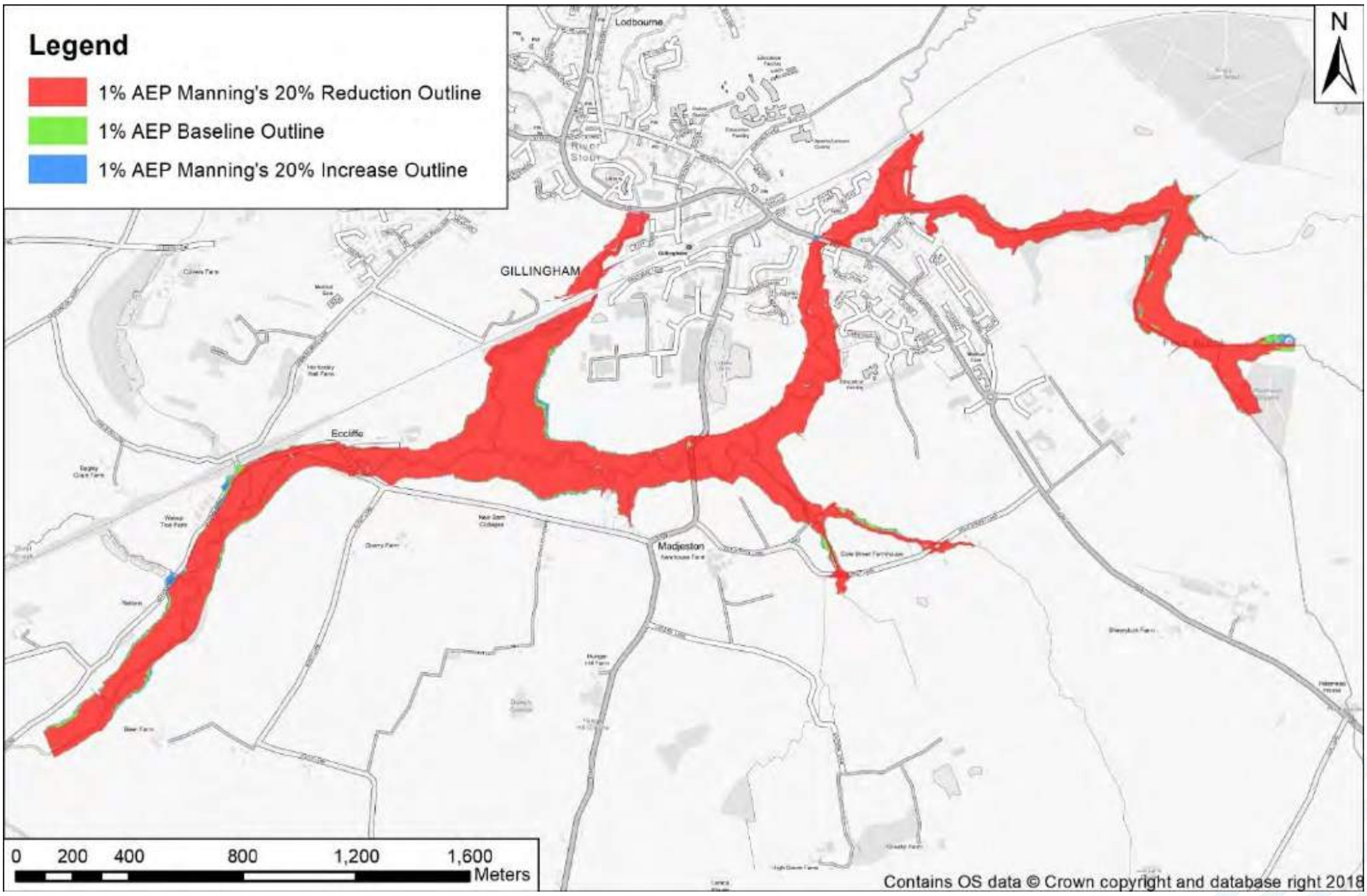


Figure 7-1: Manning's n sensitivity testing - changes to flood extents

7.2 Downstream boundary

Sensitivity of the 1D and 2D downstream boundary conditions has been undertaken to ascertain the potential influence the downstream boundary is having on model results within the study area. The downstream boundary sensitivity analysis was based on a more recent version of the model compared to manning's and storm duration testing due to changes made to the downstream boundary during the model development. In the 1D domain an HQ boundary has been used which was derived using the ISIS (now known as Flood Modeller) 'Approximate QH boundary' utility tool. To sensitivity test the 1D boundary the flow element has been increased and decreased by 20% There are two 2D HQ downstream boundaries located either side of the main 1D channel. The stage-discharge curves have been generated automatically in TUFLOW using the 'a' slope value. To test the sensitivity of the 2D boundaries, the 'a' slope value was increased and decreased by 20%.

