



**HYDRAULIC MODELLING STUDY (INCORPORATING PROPOSED CULVERT & CHANNEL PROFILE AMENDMENTS)
FOR
PROPOSED RESIDENTIAL DEVELOPMENT, FORMER TETROSYL SITE
WALMERSLEY OLD ROAD, BURY, GREATER MANCHESTER
ON BEHALF OF
BARRATT HOMES MANCHESTER**

APRIL 2021

[ISSUE 2]

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COLE EASDON CONSULTANTS (CEC)

DOCUMENT ISSUE RECORD

Client: Barratt Homes Manchester

Project: Proposed Residential Development, Former Tetrosyl Site, Walmersley Old Road, Bury, Greater Manchester

Job Number: 6155

Document Title: Hydraulic Modelling Study (Incorporating Proposed Culvert and Channel Profile Amendments)

Issuing Office: Bristol

Issue No.	1	2		
Date	February 2021	April 2021		
Description / Status	EA Submission	EA Submission		
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1.0 INTRODUCTION

- 1.1 This *Hydraulic Modelling Study and Flood Risk Assessment* has been prepared by Cole Easdon Consultants Limited (CEC) on behalf of Barratt Homes Manchester. The Study builds on the previously approved *Hydraulic Modelling Study* that accompanied approved planning application ref: 63533 and which describes hydraulic modelling work carried out as part of a flood risk study in respect of fluvial flood risk associated with land at the former Tetrosyl Site, Walmersley Old Road, Bury, Greater Manchester.
- 1.2 The approved Development Proposal comprises the redevelopment of the former Tetrosyl factory site including the removal of Mill Reservoir and existing buildings and hardstanding and the erection of 268 dwellings with associated access, parking and landscaping. Refer to CEC Figure 6155/500/Figure 1 [*Site Location Plan*] in Appendix 1.
- 1.3 The approved Modelling Study determined flood levels and the extent of fluvial flooding at the existing site associated with flooding of a tributary of Pigs Lee Brook (PLB). The approved study also quantified flood levels and extent associated with a residential development proposal comprising infilling of the existing Mill Reservoir, together with associated deculverting works of the PLB Tributary through the development site. The Study concluded that with appropriate proposed mitigation measures as set out in the Report that no flood risk was posed to the development site, and that there would be no increase in flood risk elsewhere for all events up to and including the 1:100 year+ 35% climate change event.
- 1.4 This current Report details post planning amendments to the approved hydraulic model in response to the subsequent receipt of additional drainage survey information, and detailed watercourse channel design requirements.
- 1.5 In January 2020, the Developer (Barratt Homes) undertook a drainage survey of the existing culvert from the downstream end of the Mill Reservoir to existing Manhole 1 (MH01). The surveyed route was found to deviate significantly from the route as determined by the Environment Agency. As such, it was considered necessary to divert the downstream extent of the existing culvert as the surveyed route would have placed the culvert directly below the location of proposed dwellings.

- 1.6 Furthermore, post planning geomorphological modelling undertaken by JBA Consulting in relation to the proposed open channel (deculverted watercourse) has resulted in design changes to the proposed channel cross section and longsection profiles.
- 1.7 Both the proposed culvert diversion and open channel design amendments have necessitated the need for additional hydraulic modelling to ensure the proposed changes will not result in an increase in flood risk.
- 1.8 For clarity, the approved hydraulic model is referred to as the ‘Approved Model’ in this Report. Post planning hydraulic modelling to account for modifications to the Approved Model as described in this Report is referred to as the ‘Modified Model’ within this Report.

Scope of Study

- 1.9 In Section 2.0, we describe the proposed design changes and model amendments in detail. Section 3.0 provides a recap of hydraulic modelling parameters used in the Approved Model. Hydraulic modelling results are presented in Section 4.0. Flood risk is discussed in Section 5.0. Finally, conclusions are presented in Section 6.0.
- 1.10 The following abbreviations are used in this Report:
- AOD - Above Ordnance Datum;
 - EA - Environment Agency;
 - FZM - Flood Zone Map prepared by the EA;
 - PLB - Pigs Lee Brook; and
 - ReFH - Revitalised flood hydrograph.

2.0 PROPOSED DESIGN CHANGES AND HYDRAULIC MODEL AMENDMENTS

Proposed Culvert Diversion

- 2.1 As described within the approved Hydraulic Modelling Study, most of the existing culvert within the development site will be abandoned and replaced with open channel along the eastern site boundary. However, it is the intention to retain the existing culvert at the southern end of the site and to connect into this culvert with a new piped culvert from the downstream end of the proposed open channel. At the time of the planning application submission the exact route of the existing culvert was not known and flood risk hydraulic modelling and preliminary design of the open channel route and culvert connection into the existing culvert (shown in Figure 2.2 below) was based upon the EA's mapped culvert route shown below in Figure 2.1.

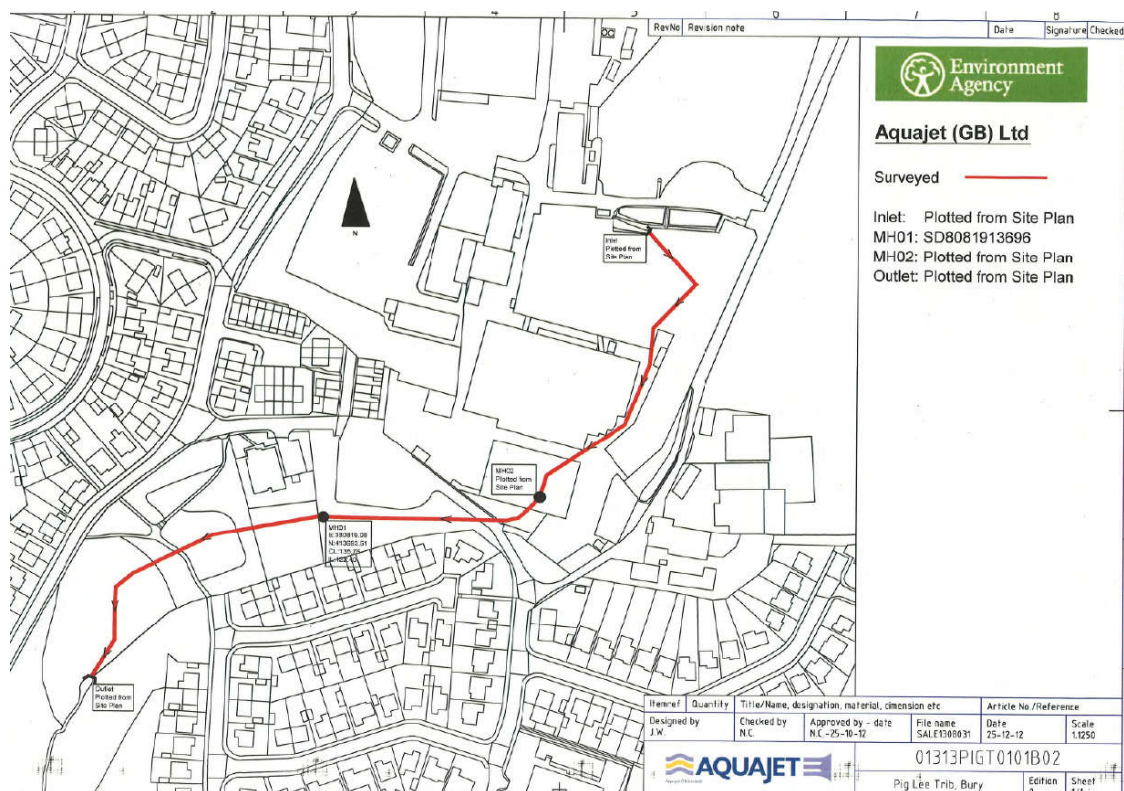


Figure 2.1: Route of Existing Culvert (Environment Agency)

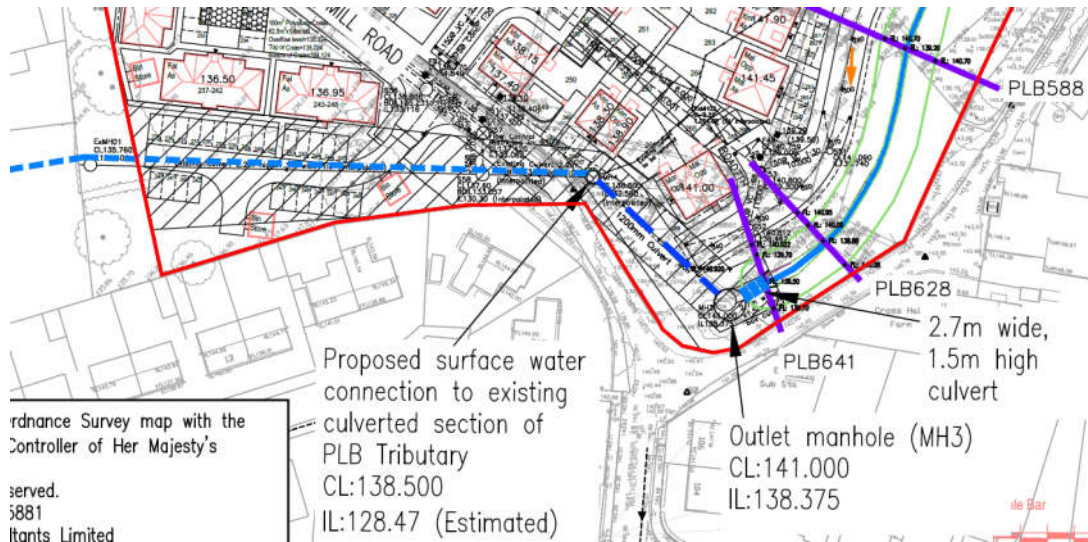


Figure 2.2: Assumed route of existing culvert (excerpt from approved CEC Plan 6155/502(E))

- 2.2 The applicant (Barratt Homes) subsequently undertook a culvert survey in January 2020. The surveyed route is shown in Figure 2.3 below. Given the siting of the existing culvert below proposed dwellings, it was considered unviable to retain this section of existing culvert. Thus the design has been amended to replace this length of culvert with new 1200/1500mm diameter pipework connecting into the extent of existing culvert to be retained at proposed manhole MH6 shown in Figure 2.3 below. Due to the c.10.5m level difference between the incoming 1500mm pipe and existing culvert outlet manhole MH6 has been designed as a cascading drop structure. The drop structure has been modelled hydraulically and designed by Hydrotec. Refer to Appendix 3 for the drop structure hydraulic modelling report by Hydrotec.
- 2.3 The Approved Model has now been modified to take account of the abovementioned design changes using design information shown in Figure 2.3 and formalised on CEC Plan 6155/514(B) [*Proposed Culvert Diversion Route*] in Appendix 5.

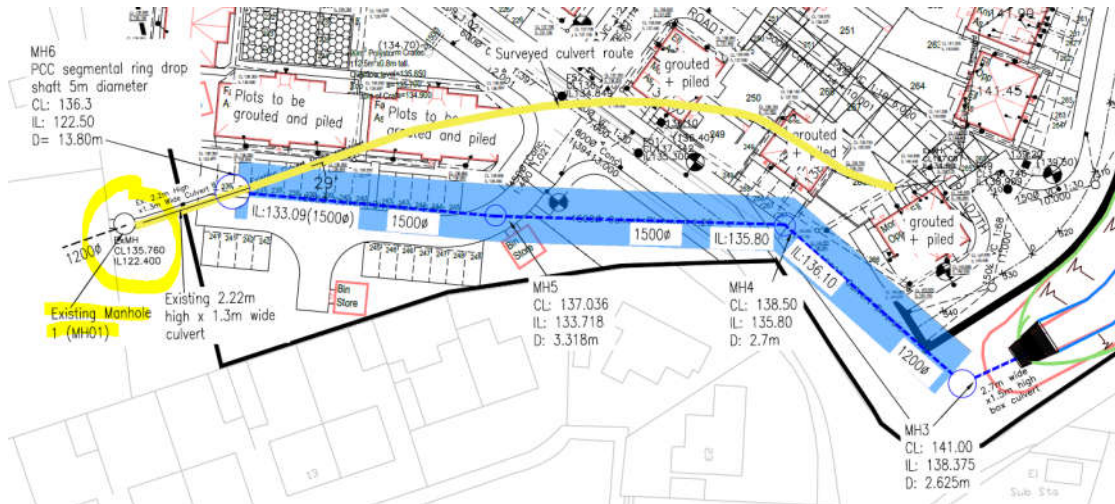


Figure 2.3: Surveyed Culvert Route & Route of Proposed Culvert Diversion (Existing Manhole 1 location circled and highlighted)

Design Changes to Proposed Watercourse

- 2.4 In July 2020, JBA Consulting was instructed by the Applicant to undertake geomorphological modelling of the reach of proposed open watercourse between chainages PLB352 and PLB641, a length of some 289m. Refer to Appendix 3 for the Geomorphology report. In order to accommodate the proposed bar, rapids and stepped pool features indicative of local geomorphology it was necessary to modify the channel cross section profile and gradient of the proposed open channel between chainages PLB352 and PLB641. Figure 2.4 below shows the original longsection profile that was used in the Approved Model and the modified longsection profile. Original cross section profiles used in the Approved Model and modified cross sections are shown on CEC Plans 6155/507 Revisions A & D [*Hydraulic Modelling Study - Proposed Cross-sections (2 No. Sheets)*] in Appendix 5.
- 2.5 The Approved Model has now been modified to take account of the abovementioned design changes.
- 2.6 Hydraulic modelling results arising from all abovementioned modifications are presented and discussed in Section 3.0 of this Report.

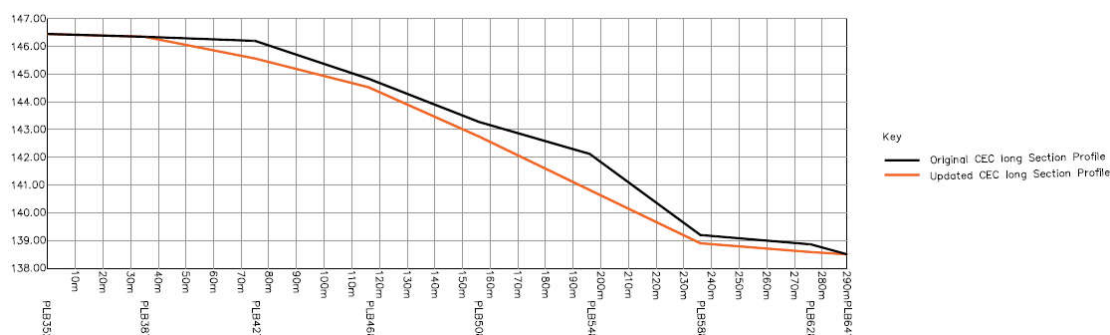


Figure 2.4: Comparison of Original and Modified Proposed Watercourse Longsection Profiles

Model Hydrology

- 2.7 There are no changes to the hydrological input used in the approved hydraulic model.
- 2.8 The Modified Model uses the same post development hydrographs that have been used in the approved post development model. This comprises the data set 'MillRes_Post' with hydrographs based on a 5.25 hour storm duration. Refer to Appendix 2.

Hydraulic Model Parameters

- 2.9 There are no changes to any of the parameters used in the approved hydraulic model.

3.0 RECAP OF HYDRAULIC MODELLING PARAMETERS

Refer to CEC Plans 6155/502(G) [*Proposed Site Layout*], 6155/503(B) [*Existing Hydraulic Model Extent (3 No. Sheets)*] and 6155/505 [*Hydraulic Modelling Study - Mill Reservoir - Existing Cross-sections*], in Appendix 5.

- 3.1 Hydraulic modelling has been carried out using Flood Modeller Software Version 4.4.6803.28334 (2019). This is a hydraulic modelling package capable of handling one-dimensional, two-dimensional and 1D-2D linked models; the latter has been used in the construction of the PLB Tributary model. For the 1D element of the model the software can represent river systems as a series of nodes and links, with sections of the river channel applied to the model as nodes and the chainages between each section of links between the node. Flow through the channel is modelled in terms of changes in water levels (or 'stage') at each cross section.

Post Development Site: 1D Hydraulic Model Extent along PLB Tributary

Refer to CEC Plan 6155/512(E) [*Schematic Plan: Post Development Model*] in Appendix 5.

- 3.2 Proposed development will comprise the infilling of Mill Reservoir, the demolition of existing buildings, the deculverting of Culvert 3 where it passes through the site (some 460m length) and ground reprofiling to facilitate development. Refer to CEC Plans 6155/502(G) [*Proposed Site Layout*] and 6155/507(D) [*Hydraulic Modelling Study - Proposed Cross-sections (2 No. Sheets)*] in Appendix 5.
- 3.3 The approved hydraulic model for the proposed situation has been utilised with the following amendments:
- i) The proposed culvert between new manholes MH4 and MH6 has been modelled as a 1500mm diameter pipe connecting into the extent of existing culvert to be retained at proposed manhole MH6.
 - ii) Proposed manhole MH6 has been modelled with a 10.5m vertical drop (modelled as a weir drop) to simulate the level difference between the incoming 1500mm pipe and existing culvert outlet.
 - iii) Modified channel cross section profile and gradient of the proposed open channel between chainages PLB352 and PLB641. Refer to CEC Plans 6155/507 Revisions A & D [*Hydraulic Modelling Study - Proposed Cross-sections (2 No. Sheets)*] in Appendix 5.

Flood Storage Area

- 3.4 The proposed FSA bed level is unchanged at 146.70mAOD. Top of bank level is unchanged at a minimum level of 149.7mAOD.
- 3.5 The FSA will essentially be a dry storage area with low flow routed through a channel and high flow retained in the FSA. The FSA bed will be vegetated with wetland grass.

Flow Control

- 3.6 An orifice flow control with a 1.21sqm cross sectional area has been inserted at the downstream end of the FSA between Sections PLB337 and PLB352. The invert level of the orifice is unchanged at a level of 146.450mAOD. The top of the wall will be set slightly lower than top of bank at 149.35mAOD (unchanged) to facilitate weir flow during design exceedance events. The weir has been modelled as a 15m wide spill unit set at 149.35mAOD (unchanged) and spills back into the downstream proposed open channel at PLB352.

Open Channel

- 3.7 The channel bed and banks will be vegetated with an appropriate grass/shrub mix. Refer to Drawing No. c-1339-01(B) [*Outline Landscape Masterplan*] (by PDP Associates) in Appendix 4.
- 3.8 The open channel will discharge into a 2.70m wide x 1.5m high culvert at the downstream end of the site (unchanged), before discharging into a 1200mm culvert at manhole MH3. Downstream of MH3, flow will be conveyed via the proposed 1200mm diameter culvert to MH4 where the culvert will be upsized to 1500mm and will then convey flow to the proposed cascading Drop Structure at MH6 which to be located on the existing culvert.
- 3.9 The modelled open channel sections and FSA have been sized so as to adequately accommodate flow arising from the 1:100 year + 35% flood event without increasing flood risk within the site or downstream of the site.

1D Hydraulic Roughness

- 3.10 There are no changes to channel roughness used in the approved hydraulic model. Refer to Table 3.1 below. New culverts downstream of the proposed open channel have been assigned a relevant roughness coefficient.

Table 3.1: Proposed Site: Modelled 1D Roughness Coefficients

Location	Proposed Vegetation/Material	Modelled Mannings 'n' Coefficient
Flood Storage Area: Bed	Wetland Grass	0.05
Flood Storage Area: Banks	Shrubs	0.086
Open channel: Bed	Wetland Grass	0.05
Open channel: Banks	Grass/Shrubs	0.05/0.086
New culverts	Concrete	0.015
2D Floodplain	Paved Areas	0.033
	Trees	0.12

Post Development Site: 2D Hydraulic Model Extent along PLB Tributary

- 3.11 All 2D parameters used within the Approved Model have been retained. For completeness these are reiterated below.
- 3.12 The 2D domain used within the Approved Model has been retained. Refer to Figure 3.1 below.
- 3.13 The following parameters and links used within the Approved Model have been retained:
- The 2m x 2m 2D grid cell size used within the Approved Model has been retained.
 - The 1D-2D link line used at the downstream end of the existing model where the culverted watercourse returns to open channel has been retained to represent the potential return point for any out of bank flooding that may occur upstream.
 - The 1D-2D link line between the right banks of PLB588 - PLB641 to allow flood flow spill into the 2D floodplain has been retained.
 - The flow link line to allow flood flow into the 2D domain when water overtops the cover level of MH2 immediately to the west of the southern site boundary has been retained.
- 3.14 As for the Approved Model, the 2D domain does not consider the floodplain upstream of the M66 culvert, as it is unlikely to impact the site due to the presence of the M66 embankment. As such the floodplain upstream of the M66 has been modelled as 1D extended sections, as per the Approved Model.



Figure 3.1: Post Development Site Model: Modelled 2D Floodplain Domain

2D Hydraulic Roughness

- 3.15 2D roughness values used within the Approved Model have been retained.

1D Boundary Conditions

Upstream Boundary Condition

- 3.16 Flow hydrographs obtained from JBA's hydrological analysis and used within the Approved Model have been retained as upstream boundary input flows in the modified hydraulic model.
- 3.17 A hydrograph for piped flow based on the proposed developed restricted discharge rate of 93l/s used within the Approved Model has been retained and input into the modified hydraulic model to represent flow from the post development to the proposed onsite culvert. Refer to Figure 3.2 below. This hydrograph has been input to the network at the location of the proposed manhole MH3. The peak of the hydrograph ($0.0093\text{m}^3/\text{s}$) coincides with the timing of the peak flow through the PLB Tributary at the Mill Reservoir outlet (Time to peak: 5.25 hours).

Post Development Surface Water Runoff Hydrograph (1:100 year +40% event)

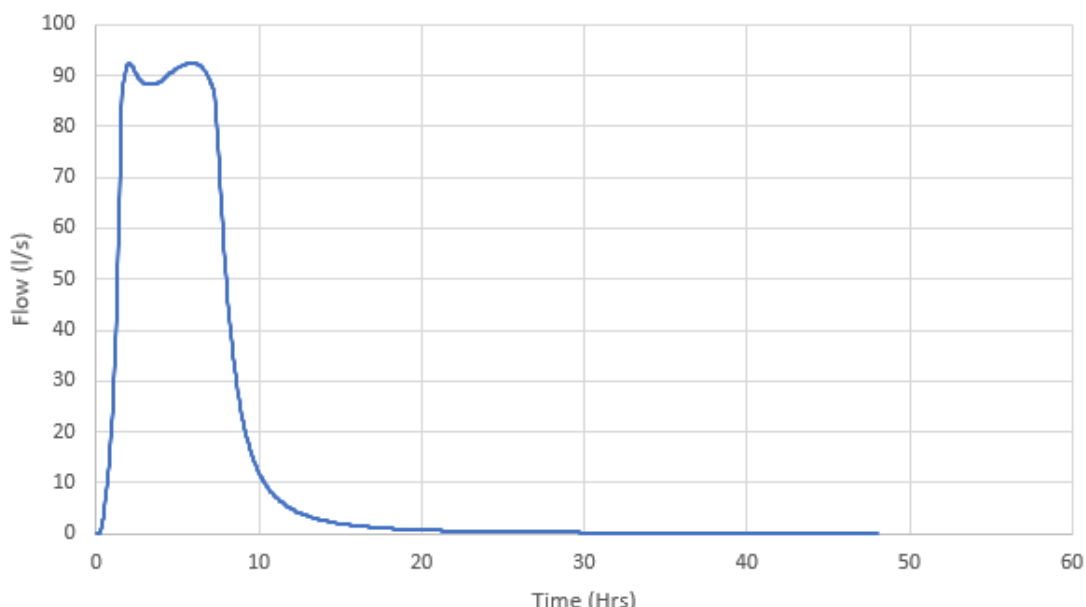


Figure 3.2: Post Development Hydrograph Input into Model at MH3

Downstream Boundary Condition

- 3.18 The boundary condition used in the Approved Model has been retained.

Modelled Scenarios - Existing Situation

- 3.19 Results from the Approved Model for the 1D-2D linked model representing the existing site and potential flood risk from Mill Reservoir have been retained, remain valid and are used for comparison for the following events:
- i) 1:20 year return period;
 - ii) 1:100 year return period;
 - iii) 1:100 year + 35% climate change return period (for 2.75 hour and 5.25 hour storm durations);
 - iv) 1:100 year + 70% climate change return period; and
 - v) 1:1000 year return period.

Modelled Scenarios - Post Development Scenario

- 3.20 The 1D-2D linked Modified Model representing the post development site has been run for the following events:
- i) 1:20 year return period;
 - ii) 1:100 year return period;



- iii) 1:100 year + 35% climate change return period;
- iv) 1:100 year + 70% climate change return period;
- v) 50% blockage of culvert trash screen downstream of new proposed open channel;
- vi) Variation of Mannings 'n' by +20% and -20% (1:100year + 35% return period event); and
- vii) 1:1000 year return period.

4.0 HYDRAULIC MODEL RESULTS

Refer to CEC Plans 6155/502(G) [*Proposed Site Layout*], 6155/503(B) [*Existing Hydraulic Model Extent (3 No. Sheets)*] and 6155/505 [*Hydraulic Modelling Study - Mill Reservoir - Existing Cross-sections*] in Appendix 5 and the modelled Longsections - Maximum Stage (Existing & Post Development) in Appendix 6.

Modelled Scenarios - Existing Situation

4.1 Results from the Approved Model for the 1D-2D linked model representing the existing site and potential flood risk from Mill Reservoir have been retained, remain valid and are used for comparison for the following events:

- i) 1:20 year return period;
- ii) 1:100 year return period;
- iii) 1:100 year + 35% climate change return period (for 2.75 hour and 5.25 hour storm durations);
- iv) 1:100 year + 70% climate change return period; and
- v) 1:1000 year return period.

Modelled Scenarios - Existing Situation

4.2 Peak water levels and flowrates at critical modelled locations taken from the Approved Model results are summarised in Tables 4.1 and 4.2 below. Modelled floodplain extents are presented in Figures 4.1 to 4.4 below. Please refer to the approved *Hydraulic Model Study* for a complete description of results.

Table 4.1: Existing Model - Maximum Stage (mAOD)

Model Node Reference	Return Period Event						Comment
	1:20 yr*	1:100 yr*	1:100 yr + 35% CC 2.75 hr storm	1:100 yr + 35% CC 5.25 hr storm	1:100 yr + 70% CC*	1:1000 yr storm*	
PLB053 U/S of M66 culvert	155.629	156.281	156.804	156.899	157.455	157.578	
PLB369 Mill Reservoir	149.305	149.494	149.539	149.564	149.624	149.641	2D flooding occurs for all events except 1:20 yr event
PLB1014 End of model	116.159	116.183	116.202	116.214	116.247	116.256	

*Scenarios run using 5.25 hr storm duration

Table 4.2: Existing Model - Peak Discharge Rate (m³/s)*

Model Node Reference	Return Period Event						Comment
	1:20 yr*	1:100 yr*	1:100 yr + 35% CC 2.75 hr storm	1:100 yr + 35% CC 5.25 hr storm	1:100 yr + 70% CC*	1:1000 yr storm*	
Start of Model	2.80	4.20	5.67	5.67	7.14	7.5	
PLB053 U/S of M66 culvert	2.66	3.84	4.98	5.15	6.50	6.84	
PLB369 Mill Reservoir	2.15	3.56	4.61	5.02	6.18	7.152	2D flooding occurs for all events except 1:20 yr event
PLB1014 End of model	1.96	3.91	4.67	5.15	6.68	7.00	A proportion of 2D floodflow does not re-enter the 1D model

*Scenarios run using 5.25 hr storm duration

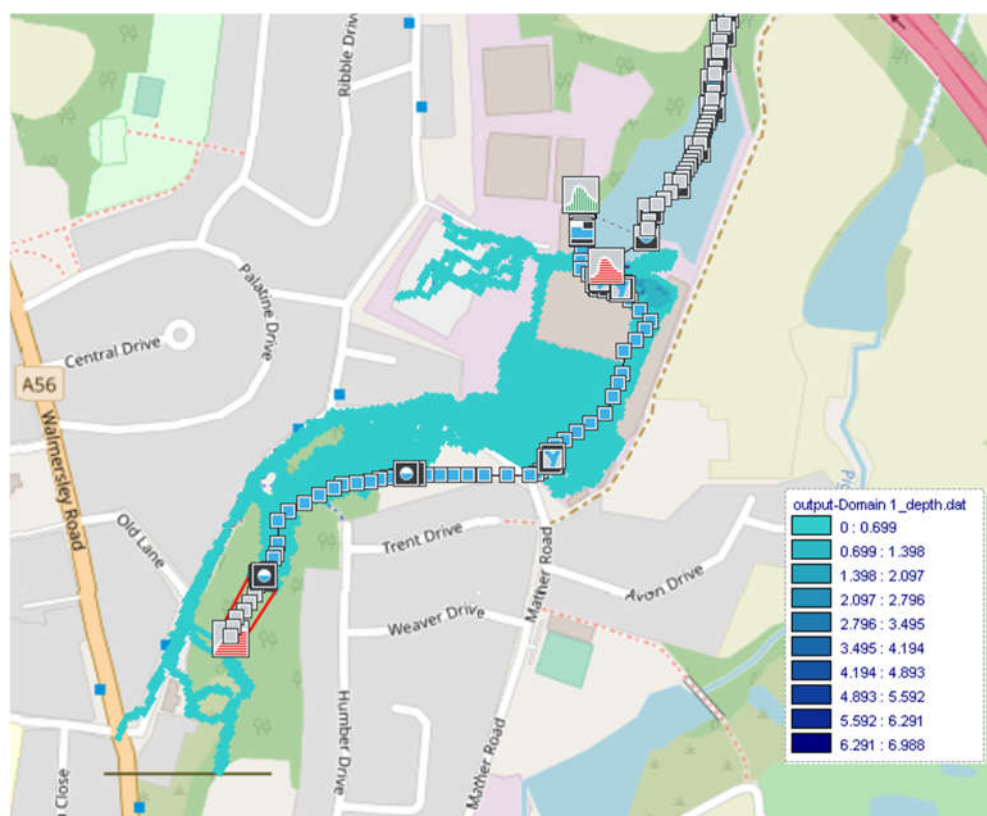


Figure 4.1: Extent of flooding from Mill Reservoir for the 1:100 year event (5.25 hour storm duration)

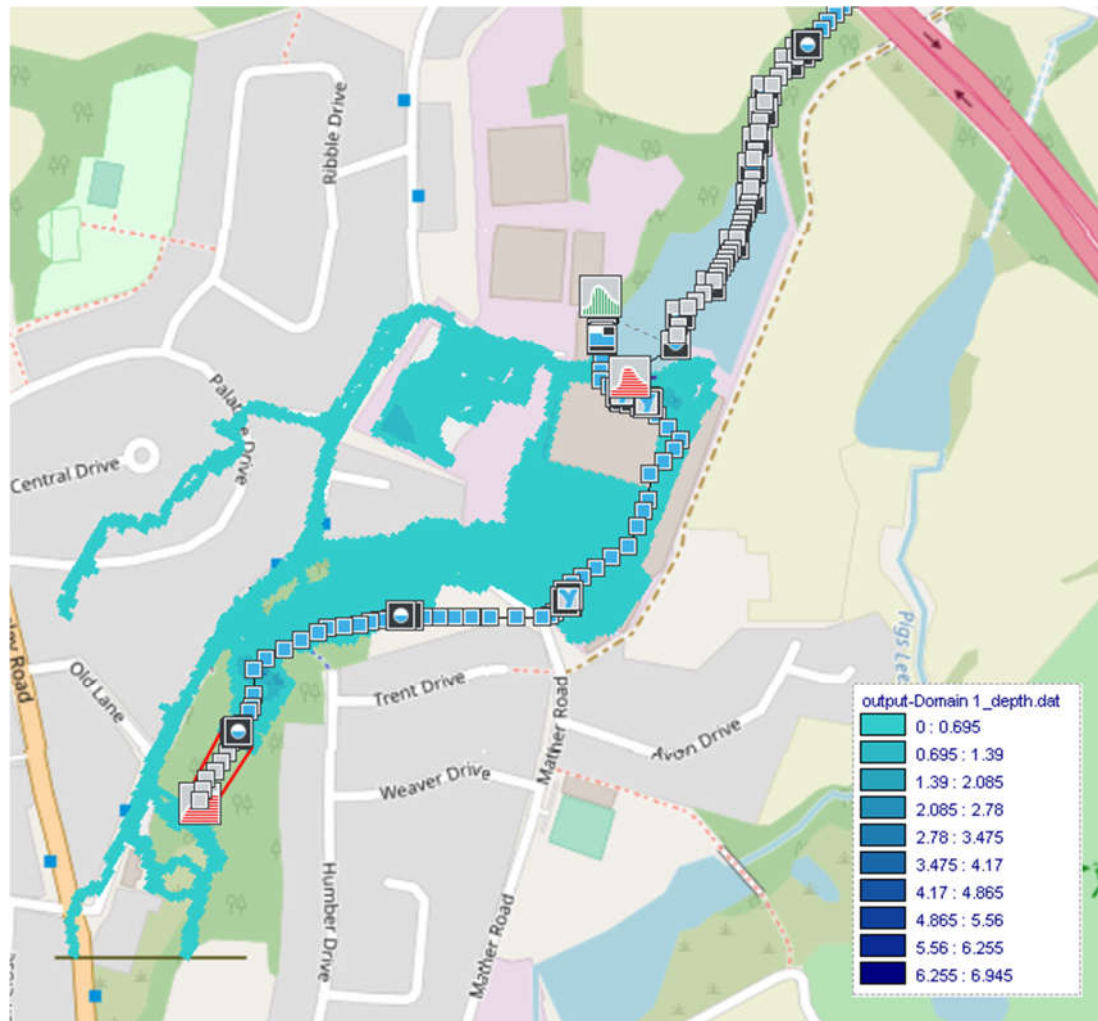


Figure 4.2: Extent of flooding from Mill Reservoir for the 1:100 year + 35% event (5.25 hour storm duration)