The results of the sensitivity showed that:

- The application of the downstream boundary is confined to an area approximately 1.7km downstream of the area of interest. Figure 7-2 highlights the difference in water levels between the two sensitivity tests and the baseline scenario. This shows that increasing and decreasing the flow element of the stage discharge boundary has influence on water levels approximately 0.7km upstream of the downstream model extent.
- The previous modelling used a HT boundary at the downstream extent of the model with channel sections extracted from LIDAR, this was a simplified approach that has been improved significantly with the inclusion of surveyed open channel sections.

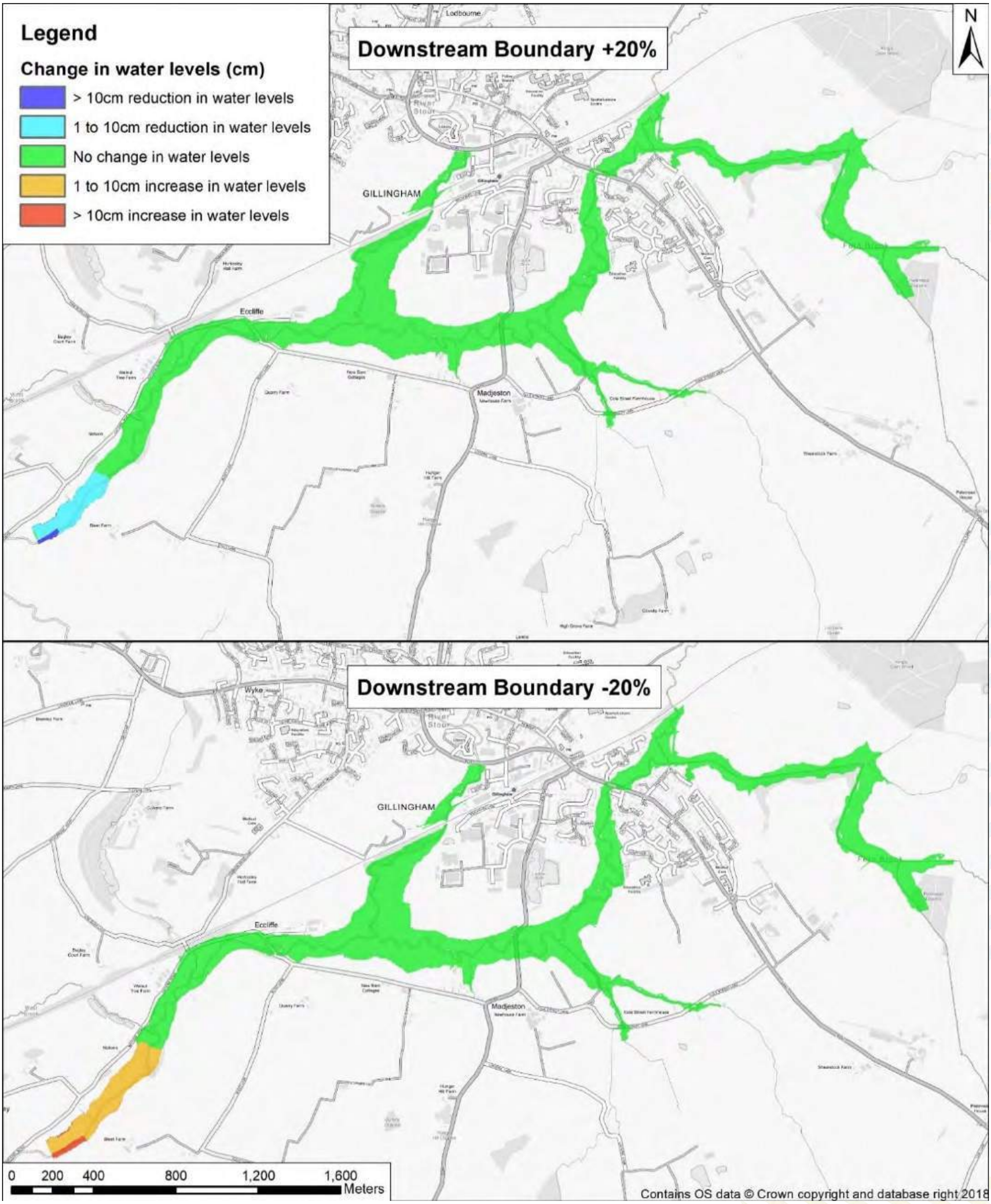


Figure 7-2: Sensitivity testing the downstream boundary - Changes in water levels