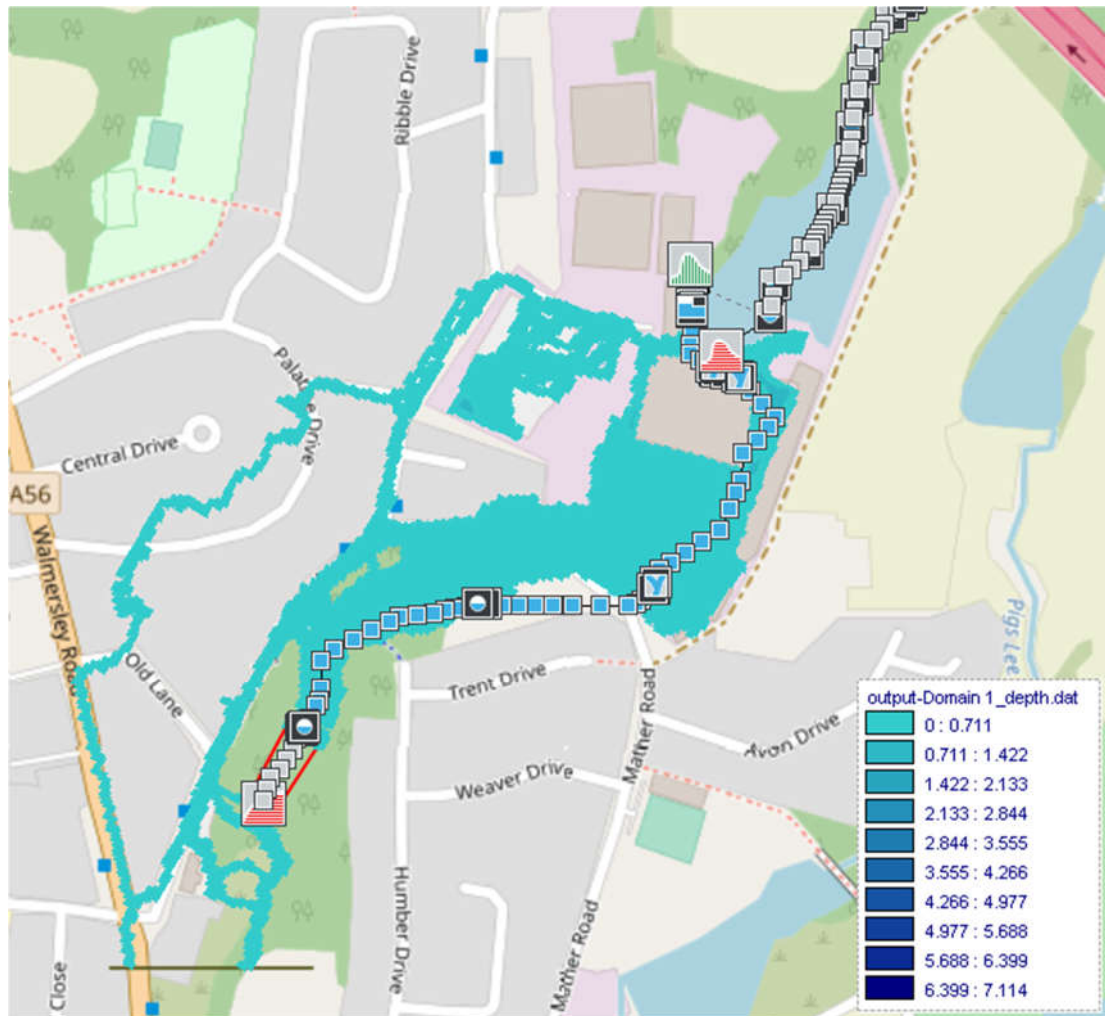


Figure 4.3: Extent of flooding from Mill Reservoir for the 1:100 year + 70% event (5.25 hour storm duration)

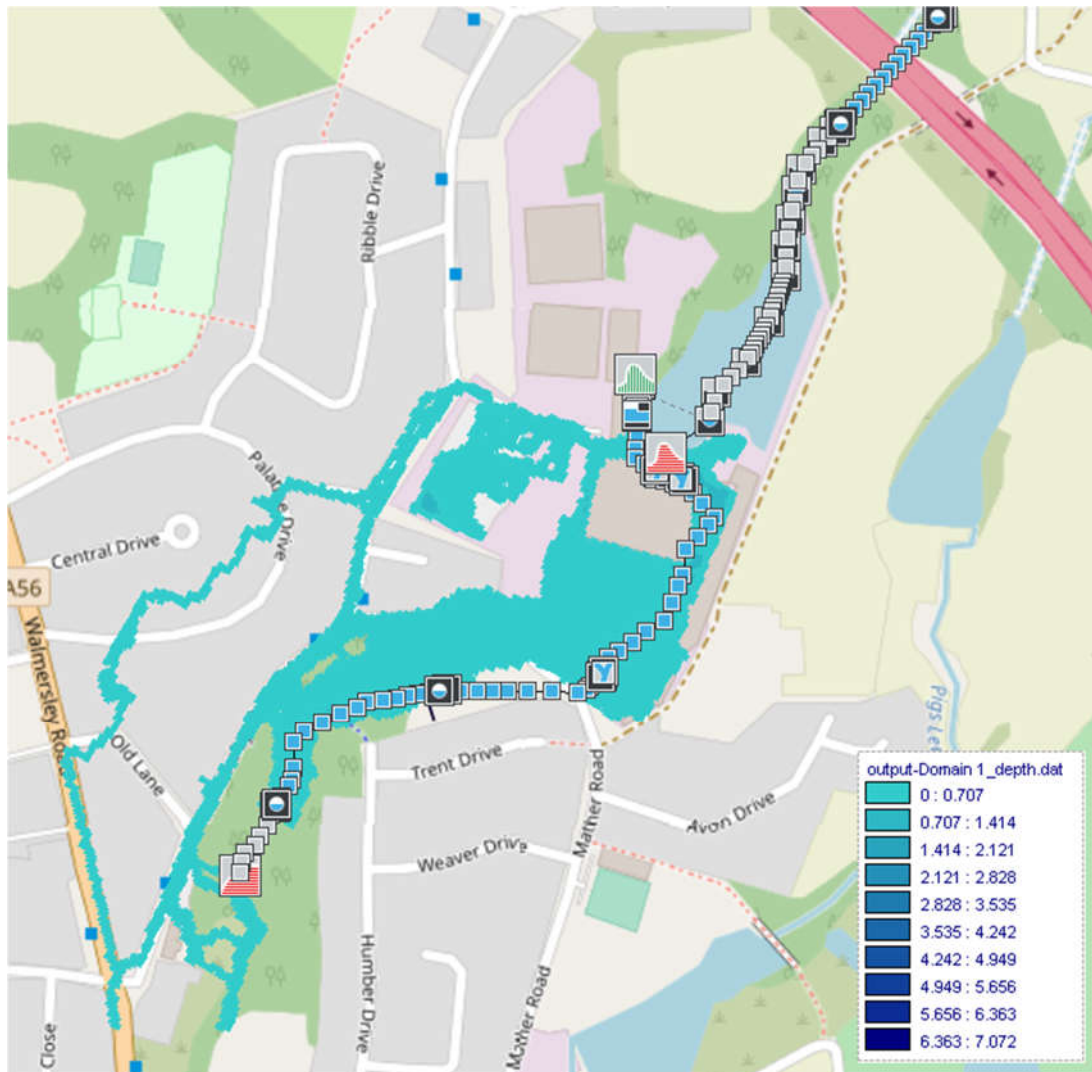


Figure 4.4: Extent of flooding from Mill Reservoir for the 1:1000 year event (5.25 hour storm duration)

Modelled Scenarios - Post Development Scenario

- 4.3 The 1D-2D linked Modified Model representing the post development site has been run for the following events:
- i) 1:20 year return period;
 - ii) 1:100 year return period;
 - iii) 1:100 year + 35% climate change return period;
 - iv) 1:100 year + 70% climate change return period;
 - v) 50% blockage of culvert trash screen downstream of new proposed open channel;
 - vi) Variation of Mannings 'n' by +20% and -20% (1:100year + 35% return period event); and
 - vii) 1:1000 year return period.

4.4 Peak water levels and flowrates at critical modelled locations are summarised in Tables 4.3 and 4.4 below.

Table 4.3: Modified Model - Maximum Stage (mAOD)*

Model Node Reference	Return Period Event								Comment
	1:20 yr	1:100 yr	1:100 yr +35% CC	1:100 yr +70% CC	1:100 yr +35% CC n +20%	1:100 yr +35% CC n -20%	1:100 yr +35% CC 50% Culvert Blockage	1:1000 yr	
PLB053 U/S of M66 culvert	155.639	156.332	156.949	157.505	156.246	158.022	156.949	157.630	
PLB308 Flood Storage Area	147.544	148.154	148.854	149.462	149.027	148.555	148.853	149.499	FSA top of bank level is 149.70mAOD minimum
PLB641 D/S end of proposed open channel	139.515	139.798	140.066	140.372	139.927	140.29	140.511	140.446	No 2D flooding for any scenario Top of bank set at 141mAOD
PLB1014 End of Model	116.235	116.336	116.427	116.516	116.497	116.331	116.427	116.517	2D flooding for the 1:1000 yr and 1:100+70% CC events only

*All scenarios run using 5.25 hr storm duration

Table 4.4: Modified Model - Maximum Discharge Rate (m3/s)*

Model Node Reference	Return Period Event								Comment
	1:20 yr	1:100 yr	1:100 yr +35% CC	1:100 yr +70% CC	1:100 yr +35% CC n +20%	1:100 yr +35% CC n -20%	1:100 yr +35% CC 50% Culvert Blockage	1:1000 yr	
Start of model	2.8	4.2	5.65	7.14	5.67	5.67	5.67	7.5	
PLB053 U/S of M66 culvert	2.67	3.893	5.241	6.623	5.433	4.874	5.241	6.968	
PLB308 Flood Storage Area	2.641	3.822	5.086	6.541	5.275	4.729	5.084	6.912	No 2D flooding for any scenario
PLB641 D/S end of proposed open channel	2.637	3.795	5.029	6.522	5.206	4.691	5.021	6.897	No 2D flooding for any scenario
PLB1014 End of Model	2.729	3.888	5.122	6.492	5.296	4.783	5.114	6.516	2D flooding for the 1:1000 yr and 1:100 +70% CC events only

*All scenarios run using 5.25 hr storm duration

1:20 year Return Event

- 4.5 All flow is contained within channel. No 2D floodflow occurs. The maximum water level within the FSA is 147.54mAOD, well below the minimum proposed top of bank level of 149.70mAOD.

1:100 year Return Event

- 4.6 All flow is contained within channel. No 2D floodflow occurs. The maximum water level within the FSA is 148.15mAOD, well below the minimum proposed top of bank level of 149.70mAOD.

1:100 year + 35% CC Return Event

- 4.7 All flow is contained within channel. No 2D floodflow occurs. The maximum water level within the FSA is 148.85mAOD, below the minimum proposed top of bank level of 149.70mAOD.

1:100 year + 70% CC Return Event

- 4.8 Flooding occurs at existing Manhole 1 located some 10m west of the site boundary at the southern end of the development. Flood duration is less than 1 hour. The flooding does not impact on any proposed dwellings. The only existing dwelling impacted by the flooding is Mill House. Floodwaters flow through wooded land before re-entering PLB Tributary. Refer to Figure 4.5 below.

- 4.9 The maximum water level within the FSA is 149.46mAOD, below the minimum proposed top of bank level of 149.7mAOD. The maximum water level at PLB641 immediately upstream of the culvert is 140.37mAOD, below the proposed top of bank level of 141.00mAOD.

50% Blockage of Culvert Trash Screen Downstream of New Proposed Open Channel (1:100year + 35% CC Return Event)

- 4.10 A culvert blockage has been modelled using the Trash Screen option in Flood Modeller. The trash screen has been modelled with a 50% blockage to simulate debris accumulation on the trash screen.
- 4.11 All flow is contained within channel. No 2D floodflow occurs. The maximum water level within the FSA is 149.853mAOD, below the minimum proposed top of bank level of 149.35mAOD. The maximum water level at PLB641 immediately upstream of the culvert blockage is 140.511mAOD, below the proposed top of bank level of 141.00mAOD.

Sensitivity Analysis: Variation of Mannings 'n' by +20% and -20% (1:100year + 35% return period event)

- 4.12 Sensitivity analysis of Manning's 'n' for the proposed model has been carried out for the 1:100 year + 35% event to represent seasonal changes in vegetation.

Mannings 'n' + 20%

- 4.13 All flow is contained within channel. No 2D floodflow occurs. The maximum water level within the FSA is 149.03mAOD, below the minimum proposed top of bank level of 149.70mAOD. The maximum water level at PLB641 immediately upstream of the culvert is 139.93mAOD, below the proposed top of bank level of 141.00mAOD,

Mannings 'n' - 20%

- 4.14 All flow is contained within channel. No 2D floodflow occurs. The maximum water level within the FSA is 148.555AOD, below the minimum proposed top of bank level of 149.70mAOD. The maximum water level at PLB641 immediately upstream of the culvert is 140.29mAOD, below the proposed top of bank level of 141.00mAOD,
- 4.15 The sensitivity analysis shows that:
- Increasing Manning's 'n' by 20% increases the peak water level in the FSA by 173mm and does not cause any out of bank flooding in the proposed open channel.
 - Reducing Manning's 'n' by 20% decreases the peak water level in the FSA by 300mm and does not cause any out of bank flooding in the proposed open channel.

1:1000 year Return Event

- 4.16 Flooding occurs at existing Manhole 1 located some 10m west of the site boundary at the southern end of the development. Flood duration is less than 1 hour. The flooding does not impact on any proposed dwellings. The only existing dwelling impacted by the flooding is Mill House. Floodwaters flow through wooded land before re-entering PLB Tributary. Refer to Figure 4.6 below.
- 4.17 The maximum water level within the FSA is 149.50mAOD, below the minimum proposed top of bank level of 149.7mAOD. The maximum water level at PLB641 immediately upstream of the culvert is 140.446mAOD, below the top of bank level of 141.00mAOD,

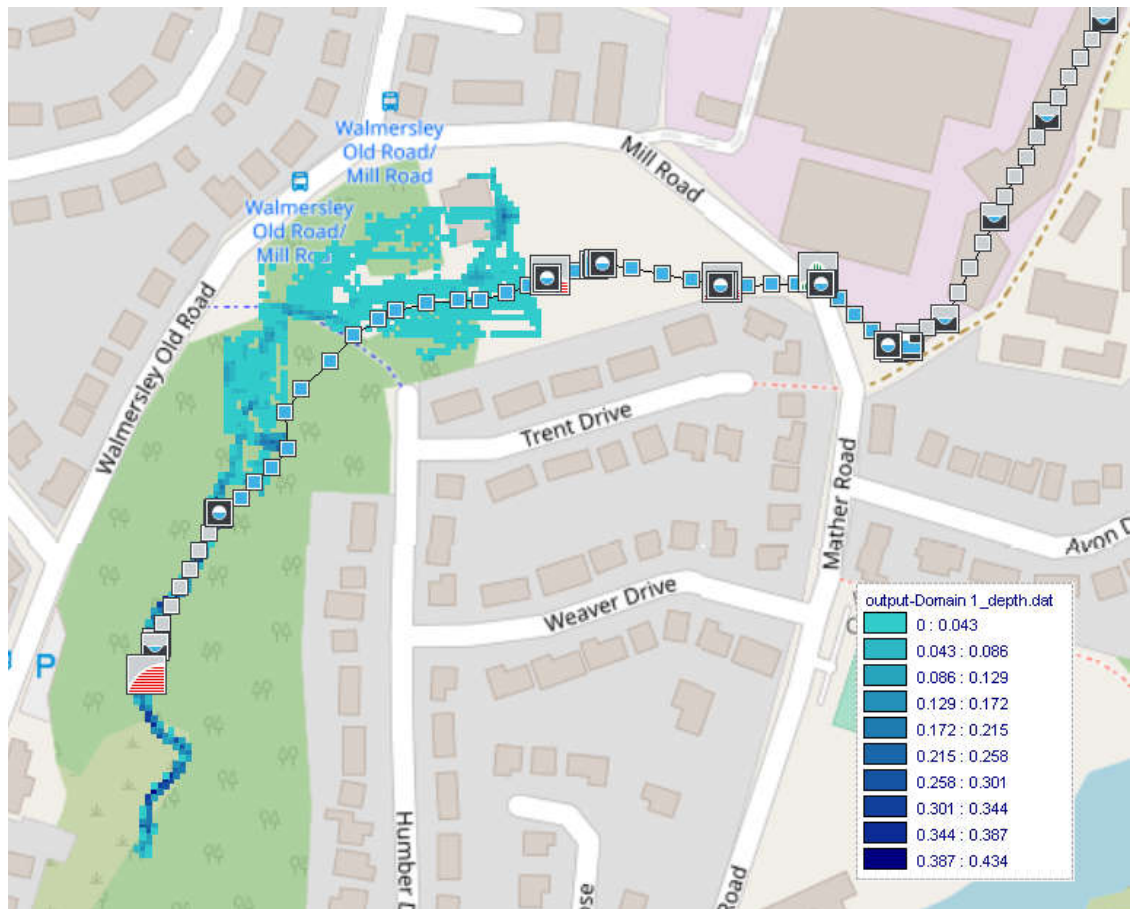


Figure 4.5: Proposed Site 2D Floodflow route (1:100yr + 70%)

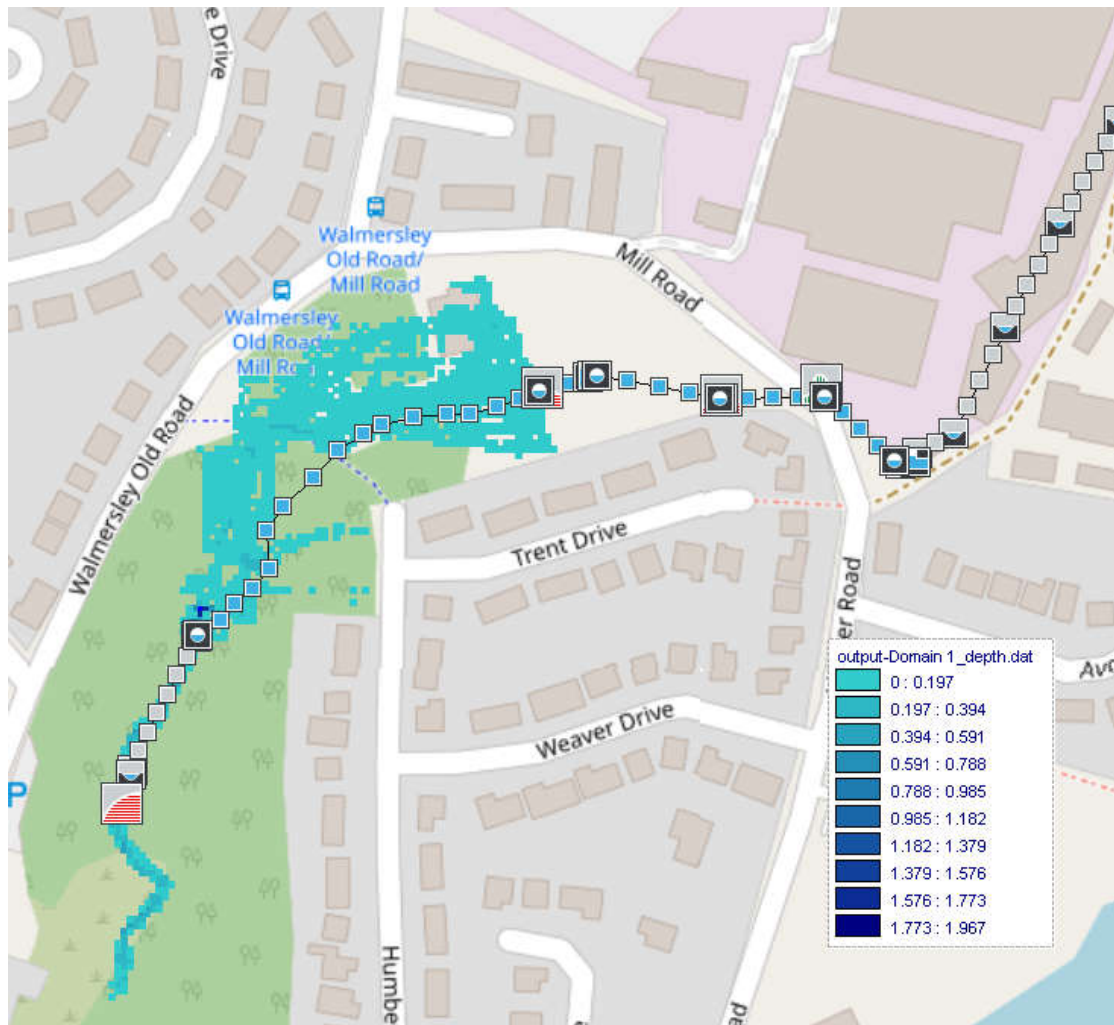


Figure 4.6: Proposed Site 2D Floodflow route (1:1000yr)

Discussion of Results

Existing Site (Summary of results from Approved Model)

4.18 Hydraulic modelling found that for the existing site:

- during the 1:20 year event all flows remain in-bank through the site, with no flooding occurring from Mill Reservoir.
- during the 1:100 year, 1:100 year+ 35% CC, 1:100 year + 70% CC and 1:1000 year events, flooding occurred within the existing site spilling from the southern bank of Mill Reservoir. Floodwaters issuing from Mill Reservoir travel southwards through the site and along Walmersley Old Road before partially re-entering the open channel section of PLB tributary some 222m downstream of the development site.
- during the 1:100 year+ 35% CC, 1:100 year + 70% CC and 1:1000 year events, a branch of flooding issues in a westerly direction from the site towards Palatine Drive;

- Mill Reservoir provides some attenuation to flood flows downstream of the development site. The reservoir response time is 1.16 hours for the 2.75 duration storm and 1 hour for the 5.25 hour duration storm.
- Compared to the 2.75 hour storm, the longer 5.25 hour storm produces a higher peak flowrate and water levels throughout the model (typically up to 100mm higher). The 5.25 hour duration storm has been used to generate hydrographs and all existing and post development modelled scenarios have been based on hydrographs produced for this storm duration.

Post Development Site (Modified Model)

4.19 Hydraulic modelling found that for the post development site:

- Variation of Manning's 'n' by +/- 20% impacts significantly on modelled water levels.
- Variation of flowrate by +/- 35% impacts significantly on modelled water levels.
- Flows remain in bank through the site during all modelled scenarios except the 1:100 year + 70% climate change and 1:1000 year events. For both events, flooding occurs at Manhole 1, some 10m west of the site boundary at the southern end of the site. However, flooding is not as extensive in comparison to the existing 1:100 year + 70% climate change and 1:1000 year flood extents as shown in Figure 4.8 below. Furthermore, the flood duration is 1 hour for the proposed scenario compared to some 4.6 hours during the existing flood scenario. There is, therefore, no increase in flood risk as a result of the proposed works. Refer to Figures 4.7 and 4.8 below.
- The FSA provides some 0.25 hour lag to flow.
- A comparison of modelled water levels at the downstream end of the model shows that the water level in the post development model is some 100 to 300mm higher than in the existing model. This difference in water level is due to a proportion of flood flow that does not re-enter the PLB Tributary in the existing site model but continues southwards past the end of the model.
- The FSA is able to accommodate flow arising from all modelled events with a minimum top of bank set at 149.70mAOD.

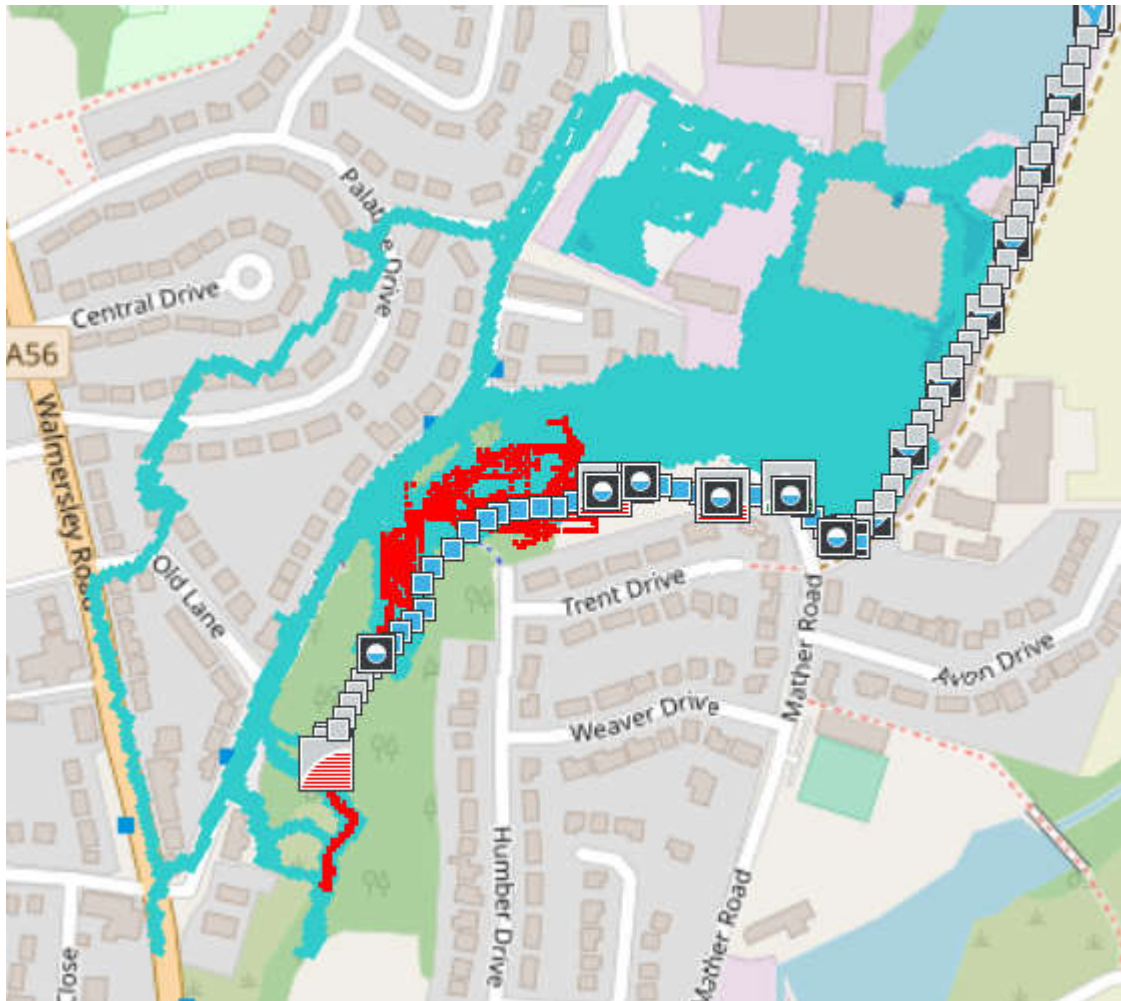


Figure 4.7: Existing (light blue) & Proposed (red) Site 2D Floodflow route & extent (1:100yr + 70%)

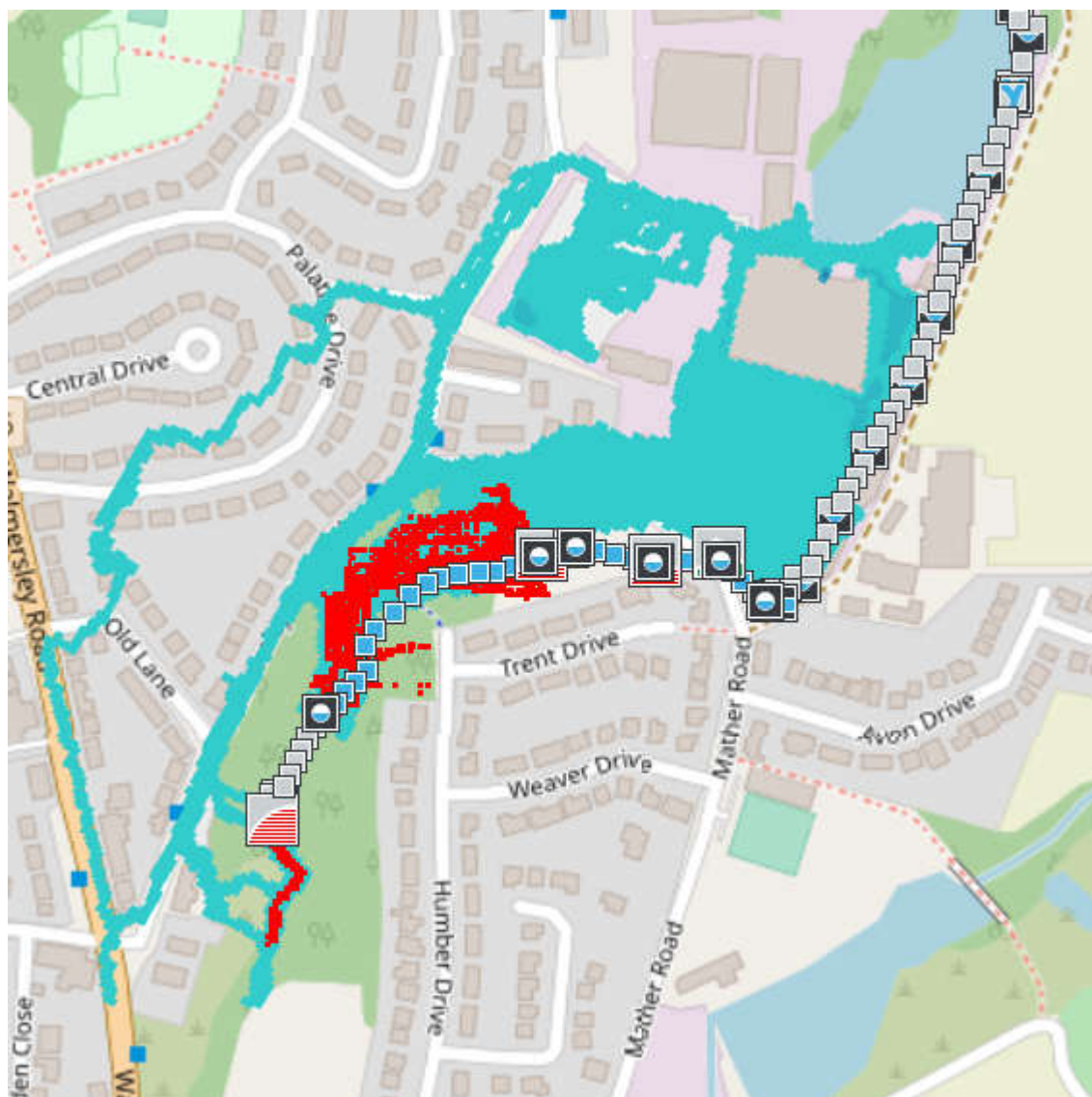


Figure 4.8: Existing (light blue) & Proposed (red) Site 2D Floodflow route & extent (1:1000yr)

Model Accuracy

Mass Balance Error

- 4.20 Mass balance errors of greater than 1% have been recorded for the 1:100 year existing event (5% error) and 1:20 year existing event (10% error). It has not been possible to reduce these errors in subsequent runs. Even if further refinement was possible, all it may show is that existing flood extent may be slightly greater than the current modelled results show. Notwithstanding this, the existing site is to be demolished. Furthermore, the modelled results establish the fact that post development flood risk will be reduced significantly when compared to the existing site for not only these two specific events, but for all modelled



events. Further modelling refinement for the existing 1:20 year and 1:100 year events (if it was possible) would not change this fact.

5.0 FLOOD RISK

Assessment of Flood Risk from Fluvial Sources

- 5.1 This section assesses fluvial flood risk arising from the design changes to the proposed watercourse as set out in Section 2.0 of this Report.

Summary of Approved Model Results

- 5.2 Although the Flood Zone Map (FZM) for the locality as produced by the EA and shown on CEC Figure 6155/500/Figure 2 [*EA Flood Zone Map*] within Appendix 1 shows that the site lies within Flood Zone 1 (Low Risk), the PLB tributary has not been modelled hydraulically by the EA. The Approved Model determined the extent of Flood Zone 3 within the existing Application site. This is shown at Figure 5.1 below.
- 5.3 The approved *Hydraulic Modelling Study Report* determined and assessed the mechanisms of flooding within the existing site arising from the PLB tributary. The Report identified that the primary flood mechanism would be rainfall events causing peak flows to exceed the in-bank capacity of Mill Reservoir resulting in overtopping.

Hydraulic Modelling Results - Approved Model

- 5.4 Approved hydraulic modelling results found that for the existing site:
- during the 1:20 year event all flows remain in-bank through the site, with no flooding occurring from Mill Reservoir.
 - during the 1:100 year, 1:100 year+ 35%, 1:100 year + 70% climate change and 1:1000 year events, flooding occurred within the existing site occurring from the southern bank of Mill Reservoir. 2D flood mapping undertaken as part of the approved *Hydraulic Modelling Study* found that floodwaters issuing from Mill Reservoir travel southwards through the site and along Walmersley Old Road before re-entering the open channel section of PLB tributary some 120m downstream of the development site.
- 5.5 Approved hydraulic modelling results found that for the post development site:
- that the proposed compensatory flood storage area (FSA) (with top of bank set at a minimum level of 149.70mAOD) and 300m reach of proposed open channel, which is to largely replace the onsite existing culvert, are able to accommodate flows arising from the 1:20 year, 1:100 year, 1:100 year+ 35% CC, 1:100 year + 70% and 1:1000 year events without flooding. The flow control overflow weir will operate during the 1:100 year + 70% climate change & 1:1000 year events;

- the proposed open channel and downstream pipework are appropriately sized to convey flows to the section of downstream existing culvert to be retained;
- flooding occurs from an existing manhole (Manhole 1) sited on the existing culvert some 10m west of the site boundary at the southern end of the site during the 1:1000 year event. However, flooding is not as extensive in comparison to the existing 1:1000 year flood extent. The flooding does not impact on any proposed dwellings. Furthermore, the flood duration is 1 hour for the proposed scenario compared to some 4.6 hours during the existing flood scenario. There is, therefore, no increase in flood risk as a result of the proposed works arising from this event; and
- flood risk is not increased as a result of the proposed works. Indeed flood risk to downstream offsite existing development will be reduced post development.

Modified Model Results

Climate Change Allowances

- 5.6 As for the previously approved *Hydraulic Modelling Study Report* the appropriate peak river flow climate change allowance to be considered for this locality within the Modified Model is 35%.
- 5.7 Results for Hydraulic modelling of the Modified Model found that for the post development site:
- that the proposed compensatory flood storage area (FSA) (with top of bank set at a minimum level of 149.70mAOD) and 300m reach of proposed open channel, which is to largely replace the onsite existing culvert, are able to accommodate flows arising from the 1:20 year, 1:100 year, 1:100 year+ 35% CC, 1:100 year + 70% and 1:1000 year events without flooding. The flow control overflow weir will operate during the 1:100 year + 70% climate change & 1:1000 year events;
 - the proposed open channel and downstream pipework are appropriately sized to convey flows to the section of downstream existing culvert to be retained;
 - flooding occurs from an existing manhole (Manhole 1) sited on the existing culvert some 10m west of the site boundary at the southern end of the site during the 1:100 year + 70% climate change and 1:1000 year events. However, flooding is not as extensive in comparison to the existing 1:100 year + 70% climate change and 1:1000 year flood extents. Refer to Figures 4.7 & 4.8 in Section 4.0. The flooding does not impact on any proposed dwellings. Furthermore, the flood duration is 1 hour for the proposed scenario compared to some 4.6 hours during the existing flood scenario. There is, therefore, no increase in flood risk as a result of the proposed works arising from this event; and

- flood risk is not increased as a result of the proposed works. Indeed flood risk to downstream offsite existing development will be reduced post development.