7.3 Storm duration

The hydrological analysis identified two critical storm durations for the modelled watercourses. For the River Lodden and River Stour, the Revitalised Flood Hydrograph (ReFH) indicated a critical storm duration of between 9 -10 hours. For the Fern Brook and Meadow watercourse which are smaller catchments, ReFH indicated a critical duration of between 5 – 6 hours. Based on the ranges provided in the hydrological assessment it was decided to test a 9 hour duration and a 5.5 hour duration using the 1% AEP event.

Figure 7-3 shows which critical duration produced the highest maximum water levels, this highlights that the 9 hour duration is critical for the majority of the model extent. There are two small areas which exhibit high water levels during the 5.5 hour duration on the upstream extent of the Fern Brook and Meadow watercourse. The difference in water levels is minimal between 1-2mm. Based on the storm duration testing it has been decided to adopt a single duration for the final design simulations using the 9 hour duration.

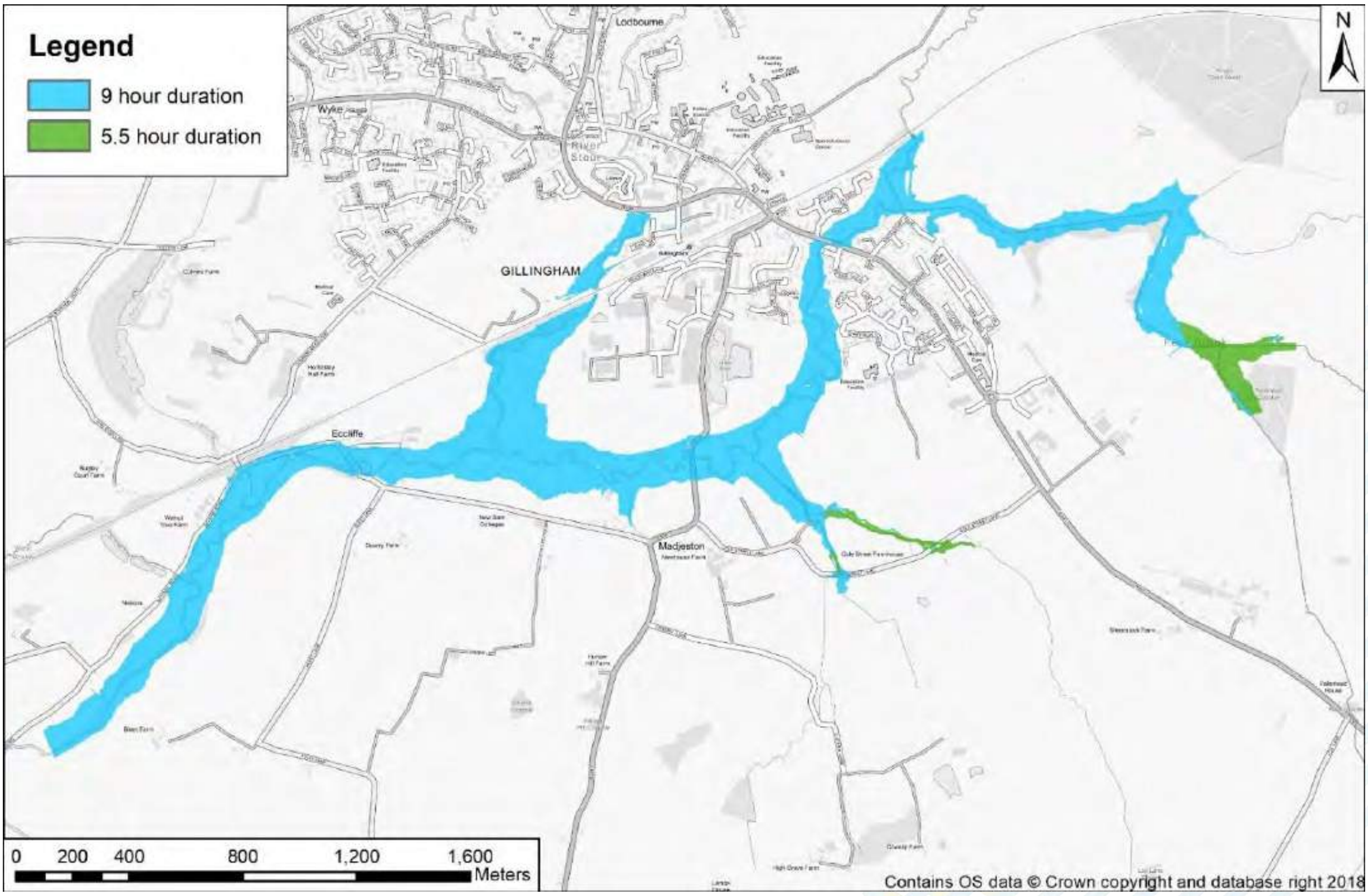


Figure 7-3: Storm duration maximum level comparison

8 Model performance

8.1 Model performance

The performance of the hydraulic model is summarised below:

- The model mass balance lies within the recommended acceptable range of +/- 1% for the 1% AEP event and smaller events. For the 0.1% AEP event this increases to a peak Mass Error of 1.15%. This can be attributed to a minor Mass Error issue with the 1D-2D linkage at the downstream extent of the model. Given the application of the downstream boundary does not influence the area of interest as indicated by the sensitivity analysis this was deemed acceptable.
- All simulations run stably with a 1D time-step of 0.5 seconds and a 2D time-step of 1 seconds. There are 60 -61 1D negative depths depending on the model event that occur at MEA2_0238.1, these occur at approximately 2.5 hours into the simulation, and then stop before the peak of the hydrograph is reached. This does not have any influence on the peak water levels and was therefore deemed an acceptable limitation. There are no 2D negative depths in any of the simulated events.
- Prior to all simulations the following check message occurred 10 times- "check 2099 - Ignored repeat application of boundary to 2D cell". Check 2099 identifies that two HX lines meet in the same cell and therefore the model receives two requests to change this cell to that of a boundary cell. All such locations have been checked to confirm that this will not cause an issue.
- Check 1152 occurred 24 times, "using centre cross-section and ignoring end cross-section(s)". This is because a centre cross-section has been digitised across a structure or open channel section to specify the dimensions and is used in precedence to the channel cross-sections snapped at either end. This is appropriate as it provides specific detail of the structure or section in question rather than inferring from upstream and downstream sections.
- Check 1284 occurs once, "CHECK 1284 - Connecting a 1D H boundary to 2D HX link." This check occurs at 1D downstream boundary and notes that the CN line it is connecting to the 2D HX link. This has been checked and confirmed to not cause an issue.
- Check 1037 occurred 10 times, "CHECK 1037 - Channel "STOU_1176M" interpolated from XS 00100 (52%) and XS 00101 (48%)". This occurs when the 1D network has been split but does not have its own cross-sectional csv therefore it is inferred by the upstream and downstream cross sections. This is the most appropriate way to model this situation. TUFLOW uses the upstream and downstream cross section to generate an appropriate cross section for that channel section. Some of the new channel sections have been surveyed with a larger than usual distance between them of up to 250m. This is the case on the Fern Brook as it is primarily a uniform man made drainage channel that that doesn't differ significantly throughout its reach. Therefore, sections at 50-100m spacing were not required. This was also the case on the downstream sections of the River Stour, this area of the model was not critical in terms of improving the existing flood map but was developed in improve the confidence in the downstream boundary application which has been achieved. Cross sections at 50m spacing were not required to deliver this improvement to the boundary conditions.
- Warning 1036 occurred 5 times, "WARNING 1036 - Maybe problems with interpolating n values for channel "Lodd_28U". Interpolating n using channel n

and downstream XSL n. Check n values." This warning occurs due to the different approaches used to specify the 1D open channel manning's n roughness. The 2006 ABD model uses a single roughness value for each 1d_nwke channel section whereas the newly incorporated sections uses material codes applied in the cross section csv files. This warning occurs at locations where the open channel sections try to interpolate between the two methods. The interpolated roughness values have been checked at each section and seem sensible. It was decided to adopt two separate approaches to represent 1D roughness coefficients as there wasn't sufficient data to update the existing model sections from the 2006 ABD study. The preferred approach to represent roughness is to use different values for bed and banks so the newly collected survey has followed this approach.

- WARNING 1100 occurred 4 times - "WARNING 1100 - Structure MEA2_0027C crest/invert (68.730) is below bed (68.750) of primary downstream channel MEA1_0337D." These have all been checked individually, these occur due to invert structure invert levels differing to that of the upstream or downstream cross section. A review of these structures indicate they are correct.

8.2 Limitations, assumptions, and uncertainty

Developing a hydraulic model requires the application of simplifications and generalisations. As such a number of assumptions are made when building the model. This can lead to model uncertainties and subsequent limitations in the results.