Fluvial Flood Risk Mitigation Measures

- 5.8 No additional flood risk mitigation measures are required beyond that proposed in the approved Flood Risk Assessment. For completeness the approved flood risk mitigation measures are reiterated below. All proposed mitigation levels remain unchanged from those presented in the approved Flood Risk Assessment.
- 5.9 The EA advise that detailed design of the proposed channel should have a preference for soft bio-engineering solutions where feasible. Geomorphological modelling has been undertaken by JBA Consulting (refer to Appendix 3). Bio-engineered solutions will be incorporated into the detailed design of the FSA and proposed open channel.
- 5.10 Setting top of bank levels no lower than 141.00mAOD at the southern (downstream) end of the proposed open watercourse channel (PLB628 - PLB641reach) will ensure that flooding within the development does not occur for fluvial flows up to the 1:1000 year event.
- 5.11 The FSA top of bank level will be set at a minimum level of 149.70mAOD so as to be able to accommodate flows arising from fluvial events up the 1:1000 year event without flooding.
- 5.12 The Applicant acknowledges that culvert repairs to the existing culvert that is to be retained will be required as part of the development implementation. This, so as to ensure that the culvert has a structural life consistent with the lifetime of the development.
- 5.13 The culverted watercourse (comprising proposed 1200mm/1500mm pipework downstream of the proposed open channel and the existing culvert to be retained) will have a minimum 10m wide easement for maintenance purposes. With respect to proposed foundations, plots 266-268 and the apartments Plots 231-248 will be founded using a piled solution to ensure that any future works on the culvert do not impact on the foundation to the properties. Refer to CEC Plan 6155/514(B) [*Proposed Culvert Diversion Route*] in Appendix 5. The culvert easement is a minimum of 5m from the centreline of the culvert, increasing with depth to circa 12m from the centreline of the culvert, where its depth is greatest. These offsets should provide sufficient room to carry out repair works in the unlikely event that it should be required.



- 5.14 The proposed open channel alignment and areas of open space adjacent the watercourse will be transferred to, and be the responsibility of a management company, that specialises in land management. Given that the culvert and its operation is fundamental in ensuring successful operation of the culverted watercourse, it is the applicant's intention to include the culvert in the transfer to the management company, along with its associated easement. Therefore, the management company will be responsible for the future maintenance of the culvert, and the open space area. The deeds to Plots 266-268 will NOT include the culvert or the easement associated with the culvert. Instead Plots 266-268 will be provided with rights of access to that area for the purpose of parking vehicles on. The management company will be funded by a fee levied on all properties on this development to cover future maintenance of the watercourse, the culvert, and all of the POS on the site.

6.0 CONCLUSIONS

- 6.1 This *Hydraulic Modelling Study and Flood Risk Assessment* has been prepared by Cole Easdon Consultants Ltd (CEC) on behalf of Barratt Homes Manchester. The Study builds on the previously approved *Hydraulic Modelling Study* that accompanied approved planning application ref: 63533 and which describes hydraulic modelling work carried out as part of a flood risk study in respect of fluvial flood risk associated with land at the former Tetrosyl Site, Walmersley Old Road, Bury, Greater Manchester.
- 6.2 The approved Development Proposal comprises the redevelopment of the former Tetrosyl factory site including the removal of Mill Reservoir and existing buildings and hardstanding and the erection of 268 dwellings with associated access, parking and landscaping.
- 6.3 The approved Modelling Study determined flood levels and the extent of fluvial flooding at the existing site associated with flooding of a tributary of Pigs Lee Brook (PLB). The approved study also quantified flood levels and extent associated with a residential development proposal comprising infilling of the existing Mill Reservoir, together with associated deculverting works of the PLB Tributary through the development site. The Study concluded that with appropriate proposed mitigation measures as set out in the Report that no flood risk was posed to the development site, and that there would be no increase in flood risk elsewhere for all events up to and including the 1:100 year+ 35% climate change event.
- 6.4 This current Report details post planning amendments to the approved hydraulic model in response to the subsequent receipt of additional drainage survey information, and detailed watercourse channel design requirements.
- 6.5 In January 2020, the Developer (Barratt Homes) undertook a drainage survey of the existing culvert from the downstream end of the Mill Reservoir to existing Manhole 1 (MH01). The surveyed route was found to deviate significantly from the route as determined by the Environment Agency. As such, it was considered necessary to divert the downstream extent of the existing culvert as the surveyed route would have placed the culvert directly below the location of proposed dwellings.
- 6.6 Furthermore, post planning geomorphological modelling undertaken by JBA Consulting in relation to the proposed open channel (deculverted watercourse) has resulted in design changes to the proposed channel cross section and longsection profiles.

- 6.7 Both the proposed culvert diversion and open channel design amendments have necessitated the need for additional hydraulic modelling to ensure the proposed changes will not result in an increase in flood risk.

Modelled Scenario - Existing Situation

- 6.8 The results from the Approved Model for the existing site scenarios remain valid for the 1:20 year, 1:100 year, 1:100 year + 35%, 1:100 year + 70% climate change and 1:1000 year events, as do the modelled floodplain extents for these events.
- 6.9 The results from the Approved Model found that for the existing site:
- during the 1:20 year event all flows remain in-bank through the site, with no flooding occurring from Mill Reservoir.
 - during the 1:100 year, 1:100 year+ 35% CC, 1:100 year + 70% CC and 1:1000 year events, flooding occurred within the existing site spilling from the southern bank of Mill Reservoir. Floodwaters issuing from Mill Reservoir travel southwards through the site and either across wooded land or down Walmersley Old Road before partially re-entering the open channel section of PLB tributary some 222m downstream of the development site.
 - during the 1:100 year+ 35% CC, 1:100 year + 70% CC and 1:1,000 year events, a branch of flooding issues in a westerly direction from the site towards Palatine Drive.

Modelled Scenario - Post Development Site

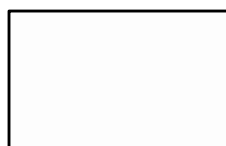
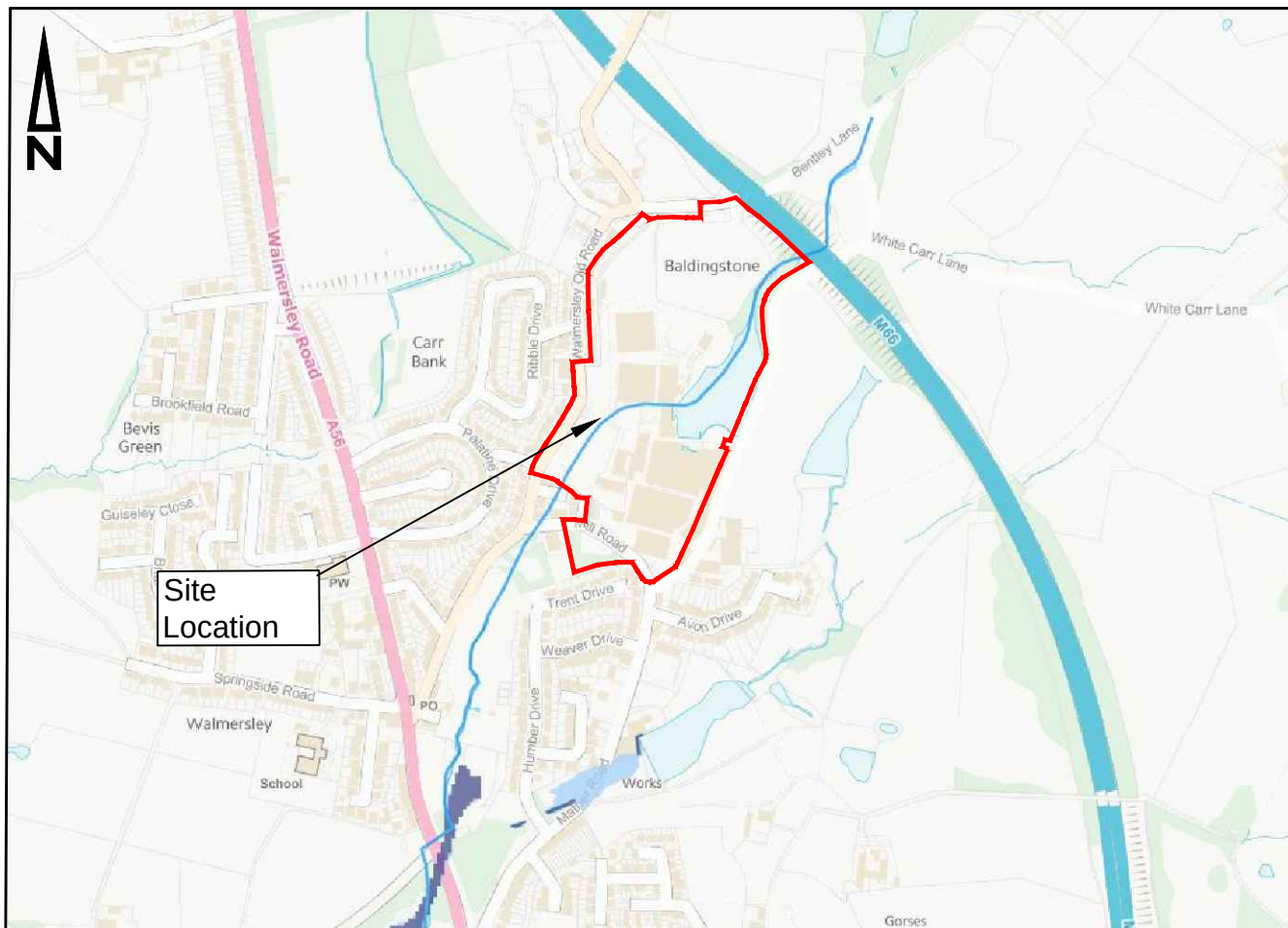
- 6.10 The Modified Model has been run for the 1:100 year, 1:100 year + 35% CC, 1:100 year + 70% CC and 1:1000 year events. The modelled results demonstrate that the proposed FSA and deculverting works can accommodate flow from all modelled scenarios without increasing flood risk to the existing downstream development or proposed site development.
- 6.11 Proposed hydraulic modelling results indicate:
- that the proposed compensatory flood storage area (FSA) (with a minimum top of bank set at 149.70mAOD) and 300m reach of open channel, which is to largely replace the onsite section of Culvert 3, are able to accommodate modelled event flows up to the 1:1000 year event without flooding. This statement remains valid for the proposed culvert diversion and open channel design amendments modelled within this study;
 - during the 1:100 year + 70% climate change and 1:1000 year events flooding occurs at Manhole 1, some 10m west of the site boundary at the southern end of the site. However, flooding is not as extensive in comparison to the existing 1:100 year + 70% climate change and 1:1000 year flood extents;



- the proposed open channel and downstream pipework are appropriately sized to convey flows to the section of downstream Culvert 3 to be retained. This statement remains valid for the proposed culvert diversion and open channel design amendments modelled within this study; and
- flood risk is not increased as a result of the proposed works. This statement remains valid for the proposed culvert diversion and open channel design amendments modelled within this study.

Cole Easdon Consultants Limited
April 2021

Appendix 1



Flood Zone 1
- Low Risk



Flood Zone 2
- Medium Risk



Flood Zone 3
- High Risk



Main Rivers

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Job Title:

Proposed Residential Development
Former Tetrosyl Site
Walmersley Old Road
Bury

Drawing Title:

EA Flood Zone Map

Client:

Barratt Homes (Manchester)

Drawn By

NJ

Checked By

DF

Date Drawn

April 2018

Drawing No.

6155/500 Figure 2

Scale

1:10,000 @ A4

Revision

Drawing Status:	
CONSTRUCTION AT CLIENT AND/OR CONTRACTOR RISK	FOR COMMENT
	FOR PLANNING
	FOR TENDER
	FOR APPROVAL
	FOR CONSTRUCTION
	AS BUILT

Appendix 2

Flood estimation report

Introduction

This report template is based on a supporting document to the Environment Agency's flood estimation guidelines. It provides a record of the hydrological context, the method statement, the calculations and decisions made during flood estimation and the results.

Contents

1	Method statement	2
2	Locations where flood estimates required	7
3	Statistical method	9
4	Revitalised flood hydrograph 2 (ReFH2) method	12
5	Discussion and summary of results	13
6	Annex	16

Approval

	Name and qualifications	Date
Method statement prepared by:	Claire French BSc (Hons) MSc (Eng) MCIWEM CWEM CSci CEnv	12/03/2018
Method statement reviewed by:	Jenni Essex BSc MSc PhD CEnv MCIWEM C.WEM	12/03/2018
Calculations prepared by:	Claire French BSc (Hons) MSc (Eng) MCIWEM CWEM CSci CEnv	12/03/2018
Calculations reviewed by:	Jenni Essex BSc MSc PhD CEnv MCIWEM C.WEM	12/03/2018

Revision History

Revision reference	Date issued	Amendments	Issued to
V1.0	12/03/2018	-	Dean Frosoni – Cole Easdon
V2.0	05/07/2019	Further consideration of storm duration	Dean Frosoni – Cole Easdon

Abbreviations

AEP	annual exceedance probability
AM.....	Annual Maximum
AREA	Catchment area (km ²)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CFMP	Catchment Flood Management Plan
CPRE.....	Council for the Protection of Rural England
FARL.....	FEH index of flood attenuation due to reservoirs and lakes
FEH.....	Flood Estimation Handbook
FSR.....	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
POT.....	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR	Standard Average Annual Rainfall (mm)
SPR.....	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

1 Method statement

1.1 Requirements for flood estimates

Overview

Flood frequency analysis of an un-named tributary of the Pigs Lee Brook is required to inform an assessment of the flood risk impacts of infilling Mill Reservoir. This reservoir is located off Walmersley Old Road, on the northern outskirts of Bury. Peak flow estimates and hydrographs are needed immediately downstream of the reservoir (at SD 80950 13850), for a range of annual exceedance probabilities (AEPs; Table 1.1a), both with and without the flood attenuation effect of the reservoir included. The hydrographs will form inputs to a hydrodynamic model that is being built by Cole Easdon.

*Table 1.1a. AEPs and corresponding return periods (T) of interest**

AEP (%)	T (years)	AEP (%)	T (years)
50	2	1.3	75
20	5	1	100
10	10	1 + 35%	100 + 35%**
5	20	1 + 70%	100 + 70%***
3.3	30	0.1	1000
2	50		

*Peak flow estimates are required for all return periods, while hydrographs are needed only for those highlighted in bold

Higher central and *upper end estimates of peak flow increases anticipated by the 2080s for the North West river basin district due to climate change¹.

1.2 The catchment

Description	The un-named tributary of the Pigs Lee Brook rises on the southern edge of Harden Moor, near Nangreaves (Map 1.2a). It flows in a general southward direction, through open farmland and Whitewall Wood, before crossing under Bentley Lane and the M66 motorway (Map 1.2b). The watercourse is designated Main River between Bentley Lane and its confluence with Pigs Lee Brook. Approximately 100 m downstream of the M66 motorway crossing, the tributary enters Mill Reservoir, located on the Tetrosyl² site. This reservoir, which dates back to at least the late 1800s³, is believed to have been constructed to supply water to Lumn cotton mills, the site's original occupier. The outflowing stream, which is culverted initially, continues for approximately 730 m, through the Bevis Green area of Bury, to its confluence with the Pigs Lee Brook. Both watercourses are culverted at this confluence, which is located approximately 45 m to the north-east of the junction of Walmersley Road and Long Lane. The catchment draining to Mill Reservoir has an area of 2.1 km². The geology of this catchment is dominated by mudstone, siltstone and sandstone of the Millstone Grit Group, although rocks belonging to the Pennine Lower Coal Measures Formation are present in the centre of the catchment. Above the bedrock, soil types vary from very acid loamy upland soils with a wet peaty surface to freely draining slightly acid loamy soils. The catchment BFIHOST and SPRHOST values are 0.447 and 37.54 respectively, indicating the predominance of impeded drainage conditions. Land cover consists mainly of agricultural land, and the
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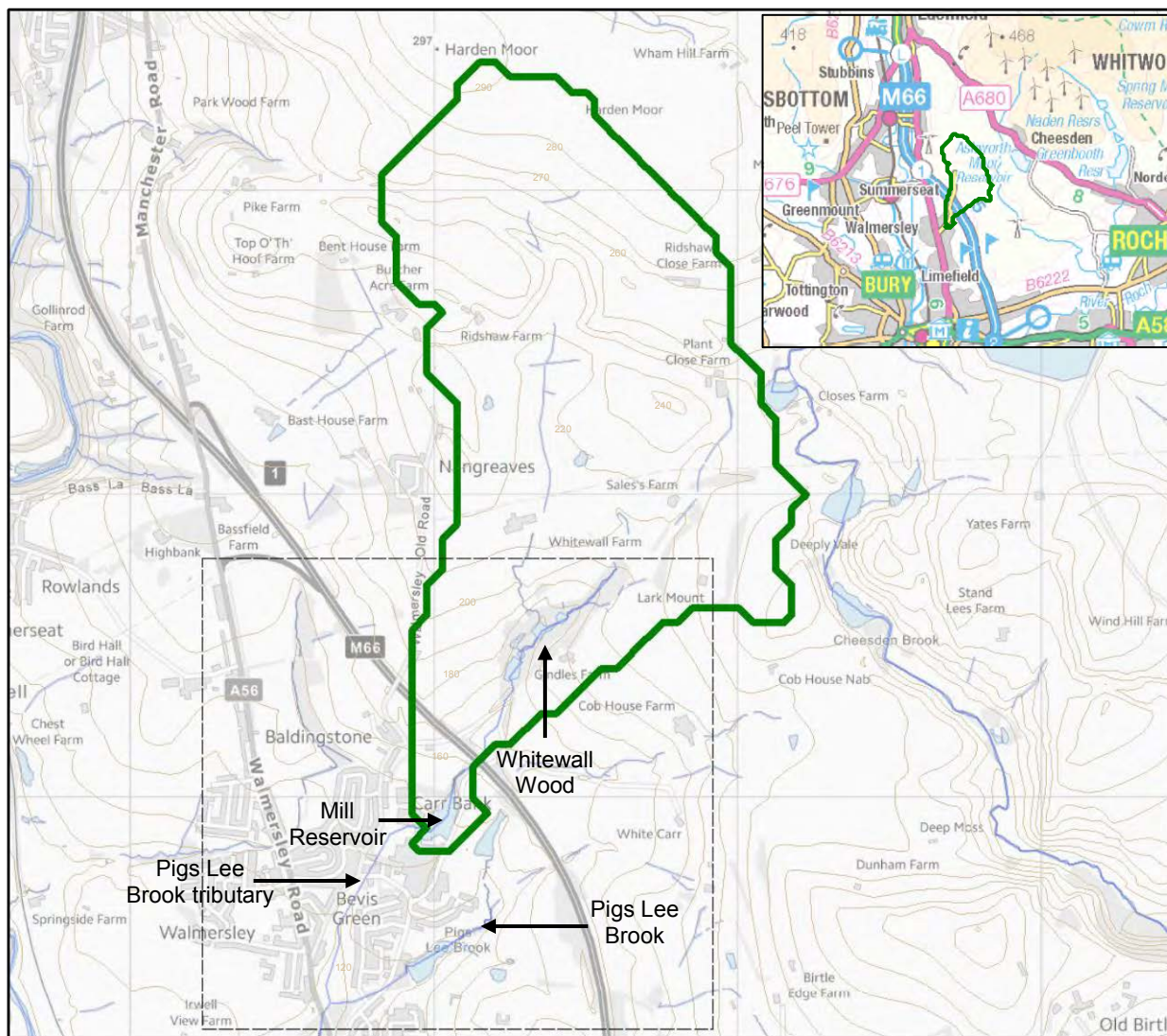
¹ <https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances>

² A chemical manufacturer, based on the site for around 45 years. Due to pressures placed upon the firm to improve its operations and respond to its market, Tetrosyl is relocating elsewhere, and redevelopment of the site for residential use is planned.

³ <http://maps.nls.uk/geo/explore/#zoom=14&lat=53.6217&lon=-2.2887&layers=1&b=1>

catchment is classified as essentially rural (URBEXT2000 of 0.012). In addition to Mill Reservoir, the catchment contains on-line reservoirs at Whitewall Wood, located approximately 300 m upstream of the M66 motorway crossing. Together, these reservoirs reportedly form part of the water supply system for the Tetrosyl site.⁴

Map 1.2a:



Study catchment

— 10 m contours

— Watercourse

--- Map 1.2b

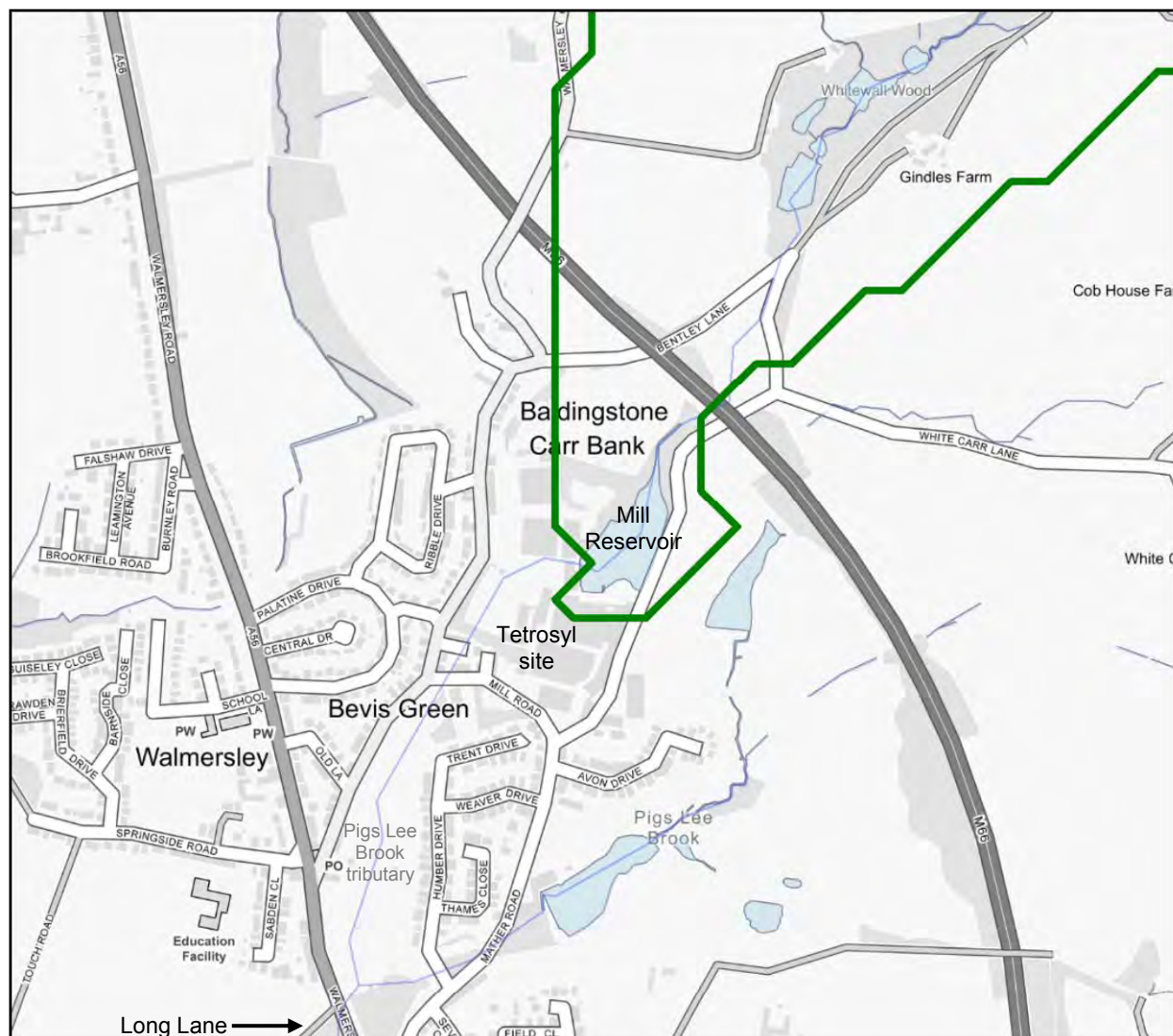


0 250 500 1,000 m
Main map only

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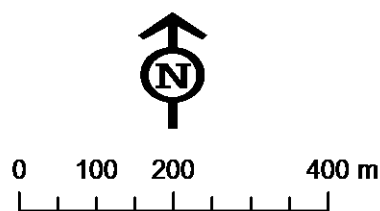
⁴ Information provided by Bury Council to PBA, in relation to PBA's (2011) FRA

Map 1.2b:



Study catchment

Watercourse



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1.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 6, released February 2018. This contains data up to water year 2015-16 for England, Wales and Northern Ireland and 28 gauges in Scotland and up to water year 2005-6 for the remaining gauges in Scotland.
--------	---

1.4 Gauging stations (flow or level)

(at or very near to the sites of flood estimates)

Water-course	Station name	Gauging authority number	NRFA number	Catchment area (km ²)	Type (rated / ultrasonic / level...)	Start of record and end if station closed
There are no gauging stations within the study catchment.						

1.5 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available?	Source of data	Details
Check flow gaugings	No	n/a	n/a	n/a
Historic flood data	Yes	Yes	PBA (2011) ⁵	As part of an FRA, undertaken by Peter Brett Associates LLP (PBA) in 2011, the Environment Agency, Bury Council, United Utilities and the Highways Agency were contacted regarding historical flooding. All four organisations responded stating that they did not hold any records of flooding in the vicinity of the site.
Flow or river level data for events	Yes	No	n/a	n/a
Rainfall data for events	Yes	No	n/a	n/a
Potential evaporation data	Yes	No	n/a	n/a
Results from previous studies	Yes	Yes	Bury Council's website	FRA prepared by PBA, in 2011, on behalf of Tetrosyl Ltd. ⁵
Other data or information	No	n/a	n/a	n/a

1.6 Hydrological understanding of catchment

Outline the conceptual model	Land lying downstream of Mill Reservoir is the main area of interest. The occurrence of flooding here will be influenced partly by the amount of flood storage available within Mill Reservoir and the other reservoirs at Whitewall Wood. PBA (2011) found that Mill Reservoir provides sufficient attenuation to prevent flooding on the Tetrosyl site during a 1 in 20 year return period event.
Any unusual catchment features to take into account?	As mentioned in Section 1.2, the study catchment contains a few on-line reservoirs. These reservoirs produce a significant flood attenuation effect (FARL = 0.858). There are no other unusual catchment features.

1.7 Initial choice of approach

Is FEH appropriate?	FEH flood frequency methods are considered appropriate, since the
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⁵ Peter Brett Associates LLP (2011) *Tetrosyl Site, Walmersley Old Road, Bury - Flood Risk Assessment*. Project Ref: 23736/001. Document Ref: Rev 1. Prepared on behalf of Tetrosyl Limited.

	study catchment drains an area greater than 0.5 km ² (the lower limit of applicability of the methods) and is not extremely heavily urbanised (FEH methods are not recommended if UBREXT2000 > 0.6).
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	To allow for the alternative approaches Cole Easdon may take to represent Mill Reservoir in its hydraulic model, both the statistical method and the ReFH2 model have been applied. The statistical method represents the flood attenuation effect due to reservoirs via the catchment descriptor FARL. The ReFH2 model itself does not take account of the flood attenuation effect, but allows subsequent routing of the design flood inflows through one or more reservoirs. Further advice on choosing between these two methods is given in Section 5.2. The ReFH2 model has been used to derive hydrograph shape in both cases. The catchment has not been split into sub-catchments.
Software to be used (with version numbers)	FEH Web Service / WINFAP-FEH v3.0.003 ⁶ / ReFH2.2

⁶ WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.

2 Locations where flood estimates required

The table below lists the locations of subject sites. The site codes are used in all subsequent tables to save space.

2.1 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km ²)	Revised AREA if altered
MillRes	L	Un-named tributary of Pigs Lee Brook	Outfall of Mill Reservoir	380950	413850	2.07	-

Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required.

Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced.

The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.

2.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000	FPEXT
MillRes	0.858* 0.924**	0.57	0.447	1.84	90.5	1249	0.012***	0.0404

Notes:

Values that have been changed from those given by the FEH Web Service or derived manually are highlighted in blue

* Existing conditions (pre-development case), i.e. including the attenuation effect of Mill Reservoir

** Post-development case, i.e. excluding the attenuation effect of Mill Reservoir

*** URBEXT2000 updated to the current year (2018)

2.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	The FEH catchment boundary for the subject site has been checked against OS 10k mapping and OS Terrain 50 data, and found to be representative of the topographic catchment of the Pigs Lee Brook tributary.
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Record how other catchment descriptors were checked and describe any changes.	FARL has been calculated manually in accordance with Section 4.3.2 of the FEH Vol. 5, for both existing conditions (i.e. the pre-development case, including the flood attenuation effect of Mill Reservoir) and the post-development case (i.e. excluding the flood attenuation effect of Mill Reservoir). The resulting value for the pre-development case (0.858) is slightly higher than the FEH value (0.837). The manually-derived value has been taken forward, for consistency in approach with the post-development case. SPRHOST and BFIHOST values have been checked against the 1:250 000 Soil Map of England and Wales and are consistent with catchment soil types. Consequently, no changes have been made to the FEH values. A check of the Urban Extent 2000 layer on the FEH Web Service against the underlying OS maps has also been made for the catchment draining to subject site MillRes. The coverage of this layer has been found to be adequate. Consequently, no changes to URBEXT2000 have been made, apart from the update of it to 2018.
Source of URBEXT	URBEXT2000
Method for updating of URBEXT	CPRE formula from 2006 CEH report on URBEXT2000

3 Statistical method

Note: Suffixes _Pre and _Post have been added to the site code to reflect existing conditions (i.e. the pre-development case, including the flood attenuation effect of Mill Reservoir) and the post-development case (i.e. excluding the flood attenuation effect of Mill Reservoir), respectively.

3.1 Overview of estimation of QMED at each subject site

Site code	QMED (rural) from CDs (m ³ /s)	Final method	Data transfer					Urban adjust -ment factor UAF	Final estimate of QMED (m ³ /s)
			NRFA numbers for donor sites used	Distance between centroids d _{ij} (km)	Moderated QMED adjustment factor, (A/B) ^a	If more than one donor			
						Weight	Weighted ave. adjustment		
MillRes_Pre	1.1	CD	-	-	-	-	-	1	1.1
MillRes_Post	1.4	CD	-	-	-	-	-	1	1.4
Are the values of QMED spatially consistent?					The QMED values appear consistent with the size and characteristics of the catchment.				
Method used for urban adjustment for subject and donor sites					WINFAP v4 ⁷				
Parameters used for WINFAP v4 urban adjustment									
Impervious fraction for built-up areas, IF			Percentage runoff for impervious surfaces, PR _{imp}			Method for calculating fractional urban cover, URBAN			
0.3			70%			From updated URBEXT2000			
Notes									
Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width; LF – Low flow statistics.									
The QMED adjustment factor A/B for each donor site is given in Table 3.2. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is (A/B) ^a times the initial estimate from catchment descriptors.									

3.2 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	Since research has shown that data transfer based on geographical distance performs better than one based solely on hydrological similarity (Kjeldsen <i>et al</i>, 2008⁸), the NRFA gauging stations (suitable for QMED) located geographically closest to the subject site have been explored as potential donors. However, none are considered suitable: all of the NRFA gauging stations within 50 km of the subject site (based on the distance between catchment centroids) are generally much larger and/or affected by reservoirs to a significantly lesser degree than the subject site. Consequently, no donor has been selected, and QMED has been estimated from catchment descriptors.
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⁷ Wallingford HydroSolutions (2016). WINFAP 4 Urban adjustment procedures.

⁸ Kjeldsen, T. R, Jones D. A. and Bayliss, A.C. (2008) Improving the FEH statistical procedures for flood frequency estimation. Science Report: SC050050. Environment Agency: Bristol.

3.3 Derivation of pooling groups

Note: Pooling group analysis has been undertaken for subject site MillRes, with FARL set to both 0.858 and 0.924 (i.e. the pre- and post-development cases, respectively). The two resulting pooling groups contain the same stations, and L-CV and L-skew values, and, hence, flood growth factors, are identical. Consequently, only the pooling group for the pre-development case is presented below.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons	Weighted average L-moments, L-CV and L-skew, (before urban adjustment)
MR_PG	MillRes	No	The default pooling group contains 16 stations and 500 station years. The WINFAP-FEH software reports that the pooling group is heterogeneous ($H2 = 2.0933$) and a review of the pooling group is desirable. The following stations have been removed from the pooling group: 76011 (Coal Burn @ Coalburn) - Artificial forest drainage, with a stream density about 60 times greater than the natural stream network; observed base flow index notably less than BFIHOST. 49006 (Camel @ Camelford) - Fairly short record (10 years) and very flat growth curve. 49005 (Bollingey Stream @ Bolingey Cocks Bridge) - Short record, only six years long. Less than the recommended eight years for pooling. 26802 (Gypsey Race @ Kirby Grindalythe) - Much more permeable than the subject site (BFIHOST = 0.959). 27073 (Brompton Beck @ Snainton Ings) - Again, much more permeable than the subject site (BFIHOST = 0.887). The following stations have been added: 27010 (Hodge Beck @ Bransdale Weir); 49003 (de Lank @ de Lank); and, 27032 (Hebden Beck @ Hebden). The modified pooling group contains 14 stations and 534 station years. The pooling group is acceptably homogeneous ($H2 = 0.4720$).	L-CV: 0.224; L-skew: 0.260

Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).

3.4 Derivation of flood growth curves at subject sites

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment*	Parameters of distribution (location, scale and shape after adjustments)	Growth factor for 100-year return period
MillRes	P	MR_PG	GL – acceptable fit and recommended in the FEH for describing flood growth in the UK	Urban adjustment made using WINFAP 4 Urban Adjustment Procedures ⁷	Location 1.000; Scale 0.218; Shape -0.262	2.95

Notes

Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis

A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters.

Growth curves were derived using the procedures from Science Report SC050050 (2008).

*Permeable adjustment not required in this case, since no permeable stations have been used in the analysis.

3.5 Flood estimates from the statistical method

Site code	Flood peak (m ³ /s) for the following return periods (in years)										
	2	5	10	20	30	50	75	100	100 +35%	100 +70%	1000
MillRes_Pre	1.1	1.5	1.8	2.2	2.4	2.7	3.0	3.3	4.4	5.6	5.8
MillRes_Post	1.4	2.0	2.4	2.8	3.1	3.5	3.9	4.2	5.7	7.2	7.5

These results suggest that infilling Mill Reservoir could increase peak flows downstream by approximately 30 per cent.