One of the main assumptions associated with the hydraulic model produced for this commission comes from the flow estimates applied to the model. The flows were calculated using the FEH Statistical method with Win-Fap V3 and data from Win-Fap FEH files V6. This improves the level of confidence we have in these flows as they were generated using industry recognised methods and the most recent available datasets. QMED was calculated from the catchment descriptor equation and adjusted using the nearby Colesbrook gauge. It was decided not to undertake sensitivity testing of the model inflows, even though this is a common sensitivity it often only shows the obvious, i.e. increase in flows results in increased flood extent and given the range of design events simulated there is sufficient evidence to gauge the model's performance to this input.

The topographical survey used for the existing ABD 2006 model, was collected as part of a 1999 survey commission. Unfortunately, the EA no longer have this survey on file and therefore were unable to provide it for this study. In order to assess the acceptability of this survey, check survey was collected by Maltby Land Surveys in 2018 and compared to the existing model sections. The results of this comparison are detailed in Section 2.2, this showed that the existing survey was still applicable with very little change to the bed levels and the overall opening area of sections. This analysis meant that additional survey for the full model extent was not required although additional survey did need to be collected for the Fern Brook and Meadow watercourse which was also collected by Maltby Land Surveys in 2018. The inclusion of new survey on these watercourses significantly improves the confidence in the modelled flood risk as the previous flood outlines seemed to have been derived using broadscale JFLOW modelling.

The LIDAR used to set the base topography in the 2D model domain is a source of model uncertainty. The bare earth DTM was filtered to remove the presence of buildings and vegetation. Following detailed checking of the DTM, some irregularities caused by this filtering process were modified by altering the topography using a number of Z-Shapes and Z-lines within the 2D TUFLOW domain. LIDAR has been used to specify the bank levels for the HX 1D-2D linkage on the newly incorporated sections of the model on the Fern Brook, Meadow watercourse and downstream end of the River Stour. This approach is preferred in this instance as it ensures that the bank levels will not be incorrectly raised. The model reaches that have been retained from the 2006 ABD study have their bank elevations based on Z-lines, as this was originally signed off from the EA it was decided not to review and updated the HX schematisation. The main issue with using the LIDAR to represent bank elevations is that vegetation, hedges and trees can impact the LIDAR data collection which are often found on top of banks. Levels from the 2D domain have been compared with those of the cross-section survey to ensure the modelled bank levels are appropriate.

The model shows a small amount of flooding in the 2D domain after the first model output interval. This is attributed to the 1D Initial Water Level (IWL) and attempts were made to remove this from the final model by amending the IWL. Unfortunately, this flooding does remain in some locations but has no bearing on the peak water levels in any of the simulated events and has therefore been deemed an acceptable limitation.

As mentioned in Section 8.1, there are 60 -61 1D negative depths depending on the model event in the model located on the Meadow watercourse at the 1D node MEA2_0238.1. These occur in all of the simulated design events but before the peak of the simulation, they cause a small flick to occur in the 1D results but have no bearing on the peak water levels. Attempts were made to address these including re-schematising the 1D domain, amendments to the 1D-2D linkage and applying a HX energy loss value of 0.5. These improved the results but negative depths remain in the final model. As they do not influence the peak model results it was deemed they are an acceptable limitation.

The model parametrisation is another assumption associated with the model development. The Manning’s n roughness coefficients have been sensitivity tested to determine their influence on the modelled flood risk as detailed in Section 7.1. This indicated that the modelled flood outlines are not particularly sensitivity to the Manning’s parameters.

Structure coefficients are another aspect of model parameterisation that is an assumption, the bridge, culvert and weir coefficients have been determined as based on the TUFLOW manual. No sensitivity analysis was undertaken on these coefficients as without extensive hydrometric data there is no justification to alter these coefficients from the recommended defaults.

Overall, the 1D model performance is very good, with only some small fluctuations in the 1D flow results on the Meadow watercourse at the sections MEA1_0272 and MEA1_0273. This is a complex area of the model that includes a small circular culvert with a dilapidated sluice structure located immediately upstream. The 1D flow plot is not a typically smooth hydrograph, but the fluctuations in flow do not have an adverse influence on 1D water levels which remain smooth and stable and therefore this is deemed to be acceptable.

One of the main limitations with the updated model is the fact the model outputs could not be calibrated / validated. There are no gauges located with the new model extent so inputting observed hydrographs has not been possible. The EA historical flood map does not show any flooding of the area so it has not been able to validate the model extents or flood frequency. The model results have been compared to the previous model which for the most part shows very similar flood outlines which indicates at least that the model outputs are sensible. The sensitivity analysis has been undertaken on the Manning’s n roughness coefficients, the downstream boundary and the critical storm duration. These model parameters were identified as the most critical in terms of understanding their influence on the model outputs and have enables improved confidence in the sensibility of the outputs. The sensitively testing demonstrates that the model results are generally insensitive to the model parameterisation.

9 Model runs

9.1 General procedures for model runs

- Prior to running the hydraulic model, the most straightforward approach is to save the “TUFLOW” folder contained within the “Model” folder, into ‘C:\Gill which will need to be created on the user computer C drive.
- All folders will then need to be uncompressed, with care taken to preserve the original folder structure.
- By setting up the folder structure this way, when the model is run both the results and the check files will be saved in their relevant TUFLOW folders on the C drive as both the .tcf and .ecf reference these locations.

9.2 Explanation of file types

.tgc = TUFLOW Geometry Control file
.tcf = TUFLOW Control File
.tbc = TUFLOW Boundary Condition file
.trf = TUFLOW Restart File
.ecf = ESTRY Control File
.tef = TUFLOW Event File
.tmf = TUFLOW Material File

9.3 Design events

Run Reference:	Gillingham_ABD_~s1~_~e1~_074 (s1 = DEF/UND scenario) (e1 = design event)	
Purpose of Runs:	To model a range of fluvial flood events using the defended or undefended model scenario	
ESTRY/TUFLOW file and Version: 2018_03_AD_iSP_w64	File names:	
	Gillingham_ABD_~s1~_~e1~_074.tcf	Gill_072.tef
	Gillingham_ABD_~s1~_~e1~_074.tcf	Gill_065.tmf
	Gillingham_ABD_073.tgc	
	Gillingham_ABD_073.tbc	
Model timesteps:	A 1 second timestep has been applied within the 2D model and a 0.5 second timestep has been applied within the 1D model.	
Run Time:	Model event duration: 21 hours for fluvial events, simulation time 8 hours.	
Return period(s)	50% AEP, 10% AEP, 5% AEP, 2% AEP, 1.33% AEP, 1% AEP, 0.1% AEP, and 1% AEP + 40% climate change factor, and 1% AEP + 85% climate change factor.	
Run Settings:	11 fluvial inflows and a downstream boundary. All parameters were left as default unless stated.	
Comments on results:	1% AEP MB Error 2017 = -0.45% 0.1% AEP MB Error - 2017 = 1.15%	

10 Conclusions and Deliverables

10.1 Model updates

- The EA 2006 ABD model has been reviewed and amended to focus specifically on the area of interest located to the south of Gillingham town centre.
- The model has been updated to include the latest LIDAR and OS MasterMap and extended to include the Fern Brook and Meadow watercourses as well as the River Stour downstream extent to improve the outflow boundary conditions. The model has been simulated using the most recent version of TUFLOW.
- A new hydrological assessment has been undertaken to derive to inflow hygrographs for the model. This has made use of recent hydrometric data and latest FEH methods to produce a more reliable assessment of the catchment hydrology.

10.2 Flood zones

The updated model outputs for the 1% AEP event and the 0.1% AEP event have been compared to the existing EA flood zones as shown in Figure 10-1 and Figure 10-2 respectively. Even with the updated model and more detailed representation of the Fern Brook and Meadow watercourse along with the new inflow hydrology the flood outlines remain very similar.