4 Revitalised flood hydrograph 2 (ReFH2) method

4.1 Parameters for ReFH2 model

Site code	Method	T _p (hours)	C _{max} (mm)	IF Impervious fraction for built-up areas	PR _{imp} % runoff for impermeable surfaces	BL (hours)	BR
MillRes	CD	1.23	319.2	0.3	70	28.15	1.32
Brief description of any flood event analysis carried out (further details should be given in the annex)				There are no data available for undertaking parameter optimisation.			
Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)							

4.2 Design events for ReFH2 method

Site code	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)	Source of design rainfall statistics (FEH99 or FEH13)
MillRes	Winter*	2.75 hours*	-	FEH13
*A storm duration of 2.75 hours is recommended by the ReFH2 software for this catchment. A winter design event has been selected given the essentially rural nature of the catchment.				
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?		Hydraulic model runs, undertaken by Cole Easdon, indicate that the existing Mill Reservoir delays the flood hydrograph by approximately 1 hour during a 100-year event. This reservoir lag time (RLAG) reduces to 0.25 hours with the proposed partial infilling of the reservoir. A longer storm duration of 5.25 hours will, therefore, also be tested. This duration has been derived using equation 8.1 of the FEH Vol. 4, with the ReFH2-recommended duration of 2.75 hours extended by adding the response time of the existing reservoir (1.16 hours) to the catchment response time (T _p = 1.23 hours).		

4.3 Flood estimates from the ReFH2 method, based on a storm duration of 2.75 hours

Note: These estimates do not take account of the reservoirs present within the catchment of the Pigs Lee Brook tributary.

Site code	Flood peak (m ³ /s) for the following return periods (in years)										
	2	5	10	20	30	50	75	100	100 +35%	100 +70%	1000
MillRes	1.3	1.8	2.3	2.7	3.0	3.3	3.6	3.9	5.2	6.6	6.6

5 Discussion and summary of results

5.1 Comparison of results from different methods

This table compares peak flows from the ReFH2 model with those from the FEH Statistical method for the post-development case.

Site code	Ratio of ReFH peak flow to FEH statistical peak		Comments
	2 year	100 year	
MillRes	0.90	0.92	The flow estimates given by the ReFH2 model are lower than those from the statistical method, even though ReFH2 does not take account of the reservoirs at Whitewall Wood whereas the statistical method does.

5.2 Final choice of method

Choice of method and reasons	To allow for the alternative approaches Cole Easdon may take to represent Mill Reservoir in its hydraulic model, two sets of flow estimates have been derived herein, as follows: - Set 1 are based on the statistical method. This method represents the flood attenuation effect due to reservoirs via the catchment descriptor FARL. Hydrographs from the ReFH2 model have then been scaled to fit the statistical peaks. This set of flow estimates should be used if Cole Easdon's model does not include Mill Reservoir, and the hydrographs are entered immediately downstream of the reservoir. - Set 2 are based solely on the ReFH2 model, which does not represent the flood attenuation effect due to reservoirs. This set of flow estimates should be adopted if Cole Easdon's hydraulic model does include Mill Reservoir and the hydrographs are entered upstream of the reservoir for routing through it. (Once the initial flood routing exercise has been undertaken, the storm duration should be adjusted for the lag effect of the reservoir(s), and the model inflow hydrographs updated accordingly.) It is important to bear in mind that, unless the hydraulic model extends upstream of Whitewall Wood, this approach will not take account of the attenuation effect of the other reservoirs within the catchment. As highlighted in Section 5.1, the flows given by the statistical method for the post-development case are higher than those given by the ReFH2 model (no attenuation). A sense check of the hydraulic modelling results should, therefore, be made, to ensure that the model does not predict more frequent flooding downstream of Mill Reservoir for the pre-development case than is consistent with the flood history of the Pigs Lee Brook tributary (bearing in mind there are no records of past flooding on the Tetrosyl site). Additional note: Since this flood estimation report was first prepared, Cole Easdon has routed the Set 1 MillRes_Post flows through a hydraulic model of Mill Reservoir (using the ReFH2 hydrographs scaled to the statistical peaks). This is a more conservative approach than using the Set 2 ReFH2 peak flows for flood routing and has been approved by the Environment Agency. The sensitivity of model output to storm duration, reflecting reservoir lag time, will be tested.
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5.3 Assumptions, limitations and uncertainty

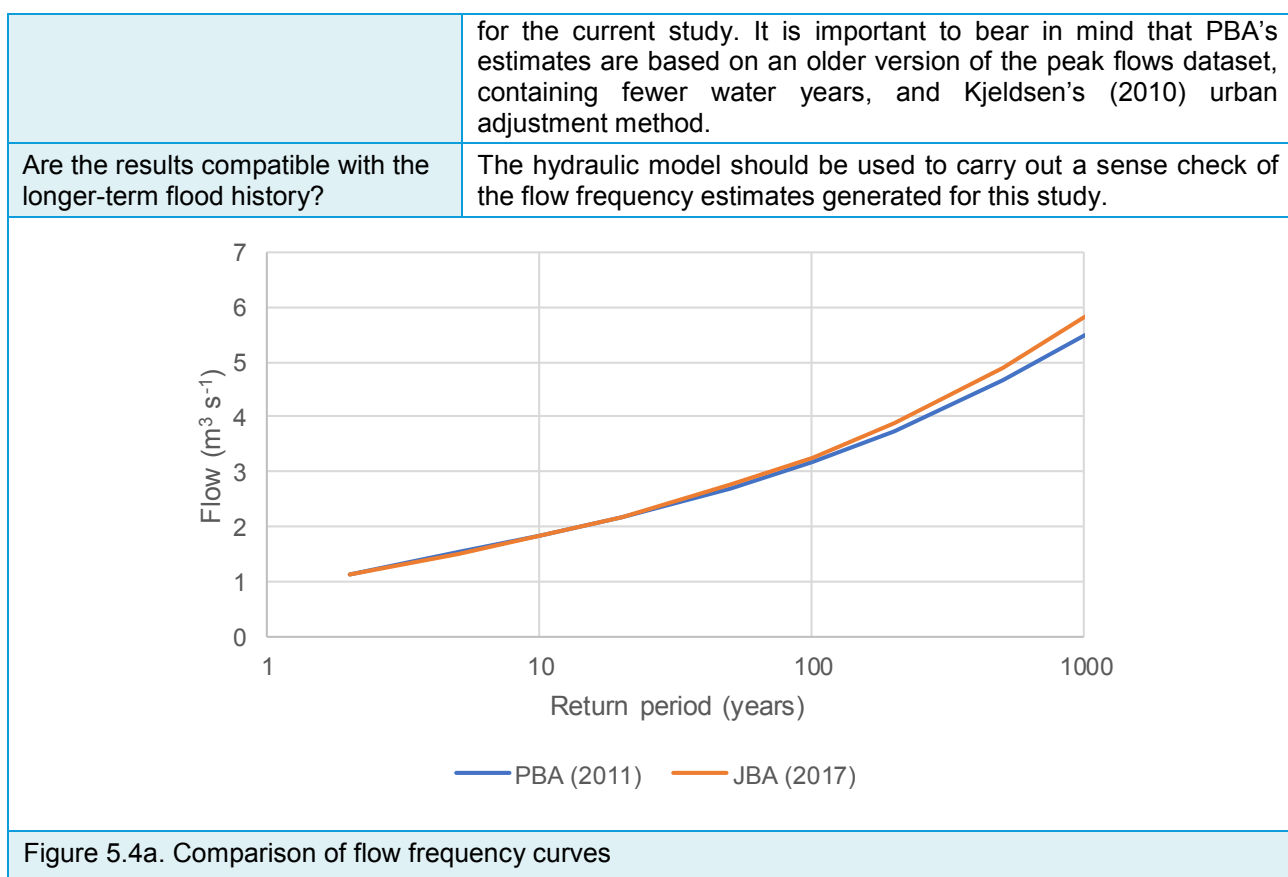
List the main assumptions made (specific to this study)	- Motorway runoff generated outside of the catchment is not discharged into the Pigs Lee Brook tributary. - The FARL values reliably capture the affect that Mill Reservoir,
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	together with the reservoirs at Whitewall Wood, have on the flood response of the Pigs Lee Brook tributary. • Mill Reservoir is kept permanently full. Reliance on the QMED equation is, therefore, reasonable, even though the FARL value based on existing conditions is less than 0.9.
Discuss any particular limitations	• The set 1 QMED estimates are based solely on catchment descriptors. • There are significant uncertainties associated with applying the statistical method to events rarer than the 200 year event, due to the typically short length of river gauge records. Consequently, the set 1 flow estimates for the 1000 year event should be treated with caution. • There are no flow records available for calibration and verification of the hydrological models.
Give what information you can on uncertainty in the results	There is no method provided in the FEH for establishing uncertainty in design flows that have been estimated for an ungauged catchment using a pooled growth curve. The uncertainty in this common situation will depend on many factors, for example, how unusual the study catchment is relative to the pooling group, and the uncertainty in flow measurement at the pooled gauging stations. A UK average measure of uncertainty is presented in a technical guidance report⁹, generated by an Environment Agency-funded R&D project into the FEH, local data and uncertainty. This report indicates that the 95% confidence limits for a 1% AEP flood estimate for a rural catchment (representative of the Pigs Lee Brook tributary), without donor adjustment, are 0.45 to 2.23 times the best estimate.
Comment on the suitability of the results for future studies	The results presented in this document should be considered in the context of current study aims and objectives. And, future analysts should satisfy themselves that no new data can be obtained that could be used to refine the estimates.
Give any other comments on the study	To increase confidence in the flow frequency estimates, it is recommended that flow monitoring is undertaken on the Pigs Lee Brook tributary. Even a short period of record would go some way to improving the estimate of QMED.

5.4 Checks

Are the results consistent, for example at confluences?	It is not possible to comment on spatial consistency, since flows have been estimated at a single point.
What do the results imply regarding the return periods of floods during the period of record?	There are no historical records of flooding available.
What is the range of 100-year growth factors? Is this realistic?	The 100 year growth factor given by the statistical method (set 1) is 2.95 and lies within the typical range. The ReFH2 model (set 2) gives a similar 100 year growth factor (3.0).
What is the range of ratios for 1000-year flow over 100-year flow?	The ratio of the final 1000 year to 100 year peak flow estimates is 1.79 for set 1 and 1.70 for set 2.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	The set 1 flow estimates for the pre-development case are fairly similar to those derived by PBA (2011) using the statistical method (see Figure 5.4a). It is worth noting, however, that PBA's flows are for a slightly larger (8 per cent) catchment area, draining to a point located approximately 635 m downstream of subject site MillRes (at NGR SD 80650 13400). Also, PBA's flood growth curve, which gives a 100 year growth factor of 2.84, is slightly flatter than that generated

⁹ Environment Agency (2017) *Using local data to reduce uncertainty in flood frequency estimation*



5.5 Final results

Set 1:

Site code	Flood peak (m³/s) for the following return periods (in years)										
	2	5	10	20	30	50	75	100	100 +35%	100 +70%	1000
MillRes_Pre	1.1	1.5	1.8	2.2	2.4	2.7	3.0	3.3	4.4	5.6	5.8
MillRes_Post	1.4	2.0	2.4	2.8	3.1	3.5	3.9	4.2	5.7	7.2	7.5

Set 2:

Site code	Flood peak (m³/s) for the following return periods (in years)										
	2	5	10	20	30	50	75	100	100 +35%	100 +70%	1000
MillRes	1.3	1.8	2.3	2.7	3.0	3.3	3.6	3.9	5.2	6.6	6.6

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)

2.75-hour duration:
2018s0142_Hydrographs.xlsx;
5.25-hour duration:
2018s0142_Hydrographs v2.0.xlsx

6 Annex

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
45816 (Haddeo @ Upton)	1.923	23	3.456	0.307	0.418	1.166
28033 (Dove @ Hollinsclough)	2.119	37	4.200	0.237	0.418	0.827
27051 (Crimple @ Burn Bridge)	2.262	44	4.539	0.223	0.156	0.818
91802 (Allt Leachdach @ Intake)	2.314	34	6.350	0.153	0.257	1.164
54022 (Severn @ Plynlimon Flume)	2.592	38	14.988	0.156	0.171	1.213
71003 (Croasdale Beck @ Croasdale Flume)	2.605	37	10.900	0.212	0.323	0.240
25003 (Trout Beck @ Moor House)	2.676	43	15.164	0.170	0.288	0.655
47022 (Tory Brook @ Newnham Park)	2.686	23	7.123	0.262	0.115	1.953
25011 (Langdon Beck @ Langdon)	2.737	28	15.878	0.238	0.318	1.120
206006 (Annalong @ Recorder)	2.895	48	15.330	0.189	0.052	2.294
25019 (Leven @ Easby)	3.033	38	5.333	0.338	0.391	1.980
27010 (Hodge Beck @ Bransdale Weir)	3.266	41	9.420	0.224	0.293	0.048
49003 (de Lank @ de Lank)	3.446	50	13.985	0.225	0.217	0.259
27032 (Hebden Beck @ Hebden)	3.454	50	3.923	0.207	0.253	0.265
<i>Total</i>		534				
<i>Weighted means</i>				0.224	0.260	

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Time (hh:mm:ss)	Time (hours)	100 yr (m ³ s ⁻¹)	100 yr + 35% (m ³ s ⁻¹)	100 yr + 70% (m ³ s ⁻¹)
00:00:00	0.00	0.14	0.19	0.23
00:15:00	0.25	0.15	0.20	0.25
00:30:00	0.50	0.19	0.25	0.32
00:45:00	0.75	0.27	0.37	0.46
01:00:00	1.00	0.44	0.60	0.75
01:15:00	1.25	0.75	1.01	1.27
01:30:00	1.50	1.25	1.69	2.12
01:45:00	1.75	1.93	2.60	3.28
02:00:00	2.00	2.68	3.62	4.56
02:15:00	2.25	3.40	4.59	5.78
02:30:00	2.50	3.95	5.34	6.72
02:45:00	2.75	4.22	5.70	7.17
03:00:00	3.00	4.18	5.64	7.11
03:15:00	3.25	3.92	5.30	6.87
03:30:00	3.50	3.64	4.79	6.03
03:45:00	3.75	3.12	4.21	5.31
04:00:00	4.00	2.71	3.66	4.61
04:15:00	4.25	2.35	3.17	3.99
04:30:00	4.50	2.03	2.74	3.45
04:45:00	4.75	1.74	2.35	2.96
05:00:00	5.00	1.48	1.99	2.51
05:15:00	5.25	1.23	1.66	2.09
05:30:00	5.50	1.01	1.36	1.71
05:45:00	5.75	0.82	1.11	1.40
06:00:00	6.00	0.69	0.93	1.17
06:15:00	6.25	0.60	0.81	1.03
06:30:00	6.50	0.55	0.75	0.94
06:45:00	6.75	0.53	0.71	0.90
07:00:00	7.00	0.51	0.69	0.87
07:15:00	7.25	0.51	0.68	0.86
07:30:00	7.50	0.50	0.68	0.85
07:45:00	7.75	0.50	0.67	0.84
08:00:00	8.00	0.49	0.66	0.84
08:15:00	8.25	0.49	0.66	0.83
08:30:00	8.50	0.48	0.65	0.82
08:45:00	8.75	0.48	0.65	0.81
09:00:00	9.00	0.48	0.64	0.81
09:15:00	9.25	0.47	0.64	0.80
09:30:00	9.50	0.47	0.63	0.79
09:45:00	9.75	0.46	0.62	0.79
10:00:00	10.00	0.46	0.62	0.78
10:15:00	10.25	0.45	0.61	0.77
10:30:00	10.50	0.45	0.61	0.77
10:45:00	10.75	0.45	0.60	0.76
11:00:00	11.00	0.44	0.60	0.75
11:15:00	11.25	0.44	0.59	0.75
11:30:00	11.50	0.43	0.59	0.74
11:45:00	11.75	0.43	0.58	0.73
12:00:00	12.00	0.43	0.58	0.73
12:15:00	12.25	0.42	0.57	0.72
12:30:00	12.50	0.42	0.57	0.71
12:45:00	12.75	0.42	0.56	0.71
13:00:00	13.00	0.41	0.56	0.70
13:15:00	13.25	0.41	0.55	0.69
13:30:00	13.50	0.40	0.55	0.69
13:45:00	13.75	0.40	0.54	0.68
14:00:00	14.00	0.40	0.54	0.68
14:15:00	14.25	0.39	0.53	0.67
14:30:00	14.50	0.39	0.53	0.66
14:45:00	14.75	0.39	0.52	0.66
15:00:00	15.00	0.38	0.52	0.65
15:15:00	15.25	0.38	0.51	0.65
15:30:00	15.50	0.38	0.51	0.64
15:45:00	15.75	0.37	0.50	0.64
16:00:00	16.00	0.37	0.50	0.63
16:15:00	16.25	0.37	0.50	0.62
16:30:00	16.50	0.36	0.49	0.62
16:45:00	16.75	0.36	0.49	0.61
17:00:00	17.00	0.36	0.48	0.61
17:15:00	17.25	0.35	0.48	0.60
17:30:00	17.50	0.35	0.47	0.60
17:45:00	17.75	0.35	0.47	0.59
18:00:00	18.00	0.35	0.47	0.59
18:15:00	18.25	0.34	0.46	0.58
18:30:00	18.50	0.34	0.46	0.58
18:45:00	18.75	0.34	0.45	0.57
19:00:00	19.00	0.33	0.45	0.57
19:15:00	19.25	0.33	0.45	0.56
19:30:00	19.50	0.33	0.44	0.56
19:45:00	19.75	0.32	0.44	0.55
20:00:00	20.00	0.32	0.43	0.55
20:15:00	20.25	0.32	0.43	0.54
20:30:00	20.50	0.32	0.43	0.54
20:45:00	20.75	0.31	0.42	0.53
21:00:00	21.00	0.31	0.42	0.53
21:15:00	21.25	0.31	0.42	0.52
21:30:00	21.50	0.30	0.41	0.52
21:45:00	21.75	0.30	0.41	0.51
22:00:00	22.00	0.30	0.40	0.51
22:15:00	22.25	0.30	0.40	0.50
22:30:00	22.50	0.29	0.40	0.50
22:45:00	22.75	0.29	0.39	0.50
23:00:00	23.00	0.28	0.39	0.49
23:15:00	23.25	0.29	0.39	0.49
23:30:00	23.50	0.28	0.38	0.48
23:45:00	23.75	0.28	0.38	0.48
24:00:00	24.00	0.28	0.38	0.47
24:15:00	24.25	0.28	0.37	0.47
24:30:00	24.50	0.27	0.37	0.47
24:45:00	24.75	0.27	0.37	0.46
25:00:00	25.00	0.27	0.36	0.46
25:15:00	25.25	0.27	0.36	0.45
25:30:00	25.50	0.26	0.36	0.45
25:45:00	25.75	0.26	0.35	0.45
26:00:00	26.00	0.26	0.35	0.44
26:15:00	26.25	0.26	0.35	0.44
26:30:00	26.50	0.26	0.34	0.43
26:45:00	26.75	0.25	0.34	0.43
27:00:00	27.00	0.25	0.34	0.43
27:15:00	27.25	0.25	0.34	0.42
27:30:00	27.50	0.25	0.33	0.42
27:45:00	27.75	0.24	0.33	0.41
28:00:00	28.00	0.24	0.33	0.41
28:15:00	28.25	0.24	0.32	0.41
28:30:00	28.50	0.24	0.32	0.40
28:45:00	28.75	0.24	0.32	0.40
29:00:00	29.00	0.23	0.32	0.40
29:15:00	29.25	0.23	0.31	0.39
29:30:00	29.50	0.23	0.31	0.39
29:45:00	29.75	0.23	0.31	0.39
30:00:00	30.00	0.23	0.30	0.38
30:15:00	30.25	0.22	0.30	0.38
30:30:00	30.50	0.22	0.30	0.38
30:45:00	30.75	0.22	0.30	0.37
31:00:00	31.00	0.22	0.29	0.37
31:15:00	31.25	0.22	0.29	0.37
31:30:00	31.50	0.21	0.29	0.36
31:45:00	31.75	0.21	0.29	0.36
32:00:00	32.00	0.21	0.28	0.36
32:15:00	32.25	0.21	0.28	0.35
32:30:00	32.50	0.21	0.28	0.35
32:45:00	32.75	0.20	0.28	0.35
33:00:00	33.00	0.20	0.27	0.34

Notes on derivation:

The 100 year hydrograph has been generated using a catchment descriptor-based ReFH2 model, and, then, scaled to fit the 100 year statistical peak based on the post-development case, i.e. excluding the attenuation effect of the Client's reservoir

33:15:00	33.25	0.20	0.27	0.34
33:30:00	33.50	0.20	0.27	0.34
33:45:00	33.75	0.20	0.27	0.34
34:00:00	34.00	0.20	0.26	0.33
34:15:00	34.25	0.19	0.26	0.33
34:30:00	34.50	0.19	0.26	0.33
34:45:00	34.75	0.19	0.26	0.32
35:00:00	35.00	0.19	0.25	0.32
35:15:00	35.25	0.19	0.25	0.32
35:30:00	35.50	0.19	0.25	0.32
35:45:00	35.75	0.18	0.25	0.31
36:00:00	36.00	0.18	0.25	0.31
36:15:00	36.25	0.18	0.24	0.31
36:30:00	36.50	0.18	0.24	0.30
36:45:00	36.75	0.18	0.24	0.30
37:00:00	37.00	0.18	0.24	0.30
37:15:00	37.25	0.17	0.24	0.30
37:30:00	37.50	0.17	0.23	0.29
37:45:00	37.75	0.17	0.23	0.29
38:00:00	38.00	0.17	0.23	0.29
38:15:00	38.25	0.17	0.23	0.29
38:30:00	38.50	0.17	0.22	0.28
38:45:00	38.75	0.17	0.22	0.28
39:00:00	39.00	0.16	0.22	0.28
39:15:00	39.25	0.16	0.22	0.28
39:30:00	39.50	0.16	0.22	0.27
39:45:00	39.75	0.16	0.22	0.27
40:00:00	40.00	0.16	0.21	0.27
40:15:00	40.25	0.16	0.21	0.27
40:30:00	40.50	0.16	0.21	0.26
40:45:00	40.75	0.15	0.21	0.26
41:00:00	41.00	0.15	0.21	0.26
41:15:00	41.25	0.15	0.20	0.26
41:30:00	41.50	0.15	0.20	0.25
41:45:00	41.75	0.15	0.20	0.25
42:00:00	42.00	0.15	0.20	0.25
42:15:00	42.25	0.15	0.20	0.25
42:30:00	42.50	0.14	0.20	0.25
42:45:00	42.75	0.14	0.19	0.24
43:00:00	43.00	0.14	0.19	0.24
43:15:00	43.25	0.14	0.19	0.24
43:30:00	43.50	0.14	0.19	0.24
43:45:00	43.75	0.14	0.19	0.23

Time (hh:mm:ss)	Time (hours)	2-yr (m³/s)	5-yr (m³/s)	10-yr (m³/s)	20-yr (m³/s)	30-yr (m³/s)	50-yr (m³/s)	75-yr (m³/s)	100-yr (m³/s)	100-yr + 35% (m³/s)	100-yr + 70% (m³/s)	1000-yr (m³/s)
00:00:00	0.00	0.12	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.18	0.23	0.14
00:15:00	0.25	0.13	0.13	0.13	0.13	0.13	0.13	0.14	0.14	0.19	0.24	0.15
00:30:00	0.50	0.13	0.14	0.14	0.14	0.15	0.15	0.16	0.16	0.21	0.27	0.18
00:45:00	0.75	0.14	0.16	0.16	0.17	0.18	0.18	0.19	0.20	0.26	0.33	0.24
01:00:00	1.00	0.17	0.19	0.20	0.21	0.22	0.23	0.25	0.26	0.35	0.44	0.34
01:15:00	1.25	0.20	0.23	0.25	0.27	0.29	0.31	0.33	0.35	0.47	0.59	0.49
01:30:00	1.50	0.24	0.29	0.33	0.36	0.38	0.41	0.45	0.47	0.63	0.80	0.70
01:45:00	1.75	0.29	0.37	0.42	0.46	0.50	0.54	0.59	0.62	0.84	1.06	0.96
02:00:00	2.00	0.36	0.47	0.54	0.60	0.65	0.71	0.78	0.83	1.11	1.40	1.30
02:15:00	2.25	0.45	0.60	0.69	0.78	0.84	0.93	1.02	1.08	1.46	1.84	1.74
02:30:00	2.50	0.56	0.76	0.88	1.00	1.09	1.21	1.33	1.42	1.92	2.42	2.33
02:45:00	2.75	0.70	0.96	1.13	1.29	1.42	1.58	1.74	1.86	2.51	3.16	3.10
03:00:00	3.00	0.87	1.21	1.42	1.64	1.80	2.01	2.22	2.38	3.21	4.05	4.03
03:15:00	3.25	1.04	1.46	1.73	2.00	2.20	2.47	2.73	2.93	3.96	4.99	5.04
03:30:00	3.50	1.20	1.69	2.02	2.34	2.58	2.89	3.21	3.45	4.66	5.87	6.00
03:45:00	3.75	1.32	1.87	2.24	2.60	2.87	3.24	3.60	3.87	5.22	6.58	6.80
04:00:00	4.00	1.39	1.98	2.37	2.76	3.05	3.44	3.83	4.12	5.57	7.01	7.31
04:15:00	4.25	1.40	2.00	2.40	2.80	3.10	3.50	3.90	4.20	5.67	7.14	7.50
04:30:00	4.50	1.37	1.96	2.35	2.74	3.04	3.44	3.83	4.13	5.57	7.02	7.41
04:45:00	4.75	1.30	1.86	2.24	2.62	2.90	3.28	3.66	3.95	5.33	6.71	7.12
05:00:00	5.00	1.22	1.74	2.10	2.45	2.72	3.07	3.43	3.70	5.00	6.29	6.69
05:15:00	5.25	1.12	1.61	1.93	2.26	2.51	2.84	3.17	3.42	4.62	5.81	6.20
05:30:00	5.50	1.03	1.47	1.76	2.06	2.29	2.59	2.89	3.12	4.21	5.30	5.66
05:45:00	5.75	0.93	1.32	1.59	1.85	2.06	2.33	2.60	2.80	3.78	4.76	5.09
06:00:00	6.00	0.82	1.17	1.41	1.64	1.82	2.06	2.30	2.48	3.35	4.21	4.50
06:15:00	6.25	0.72	1.02	1.23	1.43	1.59	1.79	2.00	2.16	2.91	3.67	3.91
06:30:00	6.50	0.63	0.88	1.05	1.23	1.36	1.53	1.71	1.84	2.49	3.13	3.34
06:45:00	6.75	0.54	0.75	0.90	1.04	1.15	1.30	1.45	1.56	2.10	2.65	2.81
07:00:00	7.00	0.47	0.65	0.77	0.89	0.98	1.10	1.22	1.32	1.78	2.24	2.36
07:15:00	7.25	0.41	0.56	0.66	0.76	0.84	0.94	1.05	1.12	1.52	1.91	2.00
07:30:00	7.50	0.36	0.49	0.58	0.67	0.73	0.82	0.91	0.98	1.32	1.66	1.71
07:45:00	7.75	0.33	0.44	0.52	0.59	0.65	0.73	0.81	0.87	1.17	1.47	1.50
08:00:00	8.00	0.31	0.41	0.47	0.54	0.59	0.66	0.73	0.78	1.06	1.33	1.34
08:15:00	8.25	0.29	0.38	0.44	0.50	0.55	0.61	0.67	0.72	0.97	1.23	1.22
08:30:00	8.50	0.27	0.36	0.42	0.47	0.52	0.57	0.63	0.68	0.91	1.15	1.13
08:45:00	8.75	0.26	0.34	0.40	0.45	0.49	0.54	0.60	0.64	0.87	1.09	1.06
09:00:00	9.00	0.25	0.33	0.38	0.43	0.47	0.53	0.58	0.62	0.83	1.05	1.02
09:15:00	9.25	0.25	0.33	0.37	0.42	0.46	0.51	0.56	0.60	0.81	1.02	0.99
09:30:00	9.50	0.24	0.32	0.37	0.42	0.45	0.50	0.55	0.59	0.80	1.00	0.97
09:45:00	9.75	0.24	0.32	0.36	0.41	0.45	0.50	0.55	0.58	0.79	0.99	0.96
10:00:00	10.00	0.24	0.31	0.36	0.41	0.44	0.49	0.54	0.58	0.78	0.98	0.95
10:15:00	10.25	0.24	0.31	0.36	0.40	0.44	0.49	0.54	0.57	0.77	0.97	0.94
10:30:00	10.50	0.24	0.31	0.36	0.40	0.44	0.48	0.53	0.57	0.77	0.97	0.93
10:45:00	10.75	0.23	0.31	0.35	0.40	0.43	0.48	0.53	0.56	0.76	0.96	0.92
11:00:00	11.00	0.23	0.30	0.35	0.39	0.43	0.48	0.52	0.56	0.75	0.95	0.91
11:15:00	11.25	0.23	0.30	0.35	0.39	0.43	0.47	0.52	0.55	0.75	0.94	0.91
11:30:00	11.50	0.23	0.30	0.34	0.39	0.42	0.47	0.51	0.55	0.74	0.93	0.90
11:45:00	11.75	0.23	0.30	0.34	0.38	0.42	0.46	0.51	0.54	0.73	0.92	0.89
12:00:00	12.00	0.22	0.29	0.34	0.38	0.41	0.46	0.50	0.54	0.73	0.92	0.88
12:15:00	12.25	0.22	0.29	0.33	0.38	0.41	0.45	0.50	0.53	0.72	0.91	0.88
12:30:00	12.50	0.22	0.29	0.33	0.37	0.41	0.45	0.50	0.53	0.71	0.90	0.87
12:45:00	12.75	0.22	0.28	0.33	0.37	0.40	0.45	0.49	0.52	0.71	0.89	0.86
13:00:00	13.00	0.22	0.28	0.32	0.37	0.40	0.44	0.49	0.52	0.70	0.88	0.85
13:15:00	13.25	0.21	0.28	0.32	0.36	0.40	0.44	0.48	0.52	0.70	0.88	0.84
13:30:00	13.50	0.21	0.28	0.32	0.36	0.39	0.44	0.48	0.51	0.69	0.87	0.84
13:45:00	13.75	0.21	0.27	0.32	0.36	0.39	0.43	0.47	0.51	0.68	0.86	0.83
14:00:00	14.00	0.21	0.27	0.31	0.35	0.39	0.43	0.47	0.50	0.68	0.85	0.82
14:15:00	14.25	0.21	0.27	0.31	0.35	0.38	0.42	0.47	0.50	0.67	0.85	0.82
14:30:00	14.50	0.20	0.27	0.31	0.35	0.38	0.42	0.46	0.49	0.67	0.84	0.81
14:45:00	14.75	0.20	0.27	0.31	0.34	0.38	0.42	0.46	0.49	0.66	0.83	0.80
15:00:00	15.00	0.20	0.26	0.30	0.34	0.37	0.41	0.45	0.48	0.65	0.82	0.79
15:15:00	15.25	0.20	0.26	0.30	0.34	0.37	0.41	0.45	0.48	0.65	0.82	0.79
15:30:00	15.50	0.20	0.26	0.30	0.34	0.37	0.41	0.45	0.48	0.64	0.81	0.78
15:45:00	15.75	0.20	0.26	0.29	0.33	0.36	0.40	0.44	0.47	0.64	0.80	0.77
16:00:00	16.00	0.19	0.25	0.29	0.33	0.36	0.40	0.44	0.47	0.63	0.79	0.77
16:15:00	16.25	0.19	0.25	0.29	0.33	0.36	0.39	0.43	0.46	0.63	0.79	0.76
16:30:00	16.50	0.19	0.25	0.29	0.32	0.35	0.39	0.43	0.46	0.62	0.78	0.75
16:45:00	16.75	0.19	0.25	0.28	0.32	0.35	0.39	0.43	0.45	0.61	0.77	0.75
17:00:00	17.00	0.19	0.24	0.28	0.32	0.35	0.38	0.42	0.45	0.61	0.77	0.74
17:15:00	17.25	0.19	0.24	0.28	0.32	0.34	0.38	0.42	0.45	0.60	0.76	0.73
17:30:00	17.50	0.18	0.24	0.28	0.31	0.34	0.38	0.42	0.44	0.60	0.75	0.73
17:45:00	17.75	0.18	0.24	0.27	0.31	0.34	0.37	0.41	0.44	0.59	0.75	0.72
18:00:00	18.00	0.18	0.24	0.27	0.31	0.33	0.37	0.41	0.44	0.59	0.74	0.71
18:15:00	18.25	0.18	0.23	0.27	0.30	0.33	0.37	0.40	0.43	0.58	0.73	0.71
18:30:00	18.50	0.18	0.23	0.27	0.30	0.33	0.36	0.40	0.43	0.58	0.73	0.70
18:45:00	18.75	0.18	0.23	0.26	0.30	0.33	0.36	0.40	0.42	0.57	0.72	0.69
19:00:00	19.00	0.17	0.23	0.26	0.30	0.32	0.36	0.39	0.42	0.57	0.71	0.69
19:15:00	19.25	0.17	0.23	0.26	0.29	0.32	0.35	0.39	0.42	0.56	0.71	0.68
19:30:00	19.50	0.17	0.22	0.26	0.29	0.32	0.35	0.39	0.41	0.56	0.70	0.68
19:45:00	19.75	0.17	0.22	0.26	0.29	0.31	0.35	0.38	0.41	0.55	0.70	0.67
20:00:00	20.00	0.17	0.22	0.25	0.29	0.31	0.35	0.38	0.41	0.55	0.69	0.66
20:15:00	20.25	0.17	0.22	0.25	0.28	0.31	0.34	0.38	0.40	0.54	0.68	0.66
20:30:00	20.50	0.17	0.22	0.25	0.28	0.31	0.34	0.37	0.40	0.54	0.68	0.65
20:45:00	20.75	0.16	0.21	0.25	0.28	0.30	0.34	0.37	0.39	0.53	0.67	0.65
21:00:00	21.00	0.16	0.21	0.24	0.28	0.30	0.33	0.37	0.39	0.53	0.67	0.64
21:15:00	21.25	0.16	0.21	0.24	0.27	0.30	0.33	0.36	0.39	0.52	0.66	0.64
21:30:00	21.50	0.16	0.21	0.24	0.27	0.30	0.33	0.36	0.38	0.52	0.65	0.63
21:45:00	21.75	0.16	0.21	0.24	0.27	0.29	0.32	0.36	0.38	0.51	0.65	0.62
22:00:00	22.00	0.16	0.21	0.24	0.27	0.29	0.32	0.35	0.38	0.51	0.64	0.62
22:15:00	22.25	0.16	0.20	0.23	0.26	0.29	0.32	0.35	0.37	0.51	0.64	0.61
22:30:00	22.50	0.15	0.20	0.23	0.26	0.29	0.32	0.35	0.37	0.50	0.63	0.61
22:45:00	22.75	0.15	0.20	0.23	0.26	0.28	0.31	0.34	0.37	0.50	0.63	0.60
23:00:00	23.00	0.15	0.20	0.23	0.26	0.28	0.31	0.34	0.36	0.49	0.62	0.60
23:15:00	23.25	0.15	0.20	0.23	0.25	0.28	0.31	0.34	0.36	0.49	0.61	0.59
23:30:00	23.50	0.15	0.19	0.22	0.25	0.28	0.30	0.34	0.36	0.48	0.61	0.59
23:45:00	23.75	0.15	0.19	0.22	0.25	0.27	0.30	0.33	0.35	0.48	0.60	0.58