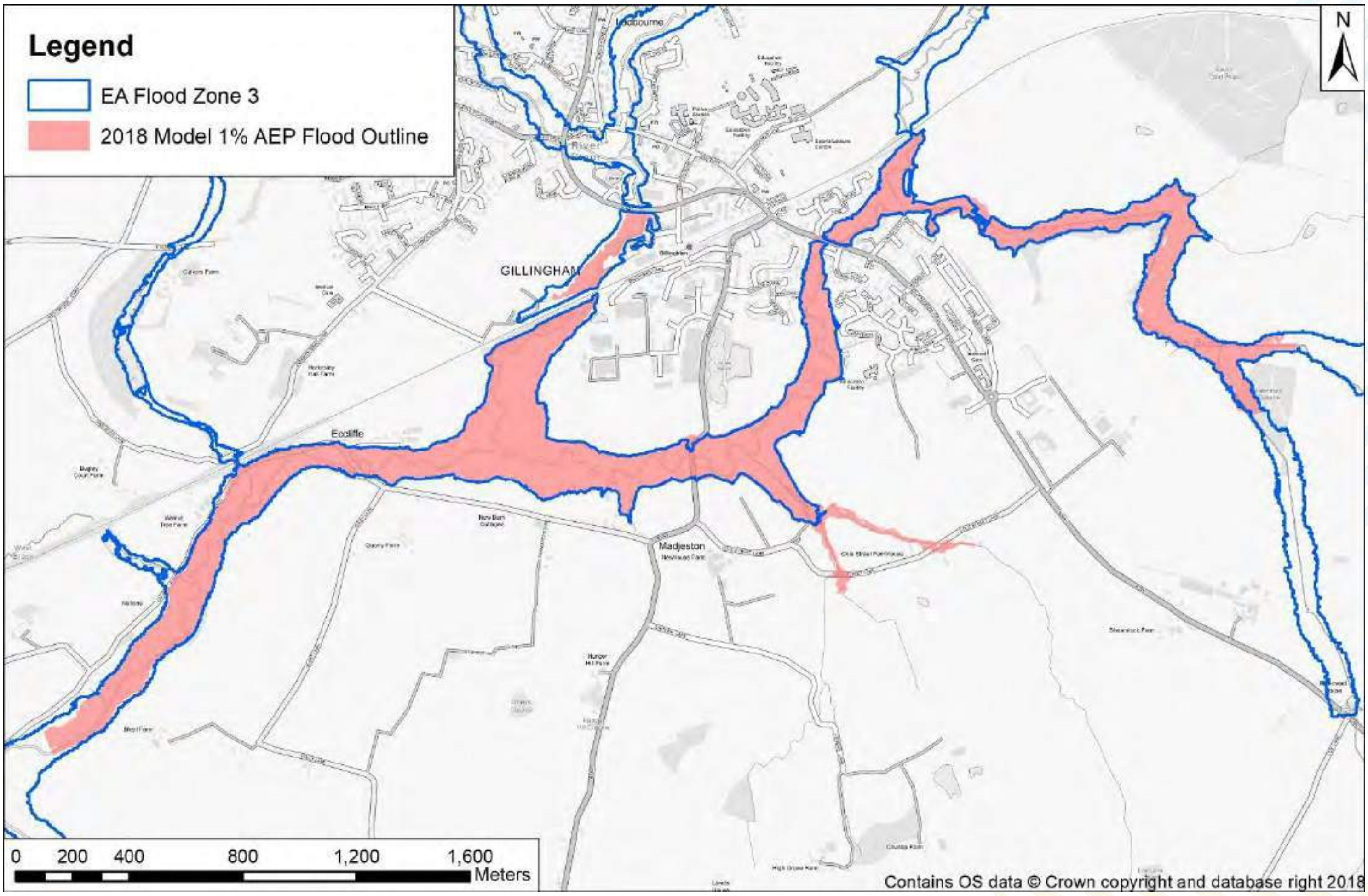
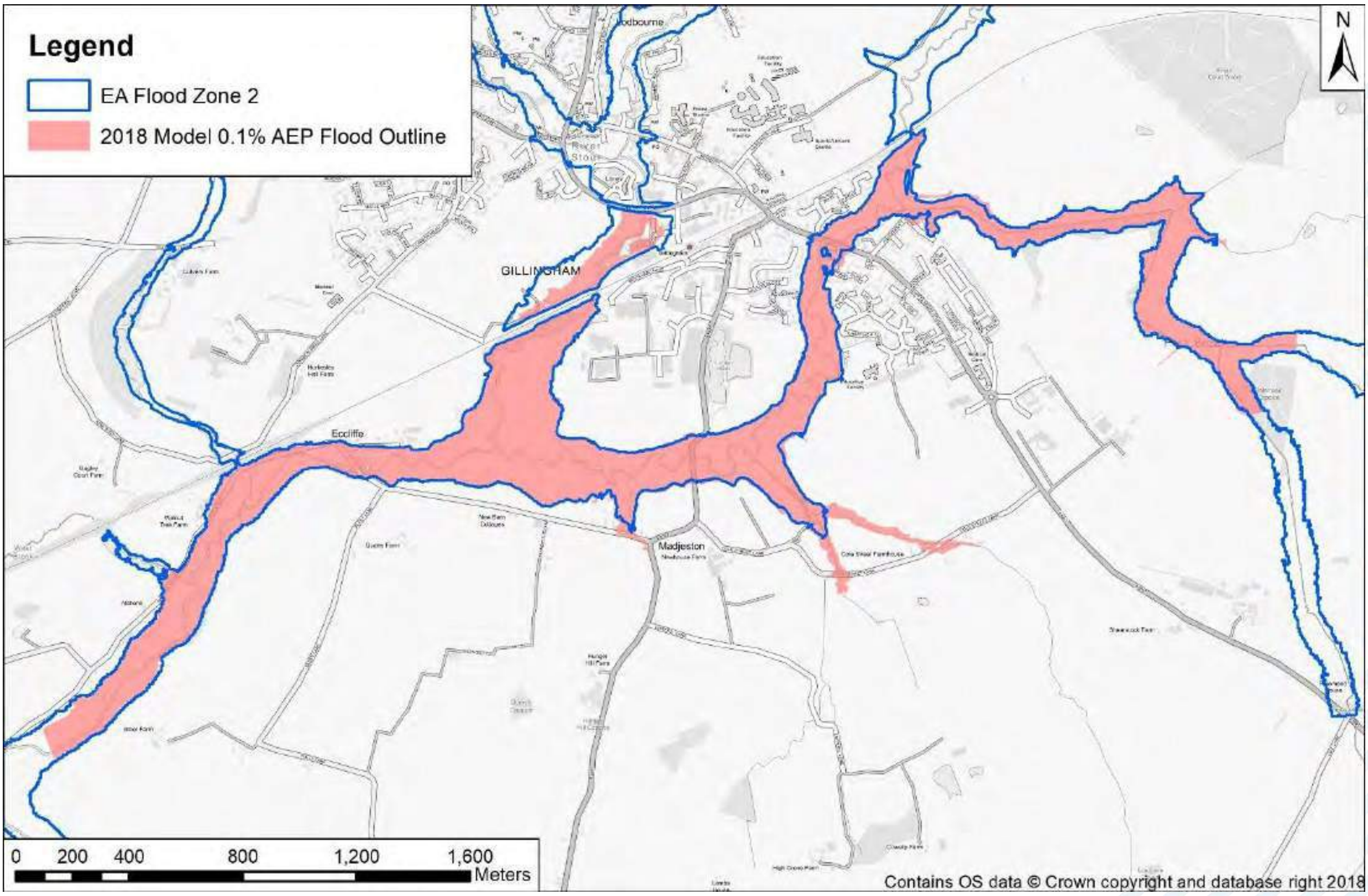