27:15:00	27:25	0.13	0.17	0.20	0.22	0.24	0.27	0.29	0.31	0.42	0.53	0.51
27:30:00	27:50	0.13	0.17	0.19	0.22	0.24	0.26	0.29	0.31	0.42	0.53	0.51
27:45:00	27:75	0.13	0.17	0.19	0.22	0.24	0.26	0.29	0.31	0.42	0.52	0.50
28:00:00	28:00	0.13	0.17	0.19	0.22	0.23	0.26	0.29	0.31	0.41	0.52	0.50
28:15:00	28:25	0.13	0.16	0.19	0.21	0.23	0.26	0.28	0.30	0.41	0.51	0.50
28:30:00	28:50		0.16	0.19	0.21	0.23	0.26	0.28	0.30	0.40	0.51	0.49
28:45:00	28:75		0.16	0.19	0.21	0.23	0.25	0.28	0.30	0.40	0.51	0.49
29:00:00	29:00		0.16	0.18	0.21	0.23	0.25	0.28	0.29	0.40	0.50	0.48
29:15:00	29:25		0.16	0.18	0.21	0.22	0.25	0.27	0.29	0.39	0.50	0.48
29:30:00	29:50		0.16	0.18	0.20	0.22	0.25	0.27	0.29	0.39	0.49	0.47
29:45:00	29:75		0.16	0.18	0.20	0.22	0.24	0.27	0.29	0.39	0.49	0.47
30:00:00	30:00		0.15	0.18	0.20	0.22	0.24	0.27	0.28	0.38	0.48	0.47
30:15:00	30:25		0.15	0.18	0.20	0.22	0.24	0.26	0.28	0.38	0.48	0.46
30:30:00	30:50		0.15	0.17	0.20	0.21	0.24	0.26	0.28	0.38	0.47	0.46
30:45:00	30:75		0.15	0.17	0.20	0.21	0.24	0.26	0.28	0.37	0.47	0.45
31:00:00	31:00		0.15	0.17	0.19	0.21	0.23	0.26	0.27	0.37	0.47	0.45
31:15:00	31:25		0.15	0.17	0.19	0.21	0.23	0.25	0.27	0.37	0.46	0.45
31:30:00	31:50		0.15	0.17	0.19	0.21	0.23	0.25	0.27	0.36	0.46	0.44
31:45:00	31:75		0.15	0.17	0.19	0.21	0.23	0.25	0.27	0.36	0.45	0.44
32:00:00	32:00		0.14	0.17	0.19	0.20	0.23	0.25	0.26	0.36	0.45	0.43
32:15:00	32:25		0.14	0.16	0.19	0.20	0.22	0.25	0.26	0.35	0.45	0.43
32:30:00	32:50		0.14	0.16	0.18	0.20	0.22	0.24	0.26	0.35	0.44	0.43
32:45:00	32:75		0.14	0.16	0.18	0.20	0.22	0.24	0.26	0.35	0.44	0.42
33:00:00	33:00		0.14	0.16	0.18	0.20	0.22	0.24	0.26	0.34	0.43	0.42
33:15:00	33:25		0.14	0.16	0.18	0.19	0.22	0.24	0.25	0.34	0.43	0.42
33:30:00	33:50		0.14	0.16	0.18	0.19	0.21	0.24	0.25	0.34	0.43	0.41
33:45:00	33:75		0.14	0.16	0.18	0.19	0.21	0.23	0.25	0.34	0.42	0.41
34:00:00	34:00		0.13	0.15	0.17	0.19	0.21	0.23	0.25	0.33	0.42	0.40
34:15:00	34:25	0.13	0.15	0.17	0.19	0.21	0.23	0.24	0.33	0.42	0.40	
34:30:00	34:50	0.13	0.15	0.17	0.19	0.21	0.23	0.24	0.33	0.41	0.40	
34:45:00	34:75	0.13	0.15	0.17	0.18	0.20	0.22	0.24	0.32	0.41	0.39	
35:00:00	35:00		0.15	0.17	0.18	0.20	0.22	0.24	0.32	0.40	0.39	
35:15:00	35:25		0.15	0.17	0.18	0.20	0.22	0.24	0.32	0.40	0.39	
35:30:00	35:50		0.15	0.16	0.18	0.20	0.22	0.23	0.32	0.40	0.38	
35:45:00	35:75		0.14	0.16	0.18	0.20	0.22	0.23	0.31	0.39	0.38	
36:00:00	36:00		0.14	0.16	0.18	0.20	0.22	0.23	0.31	0.39	0.38	
36:15:00	36:25		0.14	0.16	0.17	0.19	0.21	0.23	0.31	0.39	0.37	
36:30:00	36:50		0.14	0.16	0.17	0.19	0.21	0.23	0.30	0.38	0.37	
36:45:00	36:75		0.14	0.16	0.17	0.19	0.21	0.22	0.30	0.38	0.37	
37:00:00	37:00		0.14	0.16	0.17	0.19	0.21	0.22	0.30	0.38	0.36	
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37:30:00	37:50		0.14	0.15	0.17	0.19	0.20	0.22	0.29	0.37	0.36	
37:45:00	37:75		0.13	0.15	0.17	0.18	0.20	0.22	0.29	0.37	0.35	
38:00:00	38:00		0.13	0.15	0.16	0.18	0.20	0.21	0.29	0.36	0.35	
38:15:00	38:25		0.13	0.15	0.16	0.18	0.20	0.21	0.29	0.36	0.35	
38:30:00	38:50		0.13	0.15	0.16	0.18	0.20	0.21	0.28	0.36	0.34	
38:45:00	38:75		0.13	0.15	0.16	0.18	0.20	0.21	0.28	0.35	0.34	
39:00:00	39:00			0.15	0.16	0.18	0.19	0.21	0.28	0.35	0.34	
39:15:00	39:25			0.14	0.16	0.17	0.19	0.20	0.28	0.35	0.34	
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40:00:00	40:00			0.14	0.15	0.17	0.19	0.20	0.27	0.34	0.33	
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40:30:00	40:50			0.14	0.15	0.17	0.18	0.20	0.26	0.33	0.32	
40:45:00	40:75			0.14	0.15	0.17	0.18	0.19	0.26	0.33	0.32	
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41:30:00	41:50			0.13	0.15	0.16	0.18	0.19	0.25	0.32	0.31	
41:45:00	41:75			0.13	0.14	0.16	0.18	0.19	0.25	0.32	0.31	
42:00:00	42:00			0.13	0.14	0.16	0.17	0.19	0.25	0.32	0.30	
42:15:00	42:25			0.13	0.14	0.16	0.17	0.18	0.25	0.31	0.30	
42:30:00	42:50				0.14	0.16	0.17	0.18	0.25	0.31	0.30	
42:45:00	42:75				0.14	0.15	0.17	0.18	0.24	0.31	0.30	
43:00:00	43:00				0.14	0.15	0.17	0.18	0.24	0.30	0.29	
43:15:00	43:25				0.14	0.15	0.17	0.18	0.24	0.30	0.29	
43:30:00	43:50				0.14	0.15	0.16	0.18	0.24	0.30	0.29	
43:45:00	43:75				0.13	0.15	0.16	0.17	0.24	0.30	0.29	
44:00:00	44:00				0.13	0.15	0.16	0.17	0.23	0.29	0.28	
44:15:00	44:25				0.13	0.15	0.16	0.17	0.23	0.29	0.28	
44:30:00	44:50				0.13	0.14	0.16	0.17	0.23	0.29	0.28	
44:45:00	44:75					0.14	0.16	0.17	0.23	0.29	0.28	
45:00:00	45:00					0.14	0.16	0.17	0.23	0.28	0.27	
45:15:00	45:25					0.14	0.15	0.17	0.22	0.28	0.27	
45:30:00	45:50					0.14	0.15	0.16	0.22	0.28	0.27	
45:45:00	45:75					0.14	0.15	0.16	0.22	0.28	0.27	
46:00:00	46:00					0.14	0.15	0.16	0.22	0.27	0.26	
46:15:00	46:25					0.14	0.15	0.16	0.22	0.27	0.26	
46:30:00	46:50					0.13	0.15	0.16	0.21	0.27	0.26	
46:45:00	46:75					0.13	0.15	0.16	0.21	0.27	0.26	
47:00:00	47:00					0.13	0.15	0.16	0.21	0.26	0.25	
47:15:00	47:25					0.13	0.14	0.15	0.21	0.26	0.25	
47:30:00	47:50						0.14	0.15	0.21	0.26	0.25	
47:45:00	47:75						0.14	0.15	0.20	0.26	0.25	
48:00:00	48:00						0.14	0.15	0.20	0.25	0.25	
48:15:00	48:25						0.14	0.15	0.20	0.25	0.24	
48:30:00	48:50						0.14	0.15	0.20	0.25	0.24	
48:45:00	48:75						0.14	0.15	0.20	0.25	0.24	
49:00:00	49:00						0.14	0.14	0.20	0.25	0.24	
49:15:00	49:25						0.13	0.14	0.19	0.24	0.24	
49:30:00	49:50							0.14	0.19	0.24	0.23	
49:45:00	49:75							0.14	0.19	0.24	0.23	
50:00:00	50:00							0.14	0.19	0.24	0.23	
50:15:00	50:25							0.14	0.19	0.24	0.23	
50:30:00	50:50							0.14	0.19	0.23	0.22	
50:45:00	50:75							0.14	0.18	0.23	0.22	
51:00:00	51:00										0.22	
51:15:00	51:25										0.22	
51:30:00	51:50										0.22	
51:45:00	51:75										0.22	
52:00:00	52:00										0.21	
52:15:00	52:25										0.21	
52:30:00	52:50										0.21	
52:45:00	52:75										0.21	
53:00:00	53:00										0.21	
53:15:00	53:25										0.20	
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53:45:00	53:75										0.20	
54:00:00	54:00										0.20	
54:15:00	54:25										0.20	
54:30:00	54:50										0.20	
54:45:00	54:75										0.19	

[illegible]

Appendix 3



**FORMER TETROSYL SITE
BEVIS GREEN
WARMERSLEY
BURY**

BURY DROP STRUCTURE MH6

**PHYSICAL AND COMPUTATIONAL HYDRAULIC MODEL TESTING
for
BARRETT HOMES MANCHESTER**

FINAL REPORT

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Report 20-28 March 2021

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SUMMARY

This report covers the initial testing undertaken on a physical hydraulic model of the Bury Drop Structure MH6. The physical model was constructed to a scale of 1:7 full size and operated on the basis of Froude law similarity.

Initial testing

The Bury Drop Structure MH6 physical hydraulic model was tested at a number of flow rates representative of the specified flow range. Initial testing was conducted prior to confirmation of the tailwater levels by the Client, as such free discharge conditions were assumed for initial assessment purposes. Initial testing found that the Bury Drop Structure MH6 generally performed satisfactorily for the flow rates investigated, however, minor performance issues were identified that needed to be addressed. A continuous nappe formed at each cascade, which may prevent air from freely venting to atmosphere. Trapping air has the potential to locally pressurise the shaft, which may cause structural damage. The continuous nappes formed at each platform should be disrupted and broken up in order to allow air to be freely vented.

The proposed platform ledges acted to pool water on the platform which dissipated energy at each platform, however, at higher flow rates, a smaller proportion of the receiving flow was intercepted. This resulted in high energy flows cascading onto the platform below. The energy of these nappes formed at each platform of the cascade should be reduced, in so far as can reasonably be achieved.

Development

In order to mitigate the performance issues identified during initial testing, the following modifications to the Bury Drop Structure MH6 arrangement were made:

- At each platform, a flow deflection column was installed at the downstream ledge of each platform, to one side of the shaft, adjacent to the shaft wall. The purpose of the flow deflection columns was to deflect nappe flow away from the shaft wall, creating an air gap through which the entire chamber could be vented to atmosphere. The flow deflection columns were also intended to direct flow away from the shaft man access ladders located at each platform, in so far as was possible. This was proposed in order to reduce potential ragging of solids to the access ladders and to protect them from excessive hydraulic loading.

- The height of each platform ledge was increased by 50 % from 200 mm to 300 mm. The increase in ledge height was intended to intercept a higher proportion of the flow, which will increase the depth of pooled water retained at each platform. The increase in the retained water depth will dissipate a larger proportion of the energy of the receiving flow. It should be noted that, although the increase in ledge height will dissipate more energy, the flows through the drop structure are inherently high energy, especially at the higher flow rates. Therefore, although energy dissipation will be increased, the nappe flows produced at each platform will still exhibit significantly high energy and turbulence.

Final Testing

The modified Bury Drop Structure MH6 physical hydraulic model was tested at a range of flow rates and water levels provided by and agreed upon by the Client. Hydraulic assessment of the Bury Drop Structure MH6 physical hydraulic model found that the drop structure conveyed the flows safely and performed satisfactorily for all flow rates and tailwater levels investigated. Three different general flow conditions occurred within the drop structure, depending on the water level within the shaft:

1. At the higher water levels (100 years + 70% and 1000 year return periods), the drop structure was almost completely surcharged, i.e. up towards the shaft soffit. As flow passed across the shaft soffit at surcharged operation, air was entrained into the flow from the vent provided in the model soffit. This indicates that a vented soffit must also be installed in the prototype drop structure, as failure to do so may result in the chamber becoming depressurised, which could lead to structural damage and possible accelerated failure.
Submerged flow conditions occurred throughout the drop structure at the specified flow range of 100 years + 70% and above. At these conditions, flow paths were dictated by the arrangement of the platforms and the flow was conveyed through the drop structure and into the Outlet Culvert safely.
2. At a return period of 100 years + 35%, the water level within the drop structure was approximately at mid shaft height. In the upper section of the drop structure free-surface flow occurred and the flow cascaded from each platform to the next. This regime was characterised by high levels of air entrainment at each platform. The proposed increase in the height of the platform ledges increased the depth of the retained flow on each platform, which in turn increased energy dissipation. The proposed vertical flow deflection columns at each platform ledge acted to deflect flow away from the wall of the drop structure, allowing air to freely vent to

atmosphere. The flow deflection columns also deflected flow away from the man access ladders somewhat. At the lower depths, submerged flow occurred and the flow patterns were comparable to those at the higher water levels, but with slightly lower velocities and a higher volume of entrained air.

3. At the lower tailwater levels (return periods of 2 years, 20 years and 50 years) the water levels within the drop structure were closer to the invert of the shaft. Under these conditions, flow was also conveyed safely through the drop structure. Free surface cascading flow occurred at each landing level within the drop structure, characterised by high levels of splash, spray and air entrainment. Again, the proposed increased ledge heights increased the depth of retained water on the platforms, increasing energy dissipation, and the flow deflection columns diverted flow away from the wall of the chamber, creating an air gap through which air could be freely vented to atmosphere. The water levels at the base of the drop structure shaft, dictated by the receiving water course tailwater levels, also acted to cushion the flow and increase energy dissipation.

The solids handling of the drop structure was investigated for a range of operating scenarios. Solids were cleared from the drop structure quickly and efficiently at all flow rates investigated. At the higher water levels, solids were held in suspension and transported through the drop structure. Highly buoyant solids, representative of solids such as footballs and plastic bottles, were not drawn through the Inlet Culvert due to the water depth in the drop structure being significantly higher than the soffit of the Inlet Culvert. This is representative of the prototype flow conditions.

At a flow rate of 5670 l/s, the water level was at approximately the mid height of the drop structure. In the upper section of the drop structure free-surface cascade flow was exhibited and solids were either suspended in the flow or tracked along the base of each platform, prior to being transported over the platform ledge to the receiving platform below. In the lower section of the drop structure submerged flow conditions occurred and solids were cleared in a similar manner to those at the higher flow rates and tailwater levels. There was some temporary retention of highly buoyant solids within the submerged flow region within the shaft, however, the highly buoyant solids were cleared from the shaft towards the Outlet Culvert after a short duration of operation.

At the lowest specified flow rate of 1400 l/s, all solids were cleared throughout the drop structure. Cascading free-surface flow occurred throughout the structure and all model solids (floating, suspended and settleable), were transported across each platform, either in suspension, or across the base of the platform to pass over each platform ledge to the successive receiving platform below. Solids were directed towards the central channel formed at the invert of the shaft and out into the outlet culvert within the region of fastest moving flow.

At the lowest specified test flow rate of 1400 l/s, the lowest velocities occurred throughout the drop structure. This indicates that all solids will also be cleared forward to the Outlet Culvert at the higher flow rates of 2800 l/s and 3500 l/s, at which cascading flow also occurred and the flow conditions were comparable.

CFD Modelling of Dry Weather Flow Conditions

Following physical hydraulic modelling of the storm flow conditions, analysis of the dry weather flow conditions (DWF) was conducted. Scaled physical hydraulic modelling of the DWF conditions was not appropriate, as the lower flow rates caused surface tension effects to dominate, preventing accurate analysis to be made. Therefore, Computational Fluid Dynamics (CFD) modelling of the DWF conditions was conducted at full scale. The main purpose of the DWF analysis was to determine whether silts and settleable solids would clear at the lower flow rates.

The arrangement of the drop structure was modified to aid the transport of solids. A number of additional slots were installed in the platform ledges in order to promote flow to spread and transport solids. Primary CFD analysis found that this arrangement resulted in low velocity, recirculation regions, which would be prone to solids accumulation. A revised arrangement was proposed in which a DWF flow channel was installed at Level 1 and benching was installed at Levels 2,3 and 4, in order to promote the transport of solids. CFD analysis of this revised arrangement found that at Level 1, all flow passed through the DWF channel with a high enough velocity to transport silts and settleable solid. At levels 2 - 5, CFD modelling found that temporary stranding of solids would likely occur. This is to be expected and is not detrimental to the performance of the drop structure. The velocities and turbulence throughout Levels 2 – 5 were sufficiently high to prevent long-term accumulation of solids in any one location. It should be noted that, the proposed modifications are small compared to the length scales of the storm flows. Therefore, the findings of the physical hydraulic model study of the storm flow conditions are applicable to the revised proposed DWF arrangement.



This text report is accompanied by a video record in WMV format, which provides video sequences of pertinent details of the model operation.

A handwritten signature in blue ink, appearing to read "Sean MacCarthy".

Sean MacCarthy
Director

1.0 INTRODUCTION

1.1 Description of scheme

A drop structure is proposed on a culverted watercourse to be constructed as part of a development scheme at a former chemical plant in Bury, Greater Manchester. Barratt Homes is to undertake an approved development for 267 residential dwellings at Bevis Green, Warmersley, Bury. The site is a former mill site which, in recent years, has been occupied by Tetrosyl Chemicals. See Location Plan, Figure 1.01.

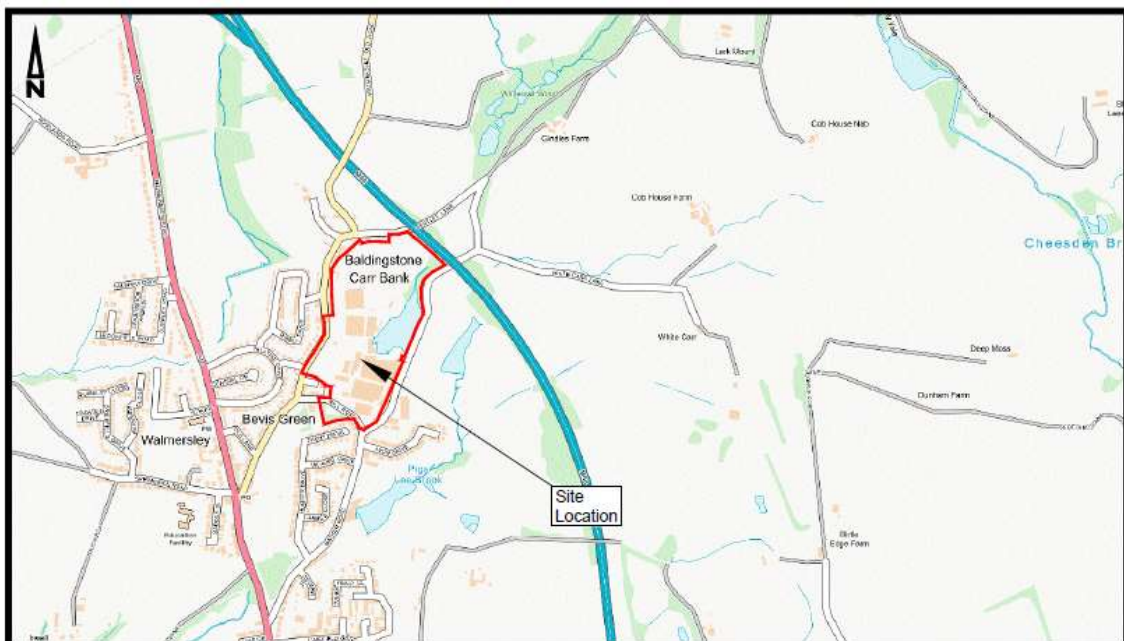


Figure 1.01 – MH6 Drop Structure Development Site Location Plan

A tributary of the Pigs Lee Brook is routed through the site, via the former mill reservoir and then southwards through a stone block culvert of varying size. The culverted watercourse exits the site at the southern boundary and enters a 1200 mm culvert before discharging to open watercourse several hundred metres downstream. As part of the approved works the former mill reservoir will be drained and replaced with a smaller Flood Storage Area (FSA). A new reach of open watercourse (some 300 m long), will be implemented to replace a length of existing culvert (downstream of the reservoir), which will be abandoned as its current alignment takes it below the proposed residential development.

Figures 1.02 and 1.03 show the route of the existing and proposed culverts, plus the diverted watercourse and the proposed location for the MH6 drop structure.

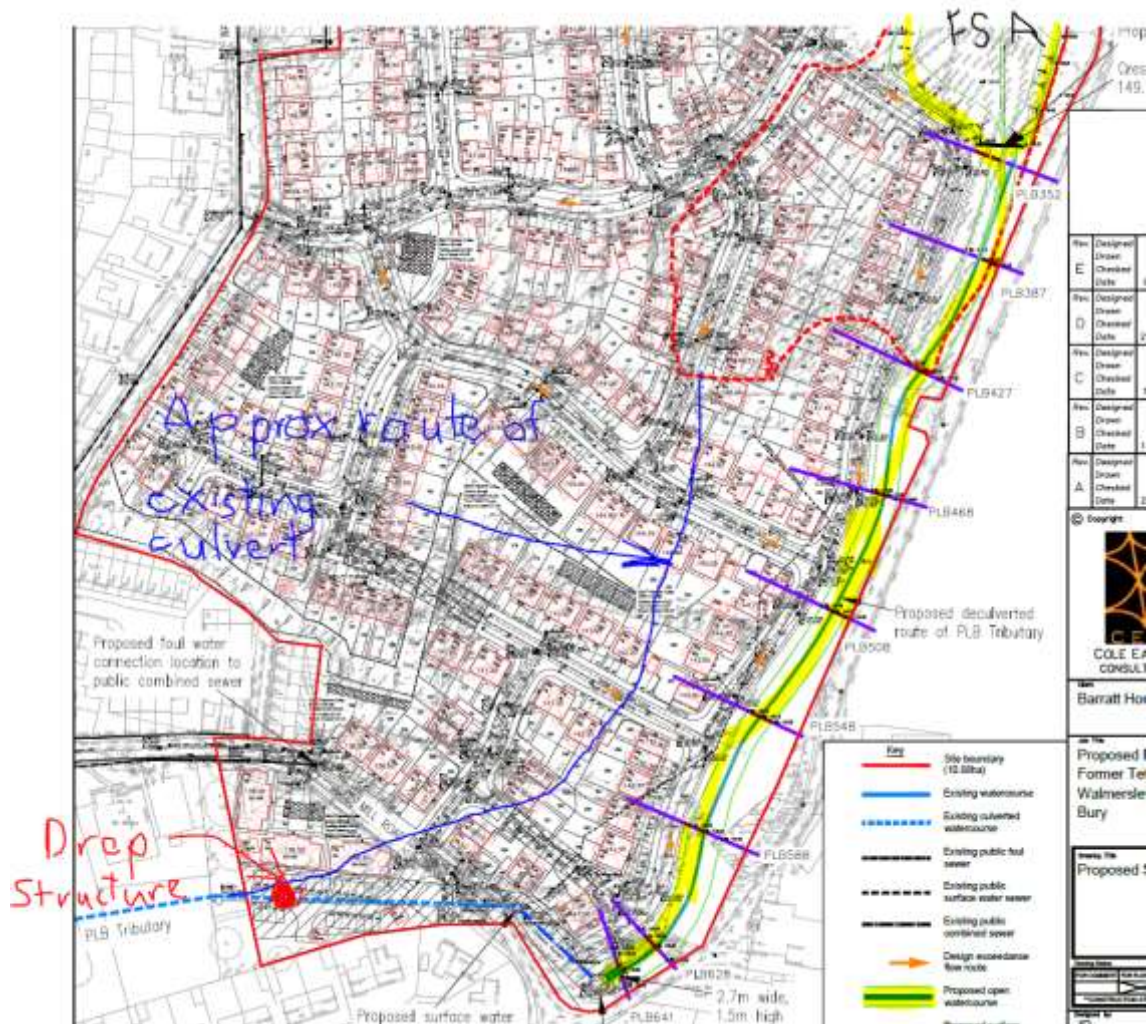


Figure 1.02 – Route of Existing Culvert, Proposed Water Course Diversion & Location of MH6

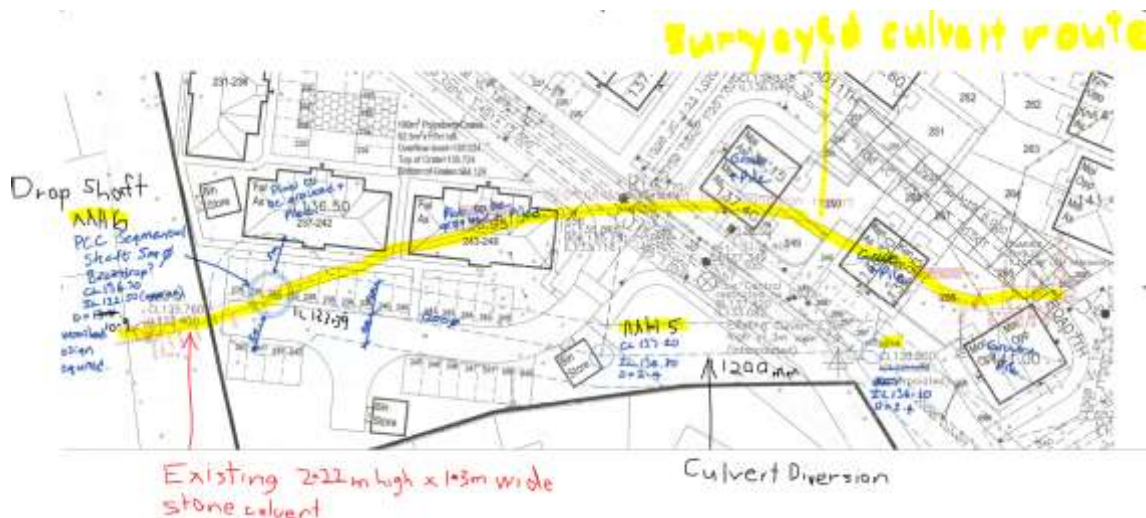


Figure 1.03 – Surveyed Route of Proposed Culvert & Location of MH6

Hydraulic flood modelling and flood risk studies have been undertaken by Cole Easdon Consultants (CEC) in support of the planning application. Detailed design of the proposed watercourse diversion has been undertaken as per the approved Flood Risk Assessment and Hydraulic Modelling Study. The design is to include a PCC segmental drop structure, drop height 10.580 m. The drop structure is to be located at the point where the proposed watercourse diversion culvert joins the existing 2.22 m high x 1.3 m wide stone culvert. Peak flows within the proposed culverted section of the watercourse are provided Table 1.01.

Table 1.01 Pigs Lee Brook – Bury Drop Structure MH6 Storm return periods and predicted flow rates (after Cole Easdon Consultants)	
Return Period (years)	Predicted Flow rate (l/s)
2	1400
5	2000
10	2400
20	2800
30	3100
50	3500
75	3900
100	4200
100 + 35%	5670
100 + 70 %	7140
1000	7500



The design of the sequential drop shaft structure was proposed by Hydrotec Consultants Ltd (HCL) prior to commencement of physical hydraulic modelling. The proposed drop shaft structure consists of a cylindrical chamber with an inlet culvert close to the top of the chamber and an outlet culvert at the base of the chamber. The outlet culvert is orientated at 29° from the inlet culvert. It is proposed to achieve the transition in height from the inlet elevation to the outlet culvert elevation with a cascade type drop shaft.

The primary purpose of a drop structure is to dissipate energy as the flow transitions from a high elevation to a low elevation, to prevent the undesirable effects of high velocity flow and to allow the safe transport of entrained air. Common design options for drop structures include the simple plunge type, the vortex type and the cascade type. The simple plunge type is a poor energy dissipator so was not considered. The vortex drop design is an efficient energy dissipator. However, this design is usually employed for large vertical drops in which the drop height is greater than 5 diameters, which is not the case for the current project. Vortex drop shafts also provide limited access for personnel and equipment. Cascade drop shafts provide efficient energy dissipation and deaeration and also provide access for personnel and equipment. For these reasons, the cascade type drop shaft was chosen for the design of the Bury Drop Structure MH6.

The inlet culvert discharges directly to a platform, Level 1, then onto platform Levels 2, 3 and 4, prior to discharge to the shaft invert and then forward into the outlet culvert. The landings are arranged at incremental orientations to allow a perpendicular inlet connection and a perpendicular orientation of the platform at Level 4 to the Outlet Culvert. Hence at a 29° offset between the Inlet Culvert and the Outlet Culvert, there are 3 incremental orientations of platforms throughout the drop (Level 1 to Level 2, Level 2 to Level 3 and Level 3 to Level 4). The incremental offset of the platforms at each level is, therefore, is $29^\circ/3 \approx 9.7^\circ$.

The landings are identical in plan and occupy slightly more than half of the chamber plan section. Each platform consists of a landing bay, ledge, access ladder and handrail (note that the handrails are not included in the physical hydraulic model, their dimension is too small to be adequately scaled.). The vertical access ladders are fully representative. A footing slab is provided for the access ladder at each landing junction in order to provide a flat, level man access, raised above the landing elevation.



The landing ledge is located at the downstream edge of the platform, this is to produce a retained water to pool behind on the platform to aid in energy dissipation of the free fall jet. The landing ledge does not extend across the full width of the platforms. A 750 mm opening is provided to one side of landing ledge in order to allow drainage during low flow conditions, and also to preferentially direct lower flows away from the access ladders.

Laterally and transversely sloped benching is proposed to form the shaft invert at the base of the cascade structure. This benching is intended to direct flow centrally and promote solids particle tracking towards the shaft centreline and into the outlet culvert within the fastest moving current at the culvert centreline. This is proposed in order to aid self-cleansing characteristics.

The detailed general arrangements of the proposed cascade drop shaft structure for MH6 are shown on Drawings 01 to 04 in Appendix 2. Figure 1.04 details the general terminology used throughout this report.

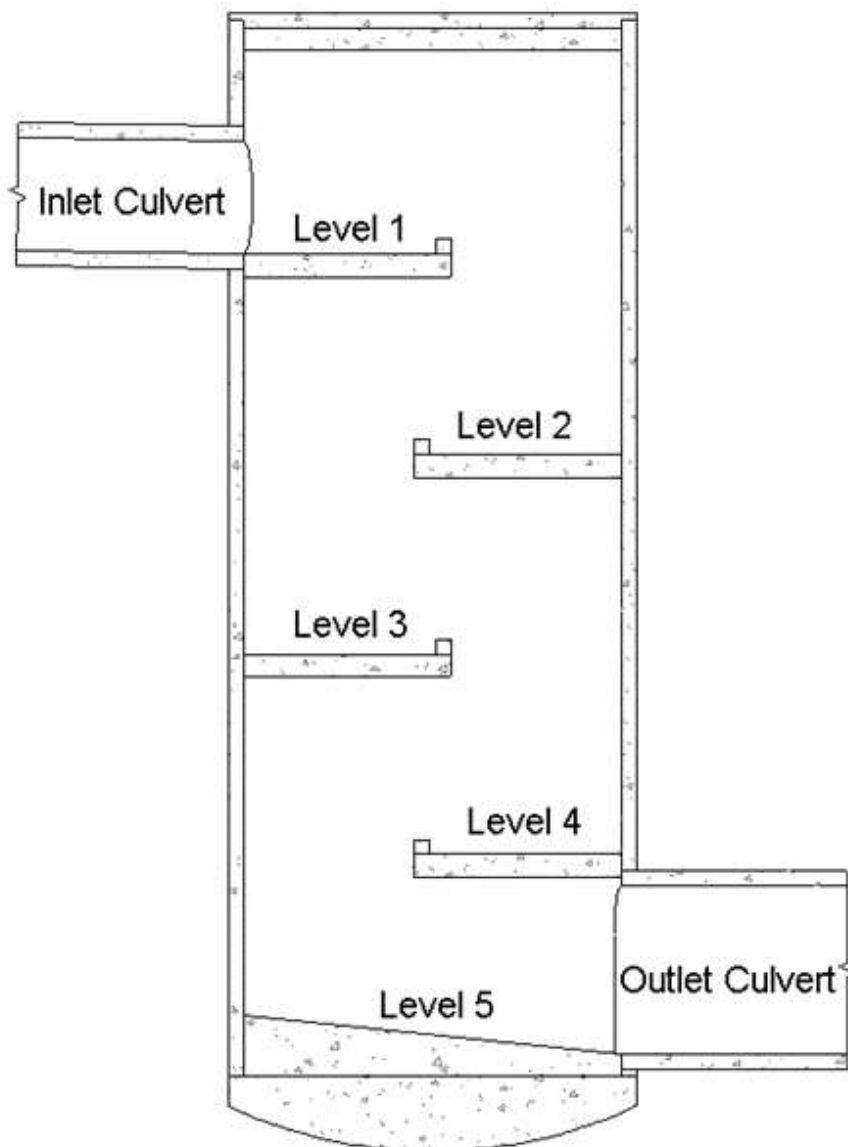


Figure 1.04 – General arrangement of Bury Drop Structure MH6, detailing the terminology used throughout this report

1.2 Purpose of testing

The purpose of the physical hydraulic model test of the MH6 Drop Structure was, as a minimum, to satisfy the following primary objectives: -

- Confirmation of satisfactory hydraulic performance at the Drop Structure for all test case scenarios of inflow and water level.
- Confirmation of satisfactory solids handling and removal from the vicinity of the structure.

1.3 Determination of model scale

In order to ensure efficient and reliable operation of the proposed drop structure, it was essential that the physical model testing was undertaken at a suitably large scale in order to ensure dynamic similarity at all pertinent flow cases.

In selecting the optimum scale for a model, consideration must be given to a range of factors both practical and theoretical. Practical considerations such as the circulating pump capacity, depth of model, the floor space utilised and ultimately cost, place limits on the maximum size of a model. Theoretical considerations regarding the correct representation of flow conditions places limits on the minimum size of a model.

The correct representation of flow conditions principally requires the Reynolds Number to be above a critical value within the model to ensure that, where fully turbulent conditions are present in the prototype, similar turbulent conditions exist within the model. If the model is too small this condition will not be met.

In mixed closed conduit and free surface models, such as drop shaft type applications, it is necessary to ensure that the Reynolds Number (Re) at surcharged locations exceeds 30,000 with a factor of safety of 2.0, hence a value of not less than 60,000, and that the Reynolds Number at free surface regions exceeds 5,000. This is to ensure that viscous scale effects are minimised and to ensure that, where fully turbulent conditions exist on the prototype, they are similarly represented on the model.

The drop shaft was modelled at a scale of 1:7. Table 1.02 shows the prototype and model Reynolds numbers at the specified minimum and maximum flows. The Reynolds numbers were calculated at the Outlet Culvert, assuming full depth. This is standard practice as it results in a conservative calculation, as if the Outlet Culvert ran at a lower flow depth for a particular flow rate, the Reynold number would be greater. As shown, the minimum Reynolds number is above the limiting value of 60,000, hence all model flows were in fully turbulent regime.

Table 1.02 Bury Drop Structure MH6 Reynolds numbers			
Return Period (years)	Prototype Flow Rate (l/s)	Reynolds Number	
		Prototype	Model
2	1400	1,148,983	61,336
1000	7500	6,155,268	330,758

1.4 Flow rates and operating philosophy

Upon agreement with the Client, the physical hydraulic model was tested at the flow rates and tailwater levels shown in Table 1.03. This provided a representative range of operating scenarios to investigate the flow conditions within the drop shaft. The tailwater levels for each flow rate were provided by the Client and calculated by CEC through numerical hydraulic modelling.

Table 1.03 Bury Drop Structure MH6 Model test flow rates and tailwater levels		
Return Period (years)	Flow rate (l/s)	Tailwater level (mAD)
2	1400	123.391
20	2800	123.771
50	3500	123.910
100 + 35%	5670	129.326
100 + 70 %	7140	136.016
1000	7500	136.140

Note: Tail water level is defined here as the water depth a (hydraulically) short distance downstream of the entrance to the outlet culvert.

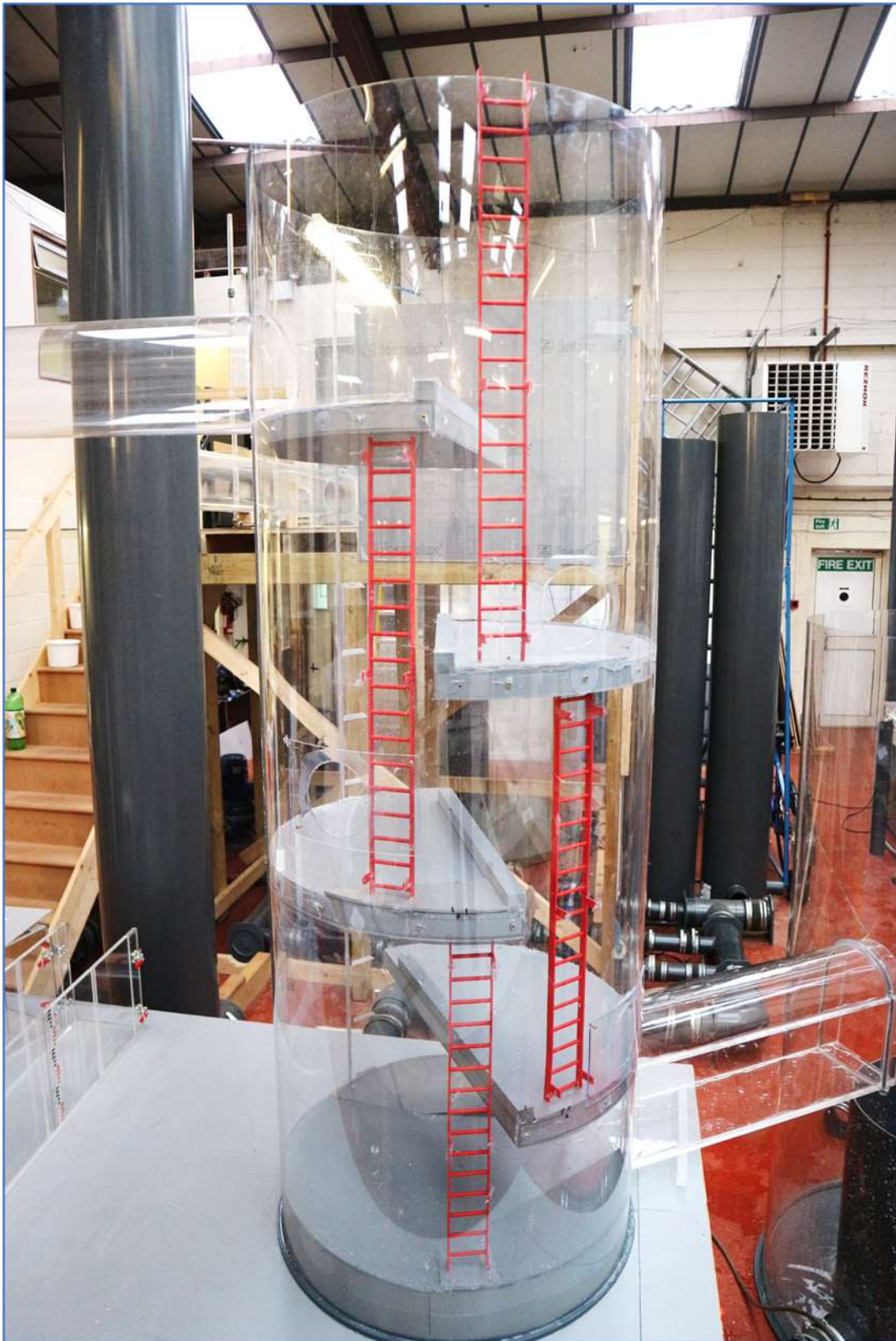
1.5 Description of model

The model of Drop Structure MH6 includes sufficient length of the proposed inlet culvert in order to allow a representative flow path to be produced on approach to MH6. The full proposed initial arrangement of MH6 was physically modelled and included all necessary benching, ledges, walls, and all relevant internal details. Sufficient length of the existing 2.22m high x 1.3m wide stone outlet culvert was provided, in order to allow a representative flow path to be produced at the exit from to MH6, hence not hydraulically short.

The model was constructed at 1/7th scale, and operated to undertake base-line tests, development and confirmatory tests for all pertinent flow and water level cases. Provision was made to allow representation of the full range of inflows from the diverted water course and the full range of tailwater levels within the receiving culvert.

The model was designed in accordance with detail extracted from the Client's general arrangement drawings, as provided, and the approved initial concept arrangement drawings. The model inlet boundary comprises a vertical plane within the proposed inlet culvert at not less than 8 x diameters upstream of MH6. Flow training measures were installed at the inlet and outlet boundaries to ensure minimum disturbances from model recirculation tanks and pipework.

General views of the initially proposed Drop Structure MH6 arrangement, prior to any development or modifications, are provided in Images 1.01 to 1.04.



**Image 1.01 – General overview of the Drop Structure MH6
physical hydraulic model**



Image 1.02 – Inlet Culvert and Level 1 platform



Image 1.03 – Platforms at Levels 2 and 3



Image 1.04 – Level 4 platform and the base of the drop shaft at Level 5

1.6 Measurement of model recirculation flows

The physical hydraulic model was equipped with external recirculating pumps, control valves and orifice plate flow meters, to enable adjustment and accurate setting of the individual flows. Sufficient model pumping capacity was provided to achieve the specified maximum design inflow of 7500 l/s at 1/7th scale.

All model recirculation flows can be set and continuously monitored using orifice plate flow meters, which were installed on each model pump recirculation line. The orifice plate sizes were determined as a function of flow and model scale and where possible, we would always attempt to run the orifice plate within the central region of its curve, to provide maximum accuracy. The model was equipped with three 150 mm pipe diameter orifice plate flow meters.

Each of the orifice plate flow meters were calibrated internally by in-house volume displacement testing and measured in line with BS EN ISO 5167-1: 2003. An orifice plate of each size was then re-calibrated via the use of an externally calibrated ultrasonic flow meter. Prior to installation on the test rig, the orifice plates were first checked visually for damage. An accurate K value was previously determined for each individual orifice plate and this value was used to produce flow data that could accurately determine the passing flow for variations in recorded deflection, measured on the model manometers. As such, a range of model flows could be set that accurately represented the full range of prototype flows, at the given scale.

The maximum required flow rate through the Drop Shaft is defined by CEC as 7500 l/s (1 in 1000 year). Based on a model scale of 1:7 the equivalent flow rate at scale is 57.7 l/s. The orifice plates were sufficiently sized for the defined flow rates and provided a suitable deflection within the central region of the orifice plate range.

The orifice plates used on the Bury drop structure MH6 model are detailed in Table 1.04.

Table 1.04 Bury drop structure MH6 Orifice plate designations		
Flow line designation	OP reference number	OP Pipe diameter
Pump 1	150/16	150 mm
Pump 2	150/23	150 mm
Pump 3	150/24	150 mm

Figure 1.05 shows a typical calibration curve for a 150 mm orifice plate utilised on the Bury Drop Structure MH6 model test rig.

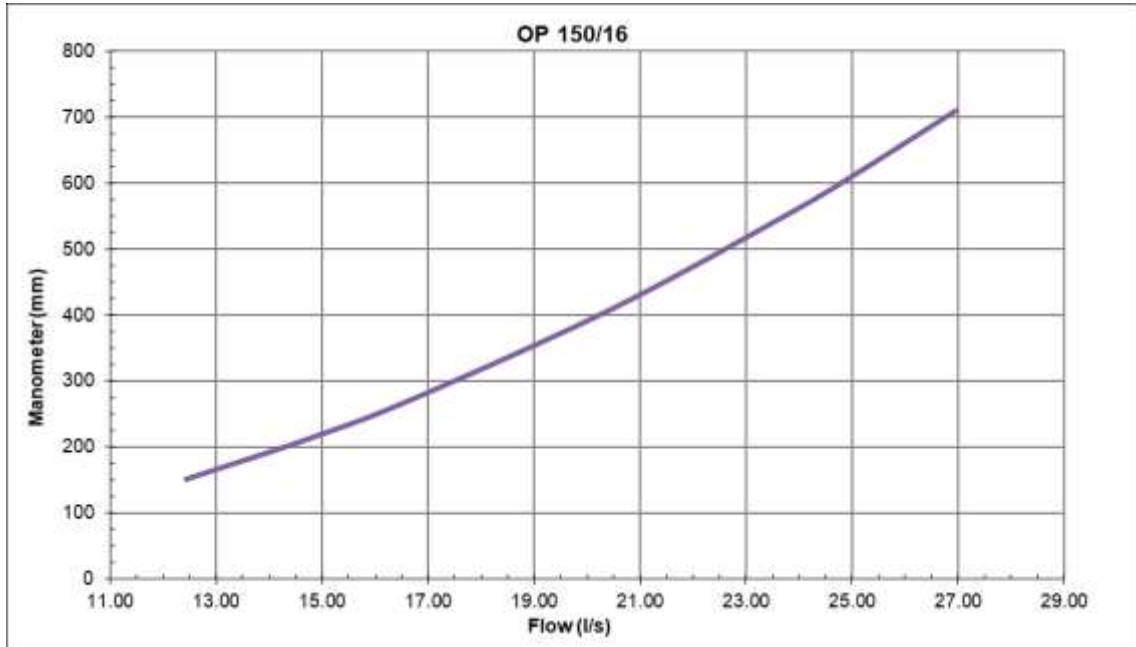


Figure 1.05 – Orifice plate calibration curve for OP Reference 150/16

1.7 Representation of solids

It is important that the Bury Drop Structure MH6 is relatively self-cleansing. To study the behaviour of grits and solids on a physical model, grit particles of varying settling velocity, sawdust and plastic particles were introduced into the model. Sawdust acts as an emulsion and also a binding agent to provide a good indicator of composite grease and gross solid type material. In addition, fibrous material including cloth, sponge and gauze is also added to ensure the full anticipated range of particles are assessed. The solids ranged from positive to negative buoyancy, the scale considerations in the modelling of the grit and floating material are discussed in Appendix A1 of this document.

It is important to represent the solids fraction of surface water and storm water in order to highlight potential problems of grit settlement and accumulation. Various methods of representing grit particles may be employed, in HCL experience grit transport and settlement can be successfully simulated using graded samples of plastic particles. Care must be taken to ensure the particles are of uniform density and free from air (hence, there must be an absence of any air voids within the particles). It is not possible to linearly scale a grit particle within a reduced scale model as this would result in particles that are too small. This can be overcome by using plastic particles with a lower settling velocity (SG) than the full scale grit particles and by determining the equivalent particle size on the basis of settling velocity.

The equivalent full scale size of the model grit particles is determined on the basis of the settling velocity of a typical sewage sample. In accordance with Froude Law affinity, the model particle settling velocity is multiplied by the square root of the model scale to determine the full scale settling velocity and hence, the equivalent particle size. Figure 1.06 demonstrates the settlement curve and equivalent particle size for the model debris at a scale of 1/7.

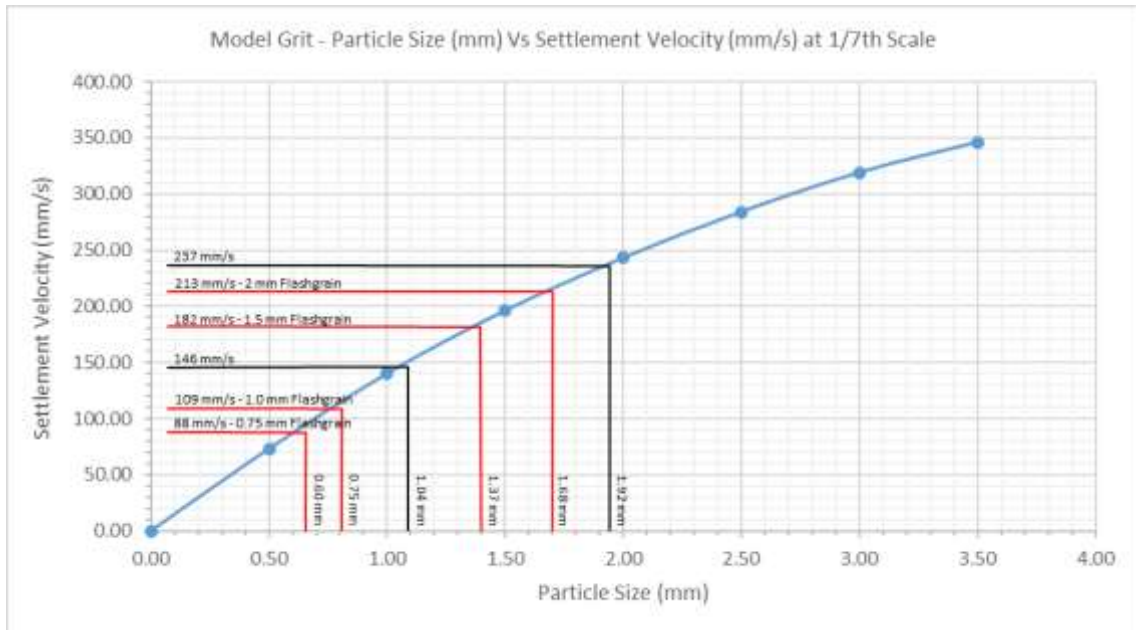


Figure 1.06 - Grit settlement curve for 1/7 scale model grit particles.

In terms of neutral and positive buoyancy debris, representing fibrous material, fat, oil and grease (FOG) is more difficult to accurately simulate, given the cohesive nature of the solids. Indicative sampling is utilised on the model using varying grades of plastic shavings, saw dust (which acts as an agglomerate to bind the debris together), varying types of cut rag type material, sponge, and shredded plastic film. When well mixed, this sample has proved effective in simulating the accumulation and transportation of a full scale floating and neutrally buoyant debris sample, with accuracy. As the volume of this type of sample is unspecified, a large volume was assumed for the purposes of the modelling exercise and was delivered into the model via the dedicated upstream inlet pipe.

Image 1.05 illustrates the range of solids samples utilised for testing, inclusive of both settling solids and floating debris.



Image 1.05 - Typical representation of modelled solids introduced into the Bury drop structure MH6 model

2.0 INITIAL ASSESSMENT

2.1 Introduction

Prior to confirmation of the tailwater levels from the Client, the physical hydraulic model of the Bury Drop structure MH6 was initially tested at a range of flow rates with a free-discharge maintained at the downstream end of the model. Hydraulic assessment of the drop structure investigated the ability of the structure to pass the required flows, together with an assessment of the flow conditions within the drop structure.

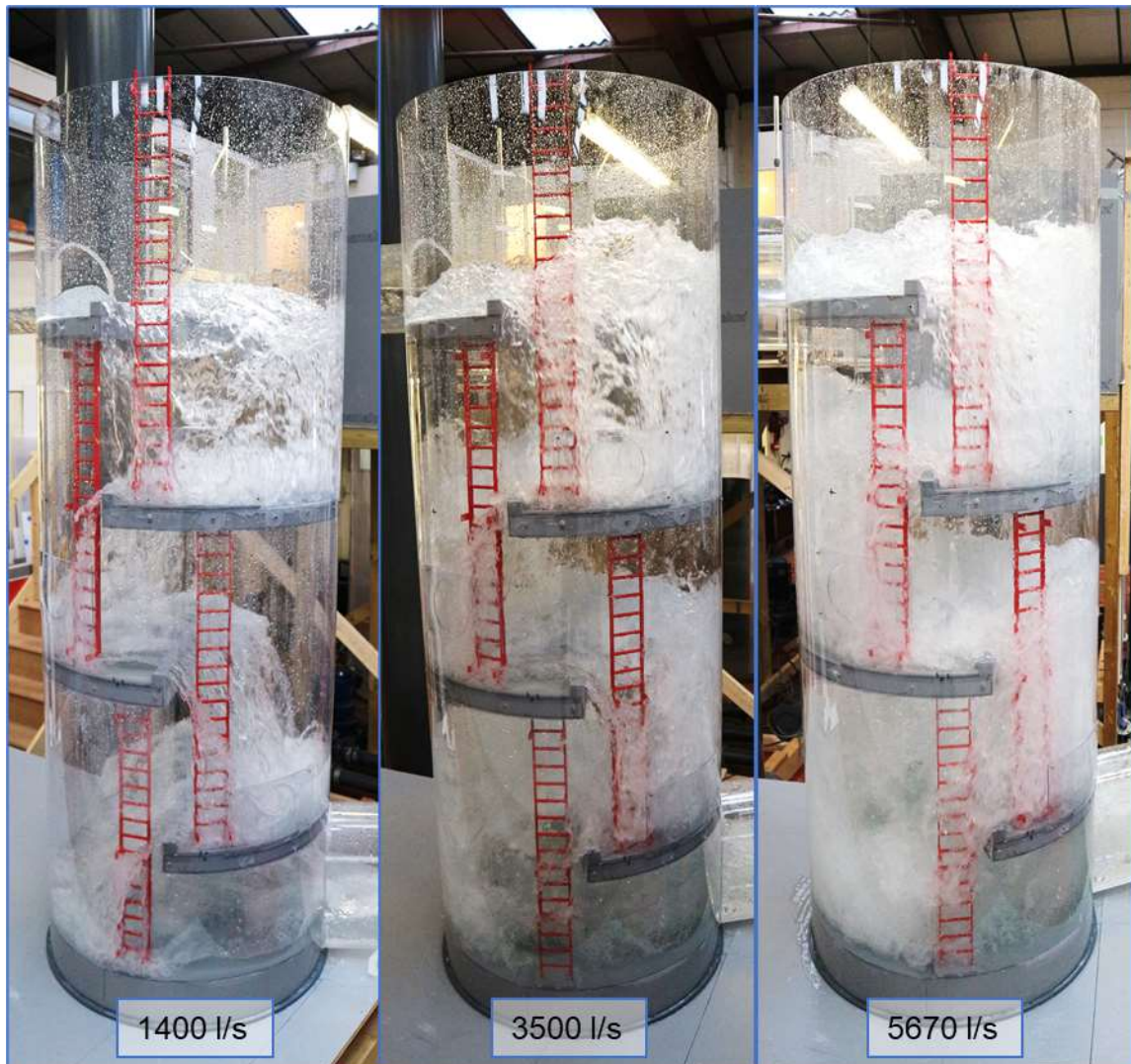
The drop structure was initially tested at flow rates of 1400 l/s, 3500 l/s and 5670 l/s (i.e. return periods of 2 years, 50 years and 100 years + 35% respectively), in order to provide a broad assessment of flow conditions for the initially proposed drop structure arrangement. During this testing minor performance issues were identified. Therefore, modifications were made to the proposed drop structure in order to improve performance prior to further, more detailed testing. The findings of this initial assessment of the Bury Drop Structure MH6 are detailed in this section.

2.2 Hydraulic Assessment

Image 2.01 shows the initially proposed arrangement of the drop structure operating at the three flow rates of 1400 l/s, 3500 l/s and 5670 l/s, which were investigated for the initial assessment.

Flow exiting the inlet culvert traversed the inlet platform (Level 1) and cascaded off the platform edge to form a free falling jet. The free fall jet impinged on the receiving surface of each intermediate platform and then discharged off the platform edge with a free fall jet before reaching the base of the shaft and exiting through the outlet culvert. Significant volumes of air were entrained into the flow, with more air entrainment occurring at the higher flow rates. This air entrainment is to be expected in cascades and is a feature and function of the cascade design, it does not pose any problems to the performance of the drop structure.

As the flow rate increased, the velocity and turbulence of the flow increased significantly, however, under operation up to the peak flow rate of 5670 l/s the drop structure was able to safely convey flow forward to the outlet culvert at the specified height of drop.



**Image 2.01 – Flow conditions within the Bury Drop Structure MH6,
Initial review flow rates of 1400 l/s, 3500 l/s and 5670 l/s**

Image 2.02 shows the Level 1 platform at each of the flow rates investigated. In each case, flow enters the drop structure via the inlet culvert and pools behind the platform ledge, before cascading onto the platform below. At the higher flow rates, the incoming flow is deeper, so the platform ledge intercepts a smaller proportion of the flow. As the flow cascades onto the ledge below, it impacts on the far wall of the chamber. At an inflow of 1400 l/s, only a small proportion of the flow impacts the opposing wall of the shaft. As the flow rate increases, the entire cascade jet impacts the opposing wall of the shaft at a high velocity.



Image 2.02 – Flow conditions at Level 1 platform at a range of flow rates

Image 2.03 shows the flow conditions at Level 3 for the three flow rates investigated. For all flow rates, the conditions at Level 3 were similar to those at Levels 2 and 4. Flow cascaded onto the Level 3 platform, where it pooled behind the platform ledge and cascades onto the platform below. Similarly to Level 1, at the higher flow rates, the ledge intercepted less of the flow. This resulted in high energy flow cascading onto the receiving platform below. As the flow rate increased, the volume of water increased, as did air entrainment. This resulted in the flow utilising a larger volume of the cascade structure at each level, as the flow rate increased.

At an inflow of 1400 l/s, the cascade free fall jet from Level 2 to Level 3 did not impinge on the far wall of the shaft, which is dissimilar to Level 1. This is because the flow had a lower horizontal velocity component than at Level 1, due to the cascade platform offset causing the flow to change direction. At the higher flow rates, the free fall jet impinged against the opposing internal wall of the shaft.

At each platform level, and for all flow rates, the flow cascaded from each platform with a continuous nappe formation, which sprung along the full length of the platform ledge. The continuous nappe formation prevented the free venting of air through each landing level, which may act to pressurise the shaft at a particular level and cause structural damage to the shaft. In order to prevent this condition, the continuous nappe formation should be broken up and dispersed at each platform level so that air is allowed to freely vent to atmosphere within the shaft.

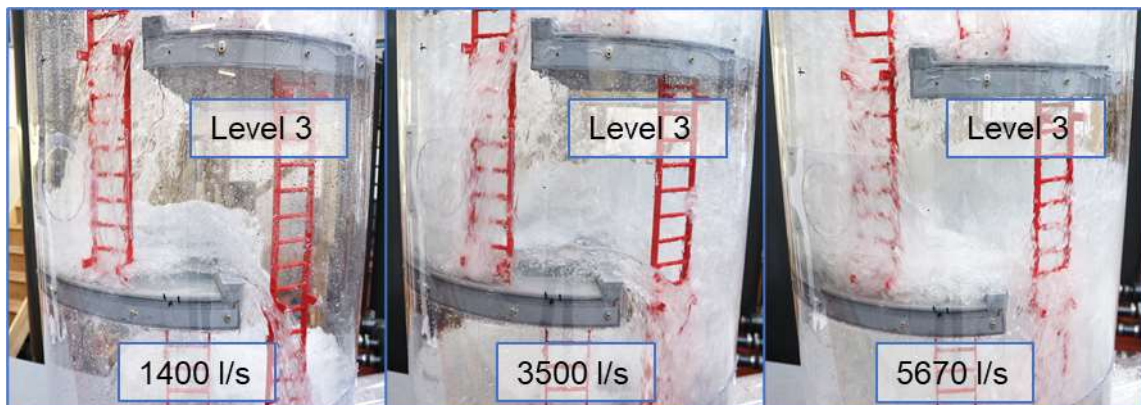


Image 2.03 – Flow conditions at Level 3 at a range of flow rates

2.3 Conclusions from Initial Testing

Initial testing found that the Bury Drop Structure MH6 generally performed satisfactorily for the flow rates investigated, however, minor performance issues were identified which should be addressed. A continuous nappe formed at each cascade, which may prevent air from freely venting to atmosphere within the shaft. Trapping air has the potential to locally pressurise the shaft, which may cause structural damage. The continuous nappe formation should be broken up and dispersed at each platform level so that air is allowed to freely vent to atmosphere within the shaft.

The proposed platform ledges acted to pool water on the platform which dissipated energy at each platform, however, at higher flow rates, a smaller proportion of the receiving flow was intercepted. This resulted in high energy flows cascading onto the platform below. The energy of these nappes formed at each platform of the cascade should be reduced, in so far as can reasonably be achieved.

The next Section of this report describes the modifications which are proposed to the Bury Drop Structure MH6 in order to address the performance issues identified by this phase of initial assessment.

3.0 DEVELOPMENT

3.1 Introduction

Following completion of the initial testing programme on the Bury Drop Structure MH6 arrangement, a number of adverse hydraulic features were highlighted, which could compromise performance, these comprised:

- A continuous nappe formed at each cascade, which prevented air from venting within the shaft. This may act to locally pressurise the chamber, which can cause structural damage.
- The platform ledges, which act to pool water and dissipate energy, intercept a smaller proportion of the flow at higher flow rates, which reduces the energy dissipation at each platform.

A programme of modifications was proposed, which were intended to mitigate these performance concerns.

3.2 Modifications

Flow deflection columns – At each platform, a flow deflection column was installed at the downstream ledge of each platform, to one side of the shaft, adjacent to the shaft wall. The purpose of the flow deflection columns was to deflect nappe flow away from the shaft wall, creating an air gap through which the entire chamber could be vented to atmosphere. The flow deflection columns were also intended to direct high velocity flow away from the shaft man access ladders located at each platform, in so far as was possible.

Platform ledge height – The height of each platform ledge was increased by 50 % from 200 mm to 300 mm. The increase in ledge height was intended to intercept more of the flow, which will increase the depth of pooled water retained at each platform. The increase in the retained water depth will dissipate a larger proportion of the energy of the receiving flow. It should be noted that, although the increase in ledge height will dissipate more energy, the flows through the drop structure are inherently high energy, especially at the higher flow rates. Therefore, although energy dissipation will be increased, the flows will still exhibit significantly high energy and turbulence.

Figures 3.01 and 3.02 provide details of the proposed modifications, full details are provided in Drawings 05 and 06 in Appendix 2.0.

Image 3.01 show a view of the proposed modifications installed at one of the platforms within the physical hydraulic model. Note that the installation is identical at each platform.



Figure 3.01 – Proposed modifications to Bury drop structure MH6 – Sections

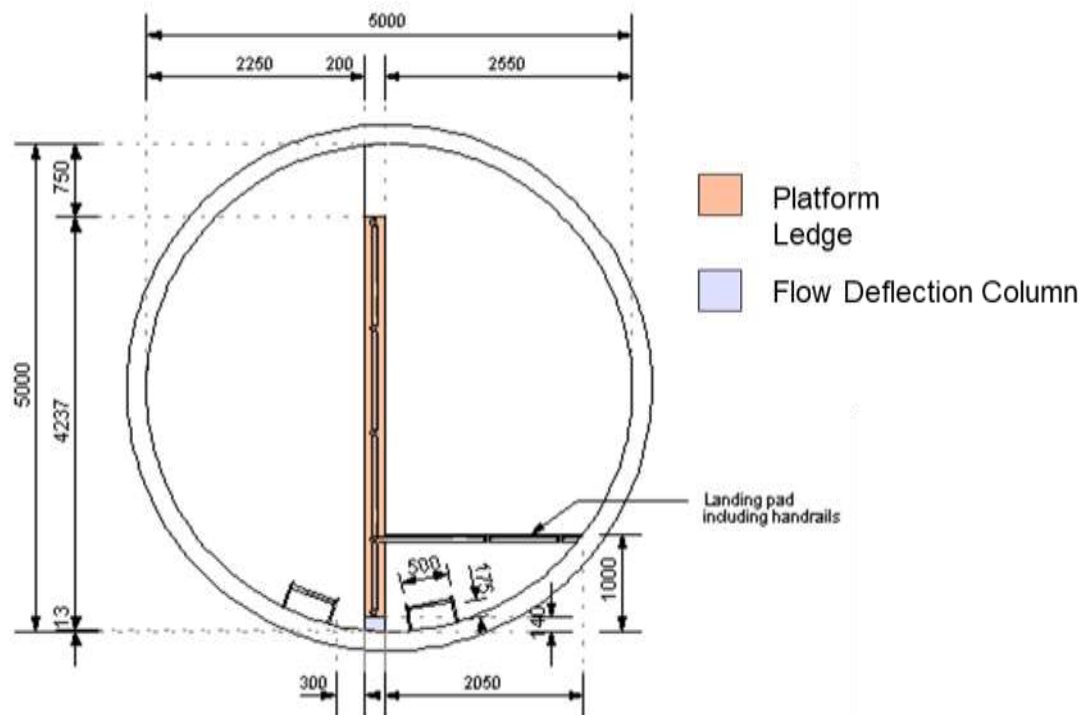


Figure 3.02 – Proposed modifications to the drop structure, plan of generic platform



Image 3.01 – Proposed modifications installed in the physical hydraulic model

4.0 FINAL TESTING

4.1 Introduction

The modified Bury Drop Structure MH6 arrangement was investigated at the flow rates and tailwater levels listed in Table 1.03. As detailed in initial testing, three distinct flow conditions were observed, depending on the tailwater level within the shaft, however, the drop structure was found to perform satisfactorily for all operating conditions investigated. Solids assessment found that solids were cleared from the drop structure for the investigated operating conditions. The findings of this investigation are described in the following sections of this report.

4.2 Hydraulic Assessment

The Bury Drop Structure MH6 was first tested at specified flow rates of 7140 l/s and 7500 l/s, with tailwater levels of 136.016 mAD and 136.140 mAD, representing return periods of 100 years + 70% and 1000 years, respectively. The flow conditions were appreciably similar in each case, due to the similar flow rates and water levels, as can be seen in Image 4.01. In each case the shaft was almost completely full of water, due to the high tailwater levels.

Flow entered the drop structure via the Inlet Culvert and was conveyed down the full height of the cascade following interception and re-direction by each of the successive platforms before exiting the shaft via the Outlet Culvert. The flow direction was altered at each successive platform level, causing recirculation regions and turbulence, which aided in energy dissipation. The flow velocities were generally high, due to the high flow rates, however, the drop structure was able to convey the flows safely.