Figure 10-1: Flood zone 3 comparison



Contains OS data © Crown copyright and database right 2018

Figure 10-2: Flood zone 2 comparison

10.3 Recommendations

- The model includes data, specifically open channel sections of the River Stour and River Lodden that are based on survey from 1999. The analysis undertaken in this study through the collection of check survey showed that the reuse of this data was appropriate. If the model is to be used in the future then this may need to be revisited.
- The 2006 ABD study highlighted the difficulties in calibrating the model due to lack of hydrometric data and flood history information. To improve the confidence in the model outputs additional data would need to be collected from future flood events.

10.4 Deliverables

In order for the EA to effectively review the updated Gillingham model and hydrology, a range of deliverables have been provided that are required by the EA if the updated model is used to undertake Flood Map Challenge. The following points outline the deliverables and their associated appendix.

- Topographical Survey (Appendix A)
    - 2018 Check Survey
    - 2018 Additional Survey
- FEH Calculation Record (Appendix B)

- Packaged TUFLOW model (Appendix C)
- Processed Results (Appendix D) – includes depth, level, velocity, and hazard grids for all the simulated design events including flood outlines in SHP and MapInfo format with and without dry islands removed.
- Areas Benefitting from Defences (ABD) polygon in SHP and MapInfo format. The ABD polygon is typically provided for the 1% AEP event however the water level doesn't reach the Brickfields flood defence during this event. The defence is enacted during the 0.1% AEP event so the ABD polygon has been provided for that event. (Appendix E).

Appendices

A 2018 Topographical survey

B FEH Calculation Record

C Hydraulic Model

D Processed Results

E Areas Benefiting from Defences (ABD)

Offices at:

Coleshill
Doncaster
Dublin
Edinburgh
Exeter
Glasgow
Haywards Heath
Isle of Man
Limerick
Newcastle upon Tyne
Newport
Peterborough
Saltaire
Skipton
Tadcaster
Thirsk
Wallingford
Warrington

Registered Office
South Barn
Broughton Hall
SKIPTON
North Yorkshire
BD23 3AE
United Kingdom

+44(0)1756 799919
info@JBA - consulting.com
www.JBA - consulting.com
Follow us:

Jeremy Benn Associates Limited

Registered in England 3246693

JBA - Group Ltd is certified to:
ISO 9001:2015
ISO 14001:2015
OHSAS 18001:2007