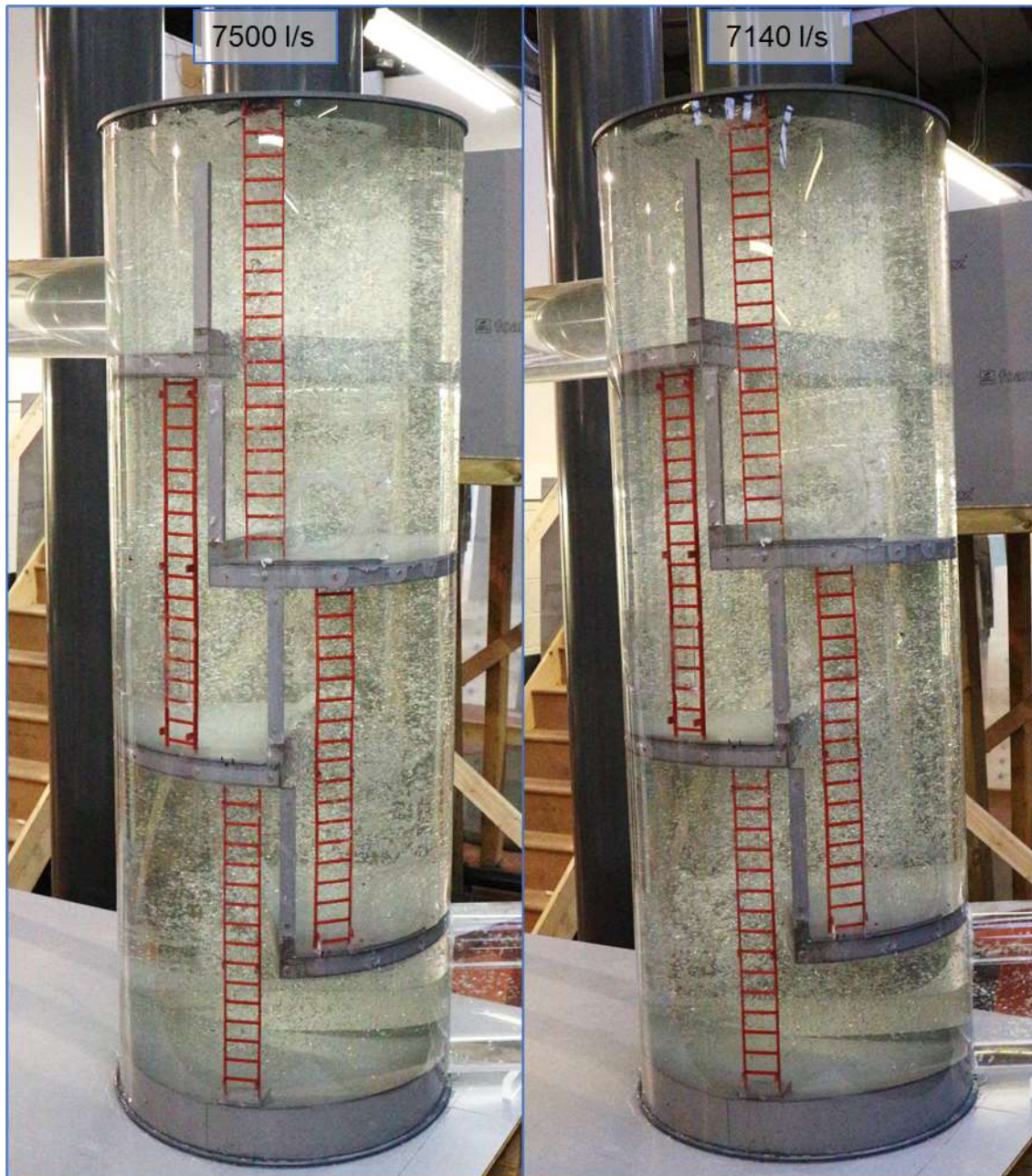


Image 4.01 – Flow conditions within the Bury Drop Structure MH6 at flow rates of 7500 l/s and 7140 l/s

Image 4.02 shows the flow conditions at platform Level 1, at a flow rate of 7500 l/s, with the flow paths indicated in the image. Note that the flow paths indicated were comparable to those at 7140 l/s. Flow entered the drop structure via the Inlet Culvert and travelled across the chamber, where it was deflected by the far wall. A certain proportion of the flow was deflected upwards towards the soffit of the chamber. This instigated a recirculation region close to the top of the chamber. As flow passed across the soffit, air was entrained into the flow from the vent located in the model soffit plate. This indicates that a vented soffit must also be installed in the full scale Bury Drop Structure MH6, as failure to do so could result in the chamber becoming depressurised, which may lead to structural damage or failure. The recirculation path resulted in a slightly higher water level at the side of the chamber opposing the inlet. Entrained air was conveyed through the drop structure and into the Outlet Culvert.

Flow passing along the invert of the platform was deflected upwards slightly, by the platform ledge, before re-orientation downwards towards the lower platform levels. At the downstream side of the platform ledge, a low velocity region occurred. This allowed some air bubbles to rise in this region. These air bubbles were then intercepted by the higher velocity flow and transported down towards the lower platform levels.

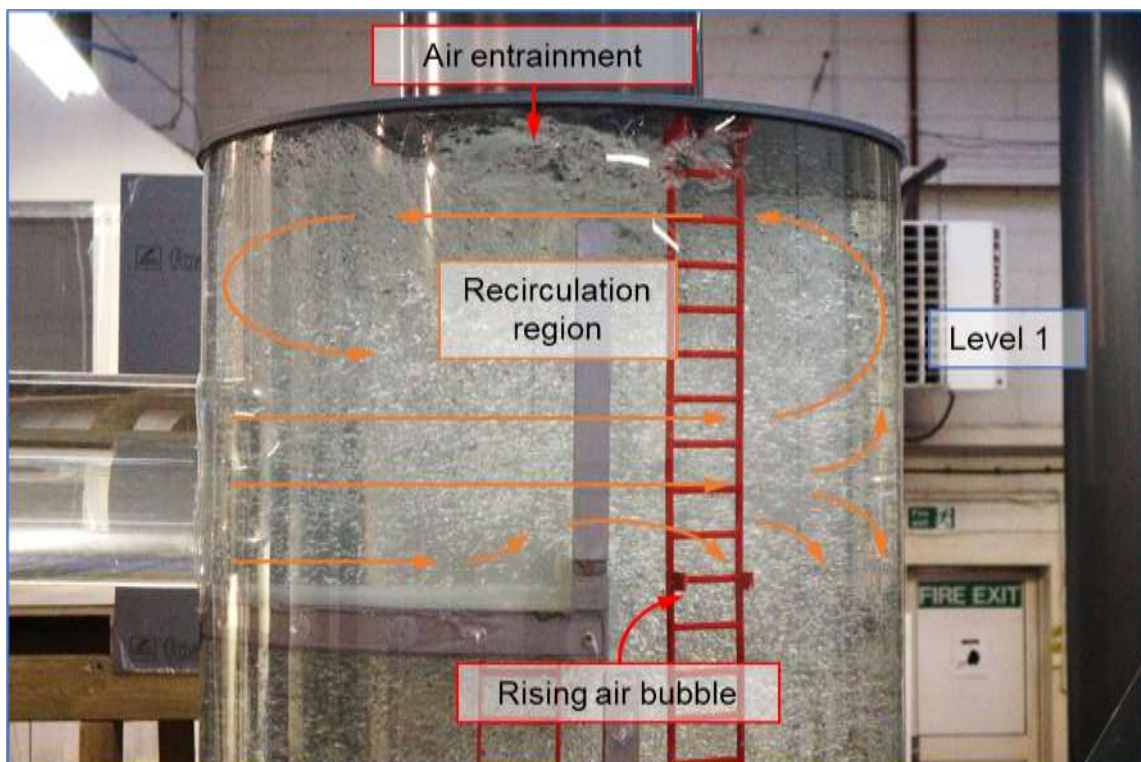


Image 4.02 – Flow conditions at Level 1 at a flow rate of 7500 l/s

Image 4.03 shows the flow conditions at Levels 2 and 3 at a flow rate of 7500 l/s. Again, the flow conditions are comparable to an inflow of 7140 l/s. Flow from Level 1 travelled down until it was intercepted by the platform at Level 2, where it was deflected towards the centre of the chamber. Flow was deflected over the platform ledge and travelled across the chamber in a relatively high velocity, narrow concentrated jet. Upon impacting the far wall of the shaft, the jet was deflected both upwards and downwards, forming two recirculation regions, which circulated in opposite directions to one another. In the topmost region of Level 2, beneath the Level 1 platform, a recirculation region occurred which occupied a relatively large depth. Opposite to the platform at Level 2, a smaller recirculation region occurred, which occupied a smaller depth. A certain proportion of the flow was deflected down, towards Level 3. At Levels 3 and 4, the flow conditions were similar to those at Level 2.

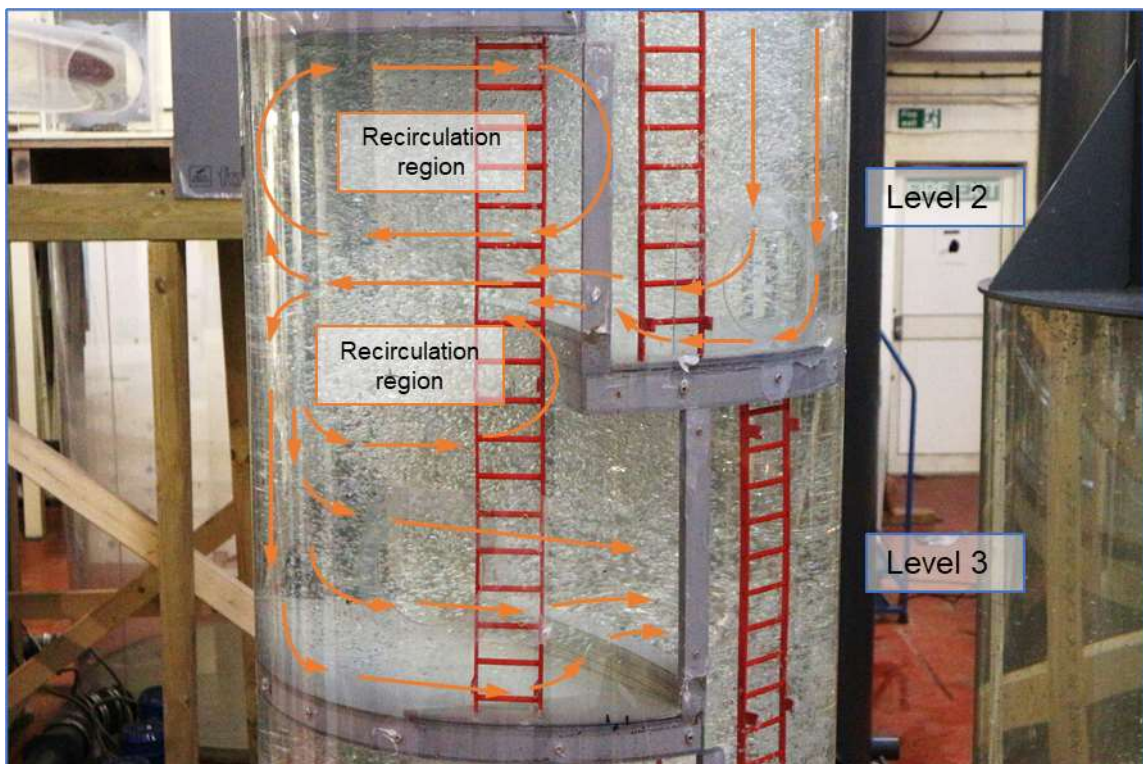


Image 4.03 – Flow conditions at Levels 2 and 3 at a flow rate of 7500 l/s

Image 4.04 shows the flow conditions at Level 5 at a flow rate of 7500 l/s. Similarly to conditions at the higher levels, a recirculation region occurred opposite to the platform at Levels 4 and 5. Flow travelled down from the Level above and was drawn towards the Outlet Culvert, utilising the majority of the flow depth at Level 5. Image 4.06 shows the truncated Outlet Culvert at a flow rate of 7500 l/s. The flow utilised the entire depth of the Outlet Culvert, with air bubbles distributed reasonably evenly through the flow depth.

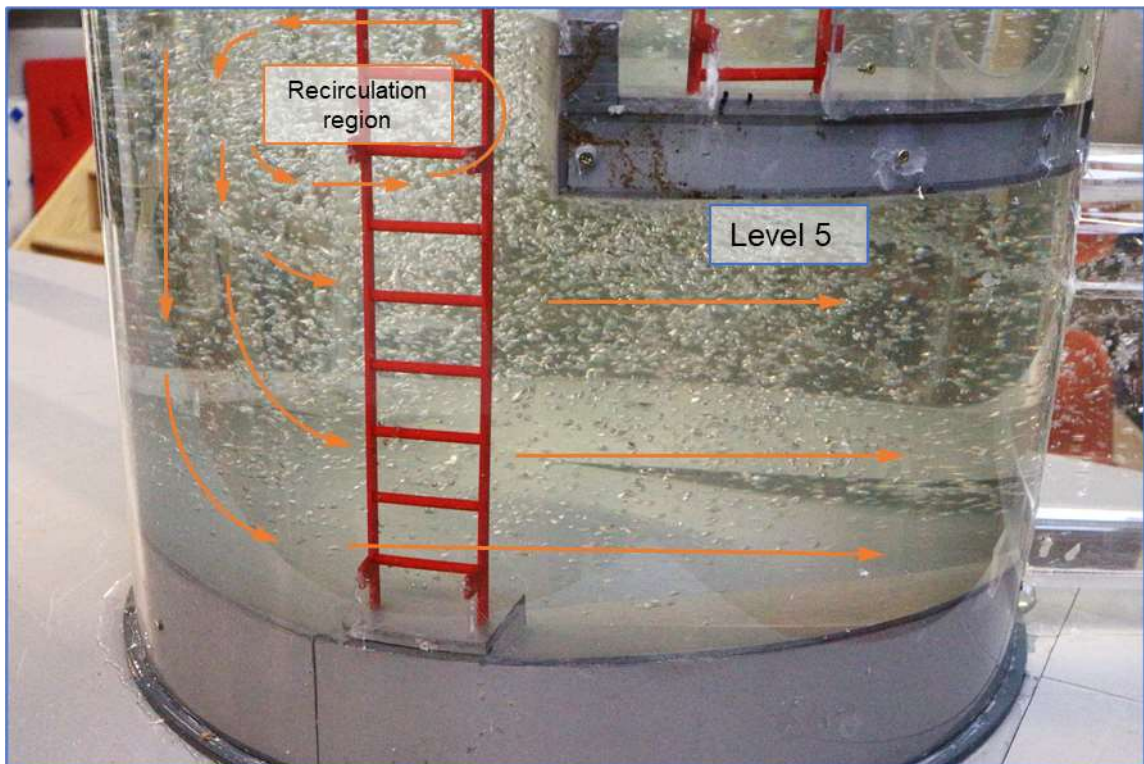


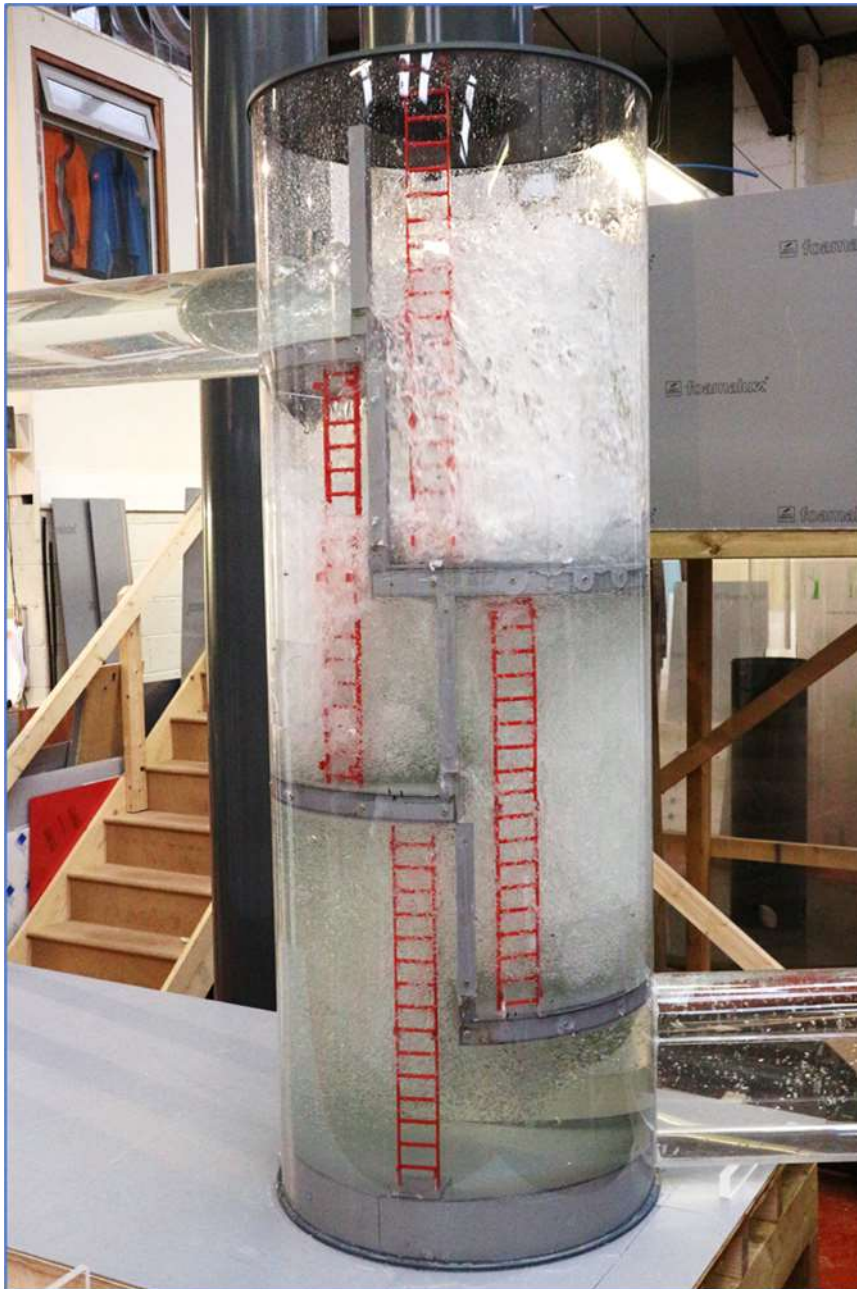
Image 4.05 – Flow conditions at Level 5 at a flow rate of 7500 l/s



Image 4.06 – Flow conditions within the truncated outlet Culvert at 7500 l/s

The Bury Drop Structure MH6 was next tested at a specified flow rate of 5670 l/s, with a tailwater level of 129.326 mAD, representing a return period of 100 years + 35%. The main difference between the flow conditions in this operating scenario and those discussed previously is that the water level was at approximately the mid depth of the shaft. This resulted in free-surface flow conditions in the upper section of the drop structure and submerged flow conditions in the lower section of the drop structure, as shown in Image 4.07. The free-surface flow conditions in the upper section of the drop structure caused significantly more air entrainment to occur than was observed at the higher tailwater levels.

Image 4.08 shows the flow conditions at Level 1. Due to the relatively high flow rate, the Inlet Culvert was full surcharged and operated at full depth. As flow entered the chamber, the flow depth decreased slightly, as the flow spread across the width of the platform. A proportion of air was entrained into the flow in this region due to turbulence at the free-surface. The increased height of the ledge at this level acted to intercept more of the flow, as would be expected, creating a deeper pool of retained water behind the ledge, which increased energy dissipation of the flow at this Level. Flow deflected over the ledge, where it cascaded into Level 2, below. The cascade was highly turbulent, resulting in significant air entrainment. The high turbulence of the flow also caused the cascade to break up, allowing air to be freely vented to atmosphere. The flow deflection column also deflected flow away from the shaft wall, allowing air to freely vent to atmosphere.



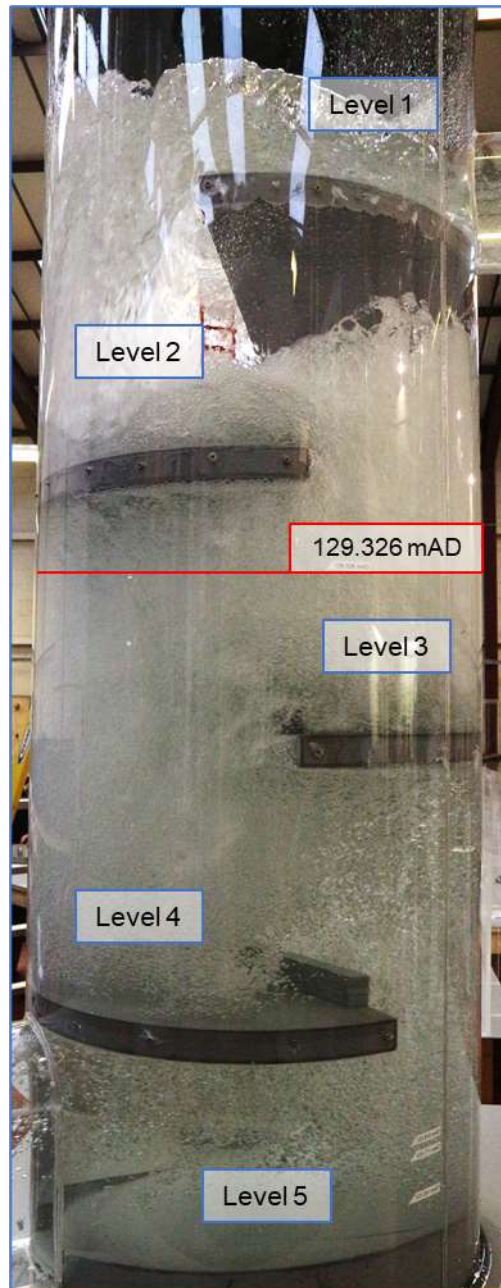
**Image 4.07 – Flow conditions within the Bury Drop Structure MH6
at a flow rates of 5670 l/s**



Image 4.08 – Flow conditions at Level 1 at a flow rate of 5670 l/s

Image 4.09 shows the flow conditions throughout the drop structure. As flow cascaded from Level 1, it impacted the Level 2 platform with high levels of energy and turbulence, entraining a significant volume of air into the flow. Again, the increased height of the platform ledge acted to pool a larger depth of retained water, which cushioned the impact of the cascading flow onto the receiving platform and increased energy dissipation.

The tailwater level of 129.326 mAD was between the Level 2 and 3 platforms, as indicated in Image 4.09. However, due to the high flow rate, the water level within the drop structure was effectively directly beneath the Level 2 platform. Flow at Level 2 was deflected over the platform ledge to Level 3. As Level 3 was submerged, the cascading flow from Level 2 was cushioned, dissipating energy. At Levels 3, 4 and 5, the flow patterns were comparable to the higher tailwater levels, as submerged flow occurred. The only significant differences in the flow conditions to those observed previously was a significantly increase in air entrainment, due to the free-surface flow conditions produced in the upper section of the drop structure. Flow velocities throughout the shaft were also slightly lower, due to the lower inflow rate.



**Image 4.09 – Flow conditions throughout the Bury Drop Structure MH6
at a flow rate 5670 l/s**

Image 4.10 shows the truncated model section of the Outlet Culvert under operation at a flow rate of 5670 l/s. The flow conditions within the Outlet Culvert were slightly different to those observed at the higher flow rates. Significantly more air was entrained into the flow at 5670 l/s and the lower flow velocity allowed more of the entrained air bubbles to rise to the topmost region of Level 5 (beneath the Level 4 platform), prior to exiting the drop structure via the Outlet Culvert. This circulation mechanism resulted in the forward transport of air within an air pocket located at the soffit of the Outlet Culvert, as well as entrained air bubble transport through the cross section of the culvert.

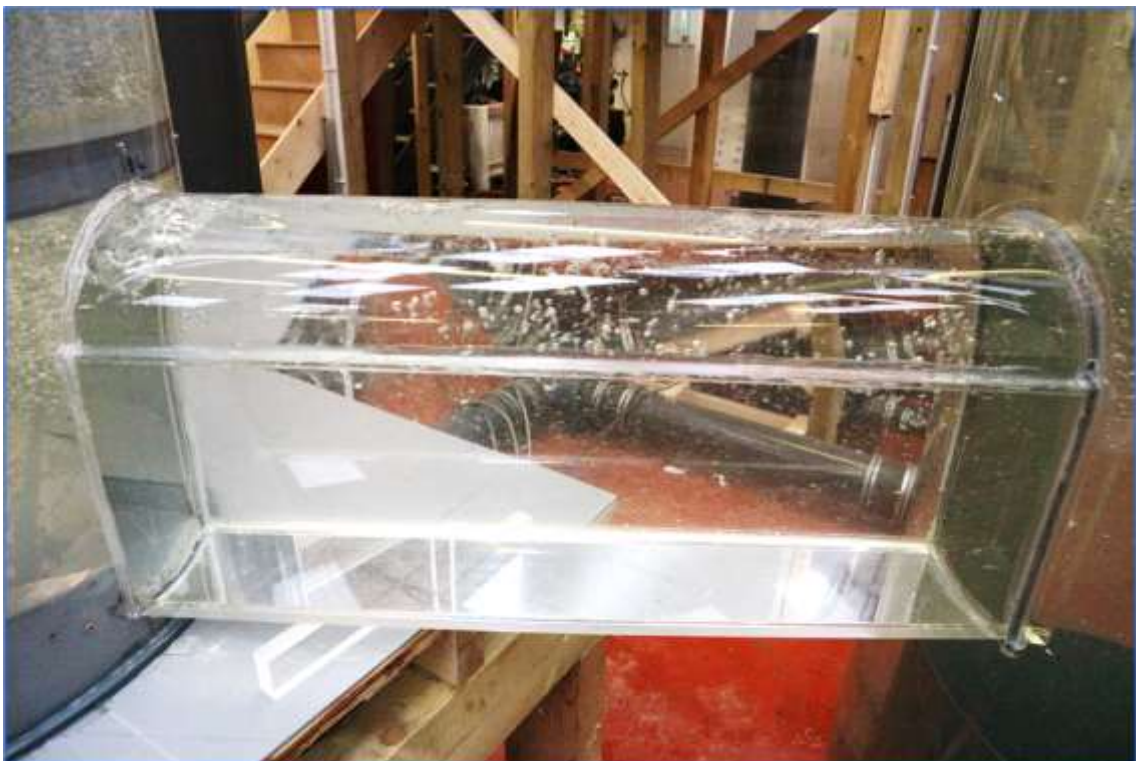


Image 4.10 – Flow conditions and air bubble transport within the Outlet Culvert at a flow rate of 5670 l/s

A continuous forward passage of entrained air was observed within the model outlet culvert. The air bubble formation at the Outlet Culvert soffit was maintained as an elongated string/narrow pocket type form and did not exhibit any appreciable growth over time due to continuous scour and passage forward within the main body of flow through the culvert.

The Bury Drop Structure MH6 was next tested at the three lowest flow rates of 1400 l/s, 2800 l/s and 3500 l/s, with tailwater levels of 123.391 mAD, 123.771 mAD and 123.910 mAD, representing return periods of 2 years, 20 years and 50 years respectively. The flow conditions at each of these operating scenarios were similar to those previously reported, in that the tailwater levels were close to the base of the drop structure, as such free-surface, cascading flow conditions were present at each platform level, as can be seen in Image 4.11.

Image 4.12 shows the flow conditions present within the shaft at Level 1 for each flow rate. Similarly to at a flow rate of 5670 l/s, flow entered the drop structure via the Inlet Culvert and pooled behind the platform ledge. Contrary to operation at 5670 l/s, however, the Inlet Culvert did not run full, due to the lower flow rates, so the incoming flow was at a lower depth and had a lower inlet velocity. Again, the proposed increase in the platform ledge height acted to pool a larger depth of retained water, increasing energy dissipation.

Flow entered the drop structure via the Inlet Culvert and was conveyed down the full height of the cascade following interception and re-direction by each of the successive platforms before exiting the shaft via the Outlet Culvert. Flow passed across each of the platforms and cascaded over the ledge onto the platform with a defined jet formation. At 1400 l/s, the cascading jet did not impinge on the opposing curved internal wall of the shaft. At 2800 l/s and 3500 l/s, however, the flow velocities were higher, so the cascading jet did impinge on the opposing wall.

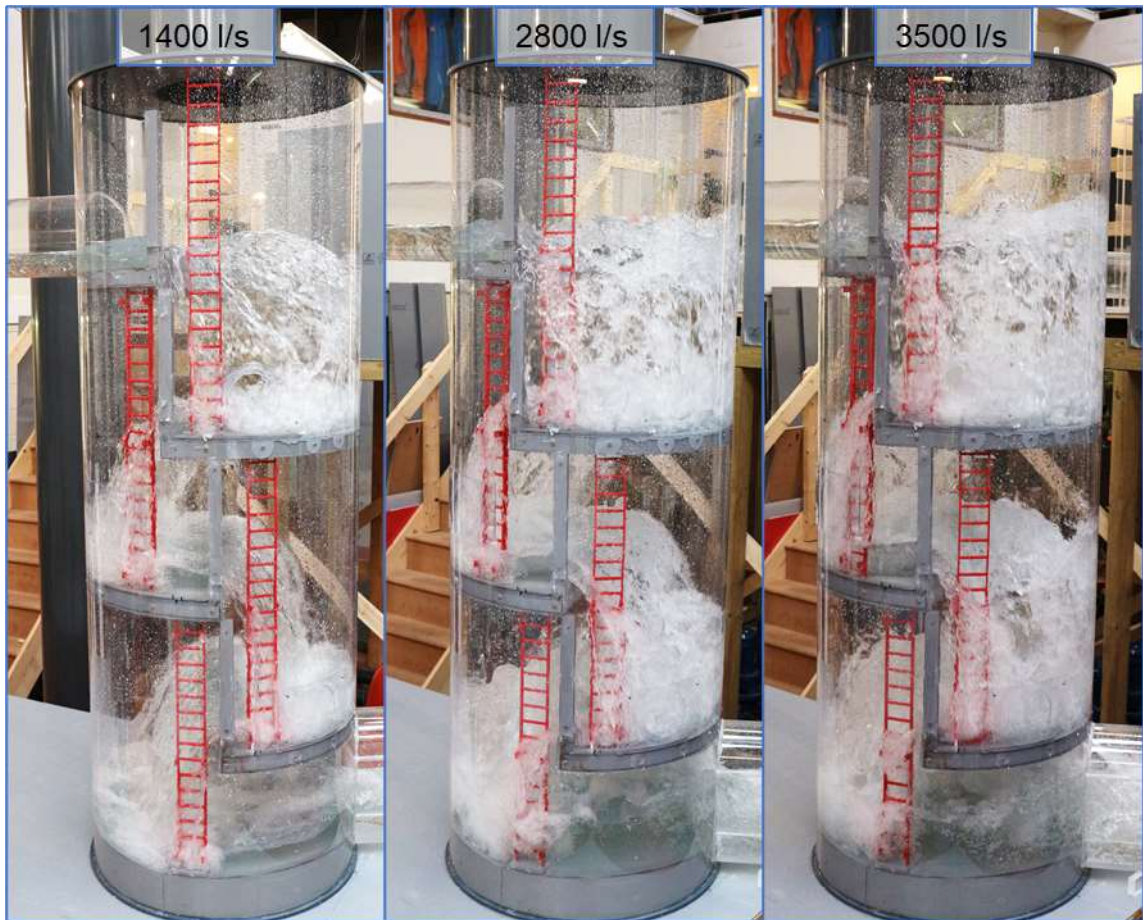


Image 4.11 – Flow conditions within the Bury Drop Structure MH6 at flow rates of 1400 l/s, 2800 l/s and 3500 l/s



Image 4.12 – Flow conditions at Level 1 of Bury Drop Structure MH6 at flow rates of 1400 l/s, 2800 l/s and 3500 l/s

Image 4.13 shows the flow conditions at Levels 2 and 3. As the cascading flow impacted on the receiving platform below, significant air entrainment occurred. Again, the flow pooled behind the platform ledge. The increased ledge height increased the depth of retained flow on each platform, increasing the energy dissipation at each platform. The flow then cascaded over the ledge, onto the platform below. These flow conditions were comparable at Levels 2, 3 and 4 at each flow rate.

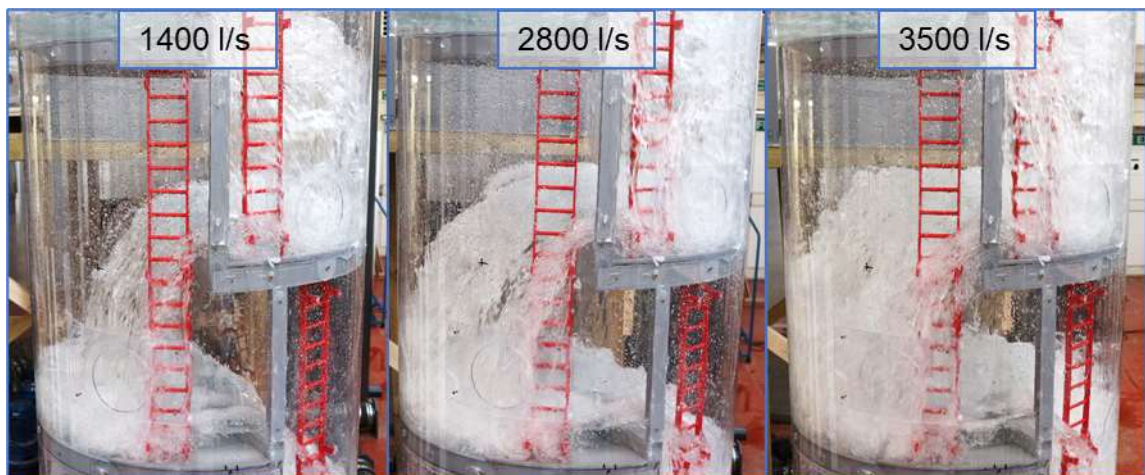


Image 4.13 – Flow conditions at Levels 2 and 3 of Bury Drop Structure MH6 at flow rates of 1400 l/s, 2800 l/s and 3500 l/s

Image 4.14 shows the influence of the flow deflection column at Level 3 at a flow rate of 1400 l/s. It can be seen that the column deflected the flow away from the shaft wall, creating an air gap between the wall and the cascading nappe. This air gap was sufficient to allow air to freely vent to atmosphere within the shaft. The flow deflection columns also directed higher velocity components of flow away from the shaft man access ladders, which were located to one side of each platform. This should reduce potential ragging of solids to the access ladders and protect them from excessive hydraulic loading.

The effect of the flow deflection columns on the flow path produced at the discharge from each platform ledge was comparable for all intermediate platforms within the shaft at flow rates at which cascading free-surface flow was present.



Image 4.14 – Flow deflection column effects at Level 3 at a flow rate of 1400 l/s

Image 4.15 shows conditions within the shaft at platform Level 5 at each of the three flow rates tested. As shown, the tailwater levels within the shaft are slightly higher than the base of the drop structure for each flow rate displayed in the image and the tailwater levels increased incrementally with flow rate. The tailwater levels at these flow rates create a pool of water at Level 5, which cushioned the cascading jet produced from Level 4 and dissipated the jet energy. Flow was then re-directed towards the Outlet Culvert, with a proportion of the entrained air vented to the free surface within the shaft prior to entry to the Outlet Culvert.



Image 4.15 – Flow conditions at Level 5 at flow rates of 1400 l/s, 2800 l/s and 3500 l/s

Image 4.16 shows the flow conditions within the truncated Outlet Culvert at 1400 l/s, 2800 l/s and 3500 l/s. In all cases, a free-surface existed within the culvert. At 1400 l/s any entrained air was evacuated from the flow at the free-surface over a relatively short distance. At 2800 l/s and 3500 l/s, entrained air bubbles remained in the flow over the length of the truncated Outlet Culvert. In the prototype culvert, these entrained air bubbles would be evacuated from the flow a short distance downstream of the downstream extent of the physical hydraulic model.



Image 4.16 – Flow conditions within the Outlet Culvert at flow rates of 1400 l/s, 2800 l/s and 3500 l/s

Conclusions from Hydraulic Assessment

Hydraulic assessment of the Bury Drop Structure MH6 physical hydraulic model determined that the proposed drop structure conveyed the flows safely and performed satisfactorily for all flow rates and tailwater levels investigated. Three different general flow conditions occurred within the drop structure, depending on the water level:

1. At the higher water levels (100 years + 70% and 1000 year return periods), the drop structure was almost completely surcharged, i.e. up towards the shaft soffit. As flow passed across the shaft soffit at surcharged operation, air was entrained into the flow from the vent provided in the model soffit. This indicates that a vented soffit must also be installed in the prototype drop structure, as failure to do so may result in the chamber becoming depressurised, which could lead to structural damage and possible accelerated failure.
Submerged flow conditions occurred throughout the drop structure at the specified flow range of 100 years + 70% and above. At these conditions, flow paths were dictated by the arrangement of the platforms and the flow was conveyed through the drop structure and into the Outlet Culvert safely.
2. At a return period of 100 years + 35%, the water level within the drop structure was approximately at mid shaft height. In the upper section of the drop structure free-surface flow occurred and the flow cascaded from each platform to the next. This regime was characterised by high levels of air entrainment at each platform. The proposed increase in the height of the platform ledges increased the depth of the retained flow on each platform, which in turn increased energy dissipation. The proposed vertical flow deflection columns at each platform ledge acted to deflect flow away from the wall of the drop structure, allowing air to freely vent to atmosphere. The flow deflection columns also deflected flow away from the man access ladders somewhat. At the lower depths, submerged flow occurred and the flow patterns were comparable to those at the higher water levels, but with slightly lower velocities and a higher volume of entrained air.
3. At the lower tailwater levels (return periods of 2 years, 20 years and 50 years) the water levels within the drop structure were closer to the invert of the shaft. Under these conditions, flow was also conveyed safely through the drop structure. Free surface cascading flow occurred at each landing level within the drop structure, characterised by high levels of splash, spray and air entrainment.

Again, the proposed increased ledge heights increased the depth of retained water on the platforms, increasing energy dissipation, and the flow deflection columns diverted flow away from the wall of the chamber, creating an air gap through which air could be freely vented to atmosphere. The water levels at the base of the drop structure shaft, dictated by the receiving water course tailwater levels, also acted to cushion the flow and increase energy dissipation.

The detailed characteristics of operation of the proposed arrangement of Bury Drop Structure MH6 may be more readily observed within the video record accompanying this text report. Video record is provided of the flow conditions within the Bury Drop Structure MH6 for each operating scenario investigated.

4.3 Solids Assessment

Solids assessment of the proposed Bury Drop Structure MH6 was initially conducted at inflow rates of 7140 l/s and 7500 l/s. As with the hydraulic assessment, the solids handling at these two inflow rates were comparable, due to the similar flow rates and water levels. In these two operating scenarios, the Inlet Culvert was surcharged as the water level exceeded the soffit level of the Inlet Culvert. Hence model solids representing highly buoyant objects, such as footballs and plastic bottles, were not drawn through the Inlet Culvert into the drop structure. This is representative of the full scale structure as at inlet water levels significantly higher than the Inlet Culvert, highly buoyant materials would not be expected to be drawn into the drop structure.

In both inflow cases tested for the initial solids assessment all other solids were cleared from the drop structure relatively quickly. FOG, rag type material and settleable solids were all held in suspension and transported through the drop structure without deposition or build-up.

The detailed characteristics of solids transport within the proposed arrangement of Bury Drop Structure MH6 may be more readily observed within the video record accompanying this text report.

Images 4.17 – 4.19 demonstrate solids suspension and transport at various positions within the drop structure under operation at an inflow of 7500 l/s, which is also representative of 7140 l/s. It can be seen that all solids are in suspension and there is no deposition or build-up of any solids.



Image 4.17 – Solids in suspension at Level 1 at a flow rate of 7500 l/s



Image 4.18 – Solids in suspension at Level 4 at a flow rate of 7500 l/s



Image 4.19 – Solids in suspension at Level 5 at a flow rate of 7500 l/s

Solids handling was next assessed at a flow rate of 5670 l/s, at which the water level was at approximately the mid depth of the shaft. At this condition, the water level was sufficiently low for highly buoyant solids to also be drawn through the Inlet Culvert into the drop structure. All solids were cleared from the drop structure. In the upper section of the drop structure, cascading free-surface flow occurred. Under these conditions, solids were either suspended in the pooled water depth retained at each platform or tracked rapidly across the base of the platform, prior to being transported over the platform ledge to the receiving platform below. This can be seen in Image 4.20.

At the lower, surcharged, platforms within the shaft the solids were transported in a similar manner to those at the higher flow rates and tailwater levels and were rapidly transported through the drop structure, see Image 4.21. Some rag type material accumulated around the ladder positions, however, this was not deemed to be excessive and is unavoidable. The highly buoyant solids, behaved slightly differently to the other solids in the submerged flow region, as depicted in Image 4.22. Due to their high buoyancy, a proportion of these solids rose within the drop structure until they met the underside of a platform. They remained there temporarily until the highly turbulent, recirculating flow transported them back into the main flow path. All these highly buoyant solids were cleared from drop structure after a short duration of operation.



Image 4.20 – Absence of solids at Level 1 at a flow rate of 5670 l/s



Image 4.21 – Solids suspension at Level 5 at a flow rate of 5670 l/s



Image 4.22 – Highly buoyant solids at Level 2 at a flow rate of 5670 l/s

Finally, solids handling was assessed at a flow rate of 1400 l/s, the lowest representative flow rate investigated within the model. At this inflow the tail water level within the shaft was close to the shaft invert and cascading free-surface flow occurred throughout the drop structure. All model solids were regularly transported and cleared through the drop structure at this flow rate. This demonstrated that all solids would be cleared from the drop structure at under all cascading flow conditions, as the lowest flow rate produces the lowest flow velocities.

Images 4.23 and 4.22 show solids at Levels 1 and 5 respectively at a flow rate of 1400 l/s. At this flow rate, fewer solids were in suspension and more solids tracked along the floor of each the platforms, due to the lower velocities. At Level 1, a small recirculation region occurred to the right of the Inlet Culvert, as viewed from upstream to downstream, in which a small number of settleable solids remained temporarily. These settleable solids were regularly agitated and cleared from the platform after a short duration of operation. Similarly to the upper section of the drop structure at 5670 l/s, solids were transported across the intermediated platforms and over the platform ledges. At Level 5, all solids were readily transported forward to the Outlet Culvert.



Image 4.23– Absence of settleable solids at Level 1 at a flow rate of 1400 l/s



Image 4.23– Absence of solids at Level 5 at a flow rate of 1400 l/s

Conclusions from Solids Assessment

Solids were readily transported and cleared through the full drop structure at all flow rates investigated. At the high inflow and high shaft water levels, solids were maintained in suspension and readily transported through the drop structure. Highly buoyant solids, such as footballs and plastic bottles, were not drawn into through the Inlet Culvert due to the water depth in the drop structure exceeding the soffit of the Inlet Culvert. This is representative of full scale flow conditions.

At a flow rate of 5670 l/s, the water level was at approximately the mid depth of the drop structure. In the upper section of the drop structure free-surface cascade flow occurred and solids were either suspended in the flow, or tracked along the base of each platform, prior to being transported over the platform ledge to the receiving platform below. In the lower section of the drop structure submerged flow conditions occurred and solids were cleared in a similar manner to those at the higher flow rates and tailwater levels. There was some temporary retention of highly buoyant solids within the submerged flow region, however, the highly buoyant solids were regularly cleared forward after a short duration of operation.

At the lowest flow rate of 1400 l/s, the lowest representative flow rate investigated within the model, all solids entering the Inlet Culvert were transported through and cleared from the drop structure. Cascading free-surface flow occurred throughout the model and solids were transported across each platform, either in suspension or across the base of the platform, prior to passing over the platform ledge to the receiving platform below. This indicates that all solids will also be cleared through the drop structure at the higher flow rates of 2800 l/s and 3500 l/s, at which cascading flow also occurred and the flow conditions were comparable.

5.0 CFD MODELLING OF DRY WEATHER FLOW CONDITIONS

5.1 Introduction

Physical hydraulic modelling of the Bury Drop Structure MH6 was conducted for the flow rates indicated in Table 1.03, which represent storm flow conditions. There is also value in understanding the performance of the drop structure under normal operation, such as dry weather flows and the average design flow. However, the initial scope of the physical hydraulic model study was to investigate significantly higher flows than those which will occur during these conditions of operation and the geometric scale of the model was chosen accordingly. This dictated a model scale of 1:7, physical hydraulic modelling of flows of less than 1400 l/s was not possible on this model apparatus due to limitations in scale. At inflows of less than 1400 l/s flow within the model was non turbulent, i.e. viscous and surface tension effects dominate, which is unrepresentative of full scale prototype flow. Therefore, any analysis of the flow conditions at the low flow rates would be not be representative or reliable.

Hydraulically, it is not expected that there are any potential risks to the integrity of the drop shaft at lower inflow rates. However, it was considered that there may be the potential for the accumulation of fine sands and silts on each of the platforms, which may not be adequately cleared to the outlet culvert at lower flow rates. The potential for persistent siltation may therefore arise on the cascade platforms. Model testing has shown that these solids will be cleared at the 2 year return period and above, however, this frequency of solids clearing may not be considered acceptable.

It is not possible to determine how solids would be deposited within the cascade at flow rates of less than 1400 l/s, or what additional loading may arise from any potential solids accumulation. To adequately model the drop structure at the lower, normal operation flow rates, a reduced model scale, such as 1:2, would be required. Construction of such a model would be impractically large and costly and is not a viable option. Therefore, computational fluid dynamics (CFD) modelling of the drop structure is a more practical modelling Approach. As such, a CFD modelling study of the Bury Drop MH6 was conducted at DWF conditions. All CFD modelling was conducted at the Q50 flow rate of 27 l/s, representing the peak flow which is exceeded 50% of the time, with a free-discharge condition at the downstream outlet boundary.

5.2 Primary CFD Modelling

Figures 5.01 and 5.02 provide details of the initially proposed revisions to the Bury Drop Structure MH6 arrangement to mitigate against potential solids accumulation at inflows of less than 1400 l/s. At each platform ledge, four slots of 150 mm width were proposed, in addition to the 750 mm gap from the end of the ledge to the shaft wall. The 750 mm gap was retained as physical hydraulic model testing showed that the drop structure performed satisfactorily at the tested return periods with this gap in place. The 150 mm slots were intended to promote spread of flow across the width of each platform to promote solids scour and transport and prevent excessive shoaling of lighter solids and silts. The addition of these 150 mm slots to the platform ledges is not expected to have any appreciable effect on the hydraulic performance of the drop structure at return periods of 2 years and above, i.e. as physically model tested.

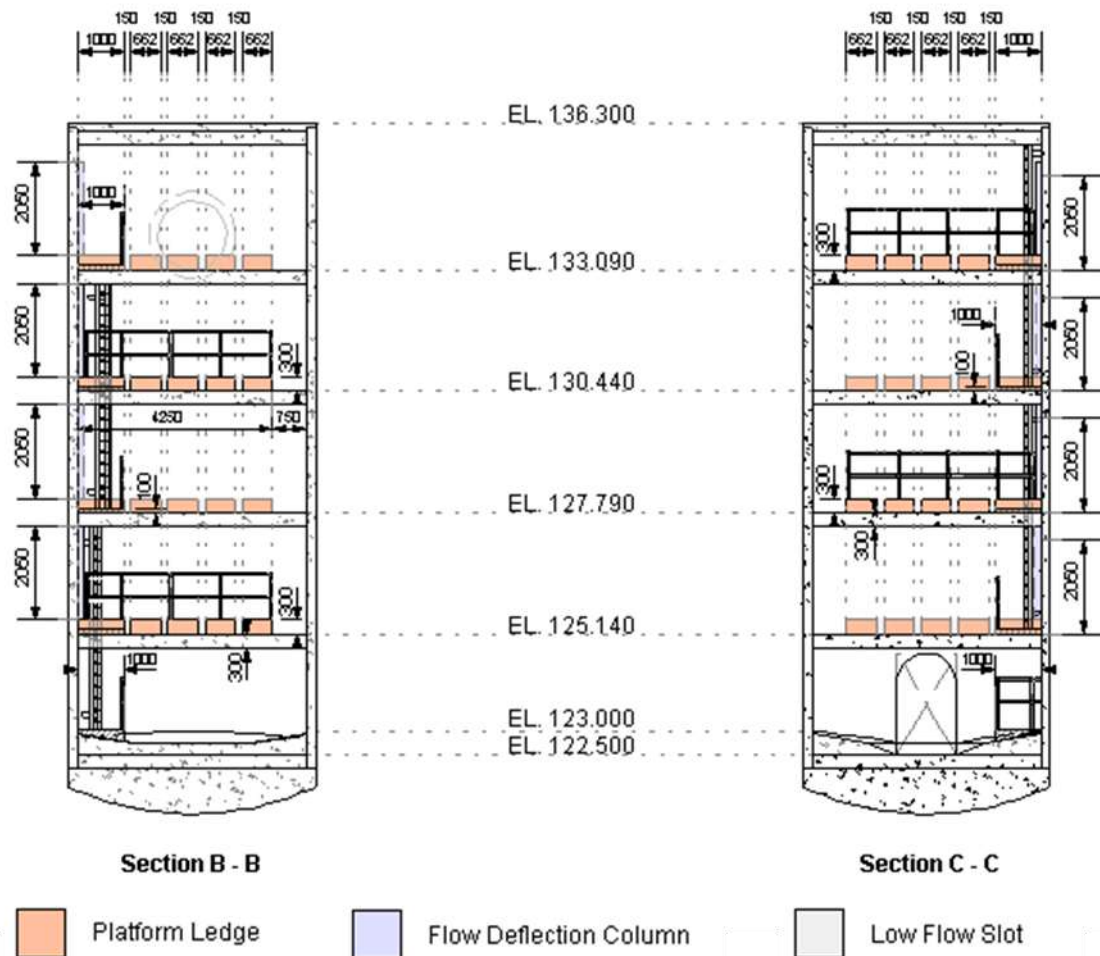


Figure 5.01 – Initially proposed low flow modifications – Sections

From Figures 5.01 and 5.02, a 3D model was generated. The 3D model was meshed into individual cells, over which the Navier-Stokes equations were solved numerically. The CFD modelling conducted in this study specifically utilised the Realisable $k-\epsilon$ turbulence model. The modelling was conducted utilising the commercial ANSYS Fluent solver.

Bury Drop structure MH6 – Physical Hydraulic Modelling

Figure 5.03 shows an image of the computational mesh used for the CFD analysis of the tank. The mesh architecture was arranged such that the zones having more pronounced velocity gradients, and thus requiring the greatest scrutiny, have finer grid pattern. Whereas, zones of less interest and impact on general flow patterns have coarser cells.

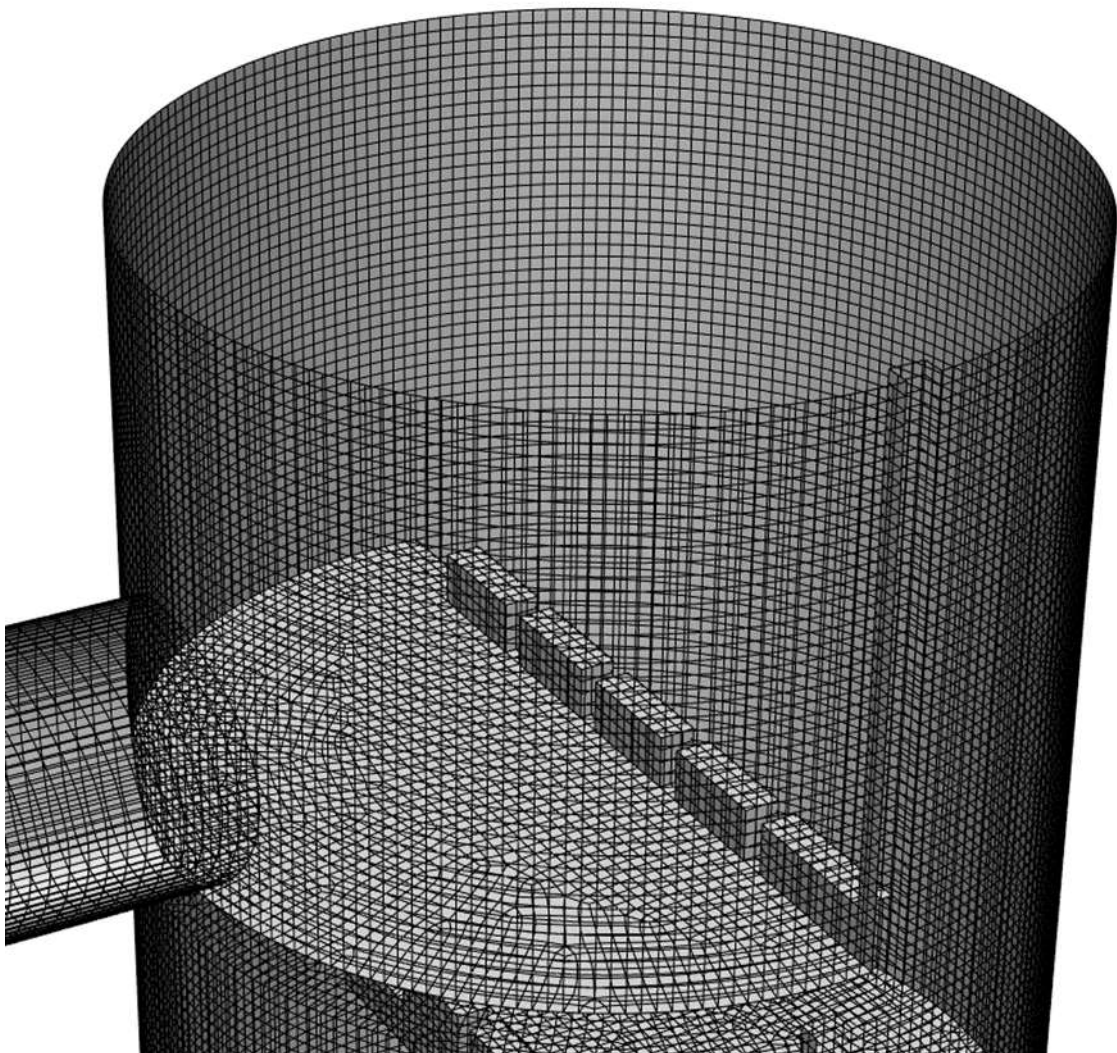


Figure 5.03 – Computational mesh used for primary CFD modelling

Figures 5.04 and 5.05 show the flow streaklines at the water surface, coloured by velocity, at the platforms of Levels 1 and 2. In both cases, large low velocity and recirculation regions occurred. This was due to the fact that the DWF velocities were too low to promote effective spreading of the flow, as was intended by the inclusion of the low flow slots. The low velocity recirculation regions would be prone to an accumulation of light solids and silt. Therefore, the proposed arrangement of the primary CFD analysis is not suitable for solids clearance at DWF conditions.

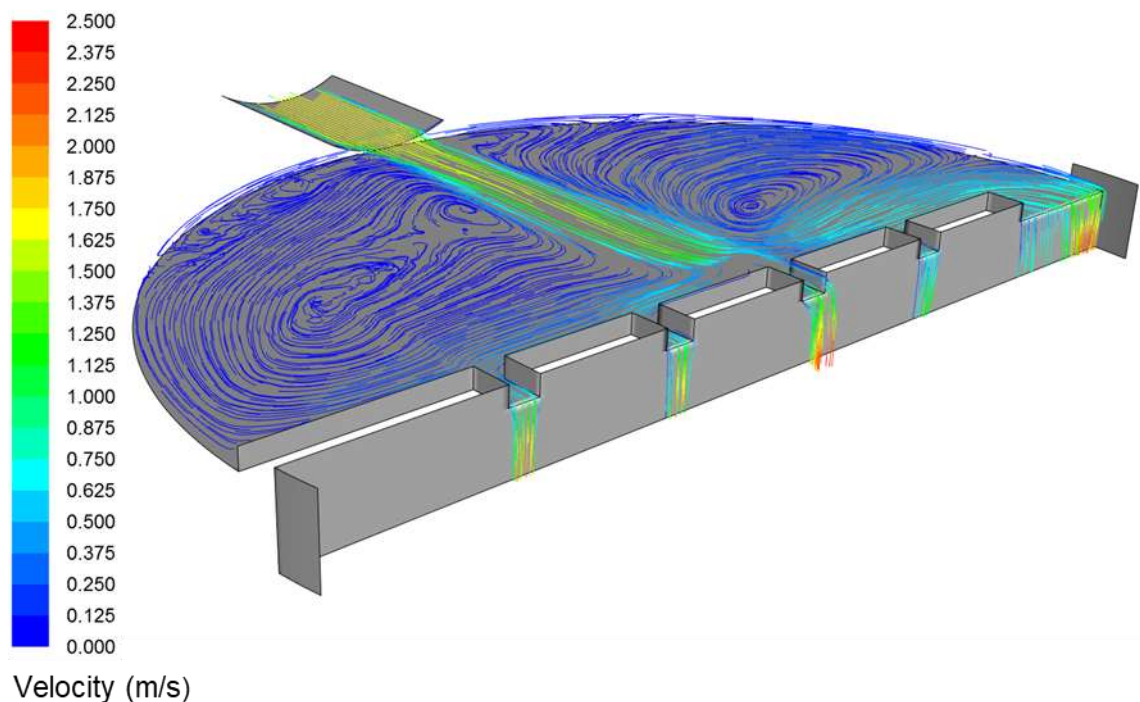


Figure 5.04 – Flow streaklines coloured by velocity at Level 1 – Initially proposed low flow arrangement

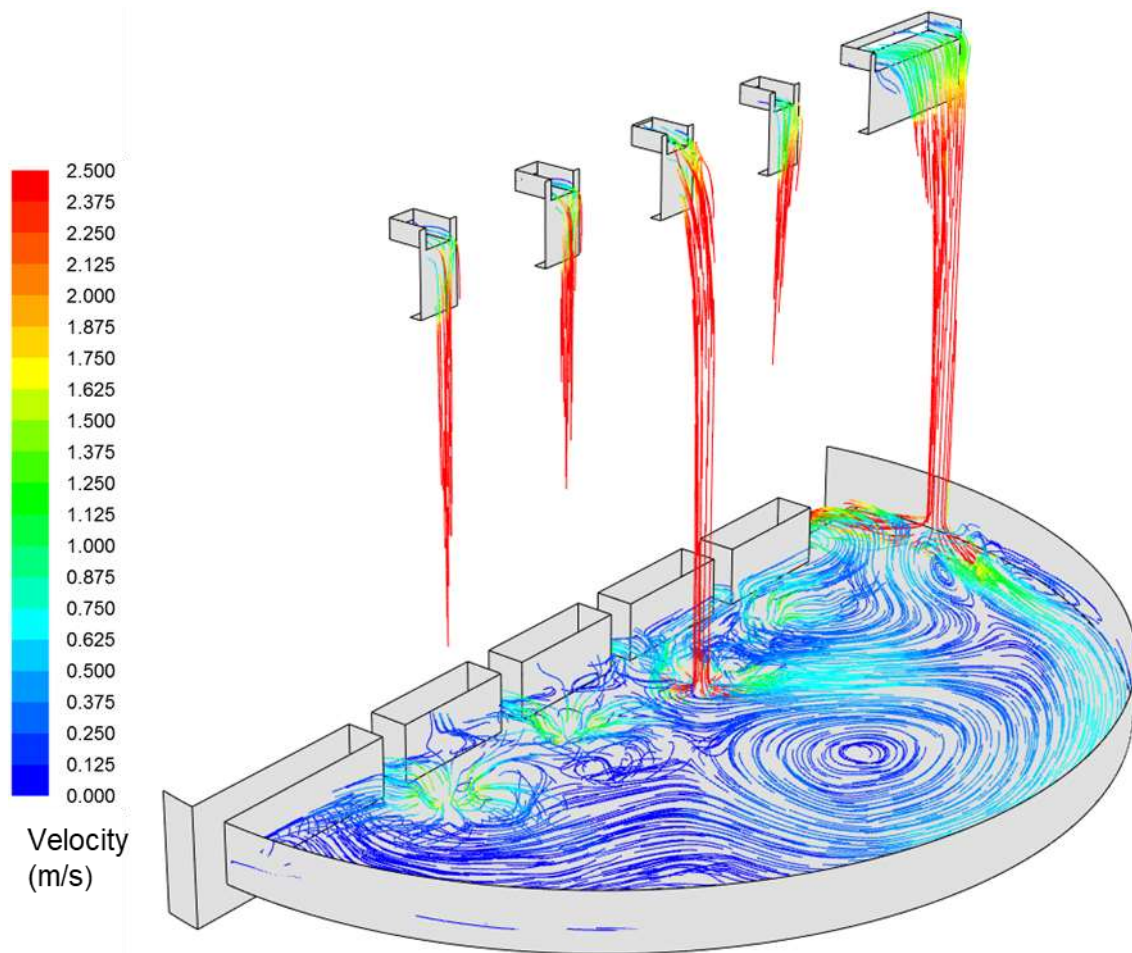


Figure 5.05 – Flow streamlines coloured by velocity at Level 2 – Initially proposed low flow arrangement

5.3 Secondary CFD Modelling

In order to further promote the clearance of solids, a secondary DWF arrangement was proposed. A DWF channel was proposed at the Level 1 platform, which runs directly from the inlet pipe to the 750 mm gap in the ledge, which was relocated to the centre of the ledge. Low gradient sloped benching was proposed across the remaining surface of the platform, to promote the transport of water and solids towards the DWF channel and the central 750 mm gap in the ledge. The low flow slots were retained at either side of the 750 mm ledge gap, as these will promote solids clearance at intermediate flows and would not impact either the DWF flow or storm flow conditions.

The proposed arrangement of the platforms at Levels 2, 3 and 4 were identical. Similarly to Level 1, the 750 mm ledge gap was relocated to the centre of the ledge. Sloped benching was proposed across the entire platform surface which would act to direct low flows and solids towards the 750 mm central gap in the ledge. The steepest part of the benching, adjacent to the 750 mm gap, is located so as to intercept the majority of the DWF from the platform above. Again, the low flow slots were retained at either side of the ledge, for the same reasons as at Level 1. The arrangement of Level 5 was identical to that of the physical modelling study. These modifications in geometry will promote solids clearance at DWF conditions, however, would have a negligible effect on the storm flow conditions. Therefore, the findings of the physical hydraulic modelling of storm flow conditions are applicable to this proposed arrangement.

These proposed modifications are shown in Figures 5.06 and 07. Full details of the proposed modifications are included in Appendix 2. The computational mesh was constructed with the same approach as that of the previous CFD modelling. The mesh at Level 2 is shown in Figure 5.08.

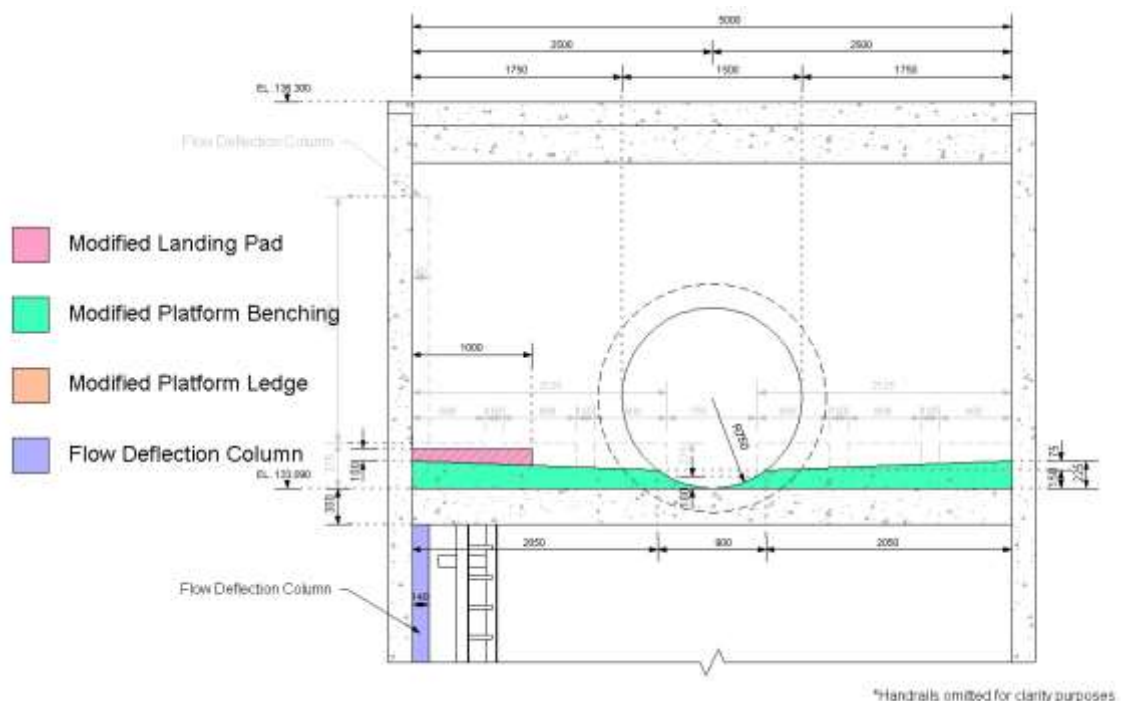


Figure 5.06 – Revised DWF modifications at Level 1

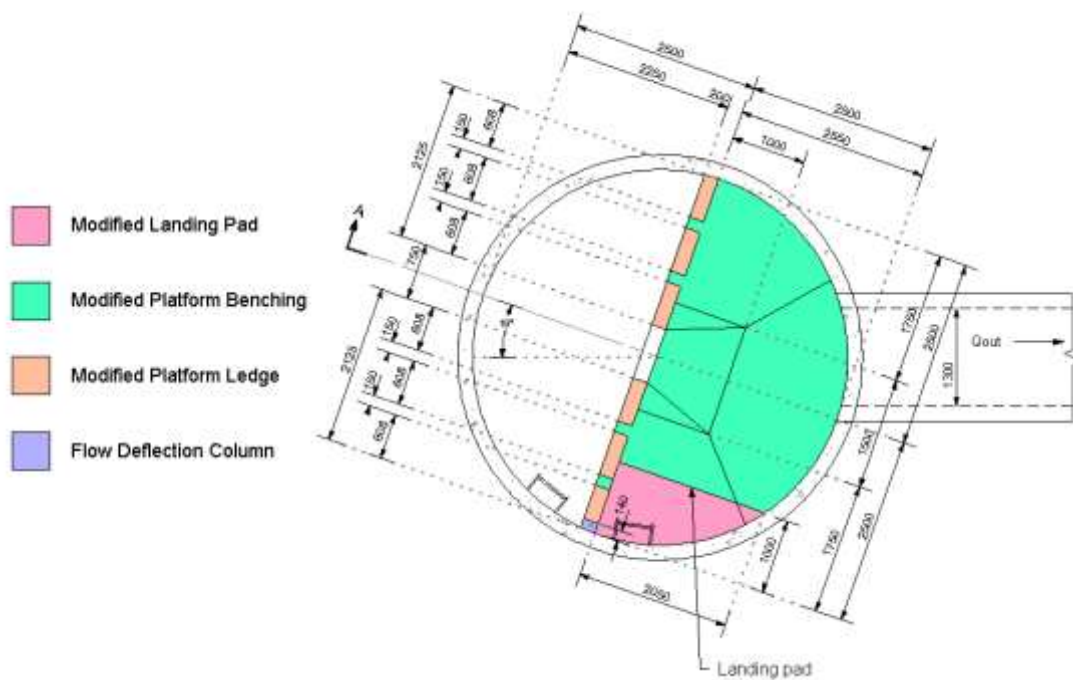


Figure 5.07 – Revised DWF modifications at Levels 2, 3 and 4

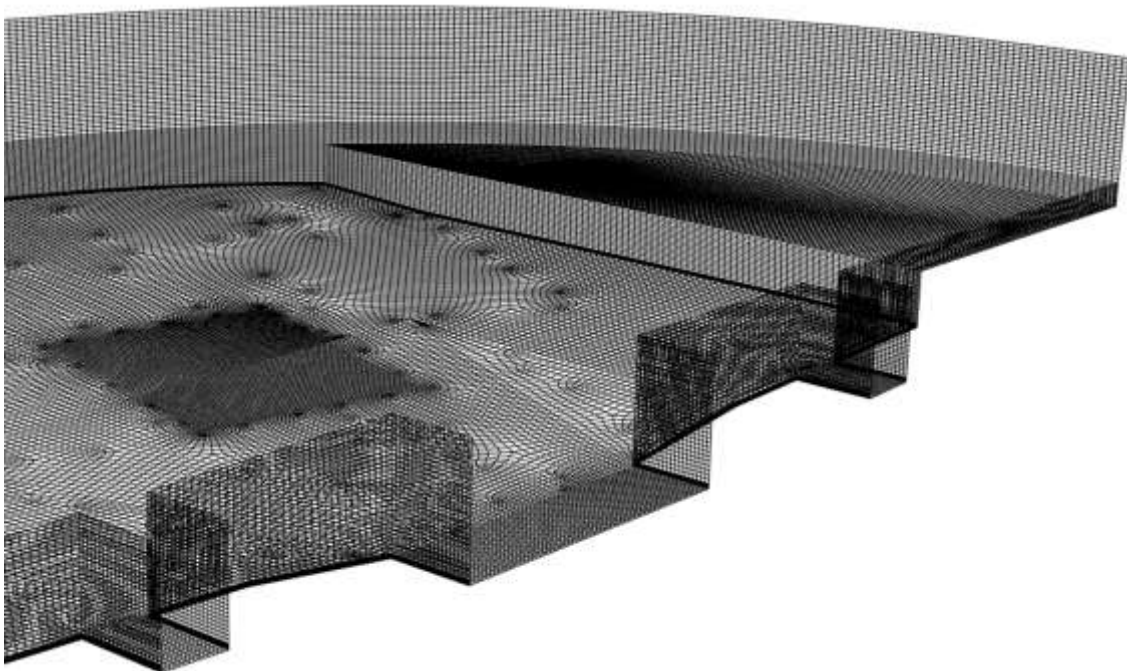


Figure 5.08 – Computational mesh at Level 2

Figure 5.09 shows an isosurface coloured by velocity of flow through the DWF channel of the platform at Level 1 at the Q50 flow rate of 27 l/s. The depth of the DWF channel was sufficient to contain all of the flow at Q50. Note that, for this reason, only the DWF channel was modelled in order to reduce computational time. All of the flow remained within the DWF channel and the velocities were sufficient to promote the clearance of silts and settleable solids through the channel. At intermediate inflow rates, the water level may increase so that not all of the flow would be contained within the DWF channel. The sloped benching outside of the DWF channel will promote the transport of solids towards the channel.

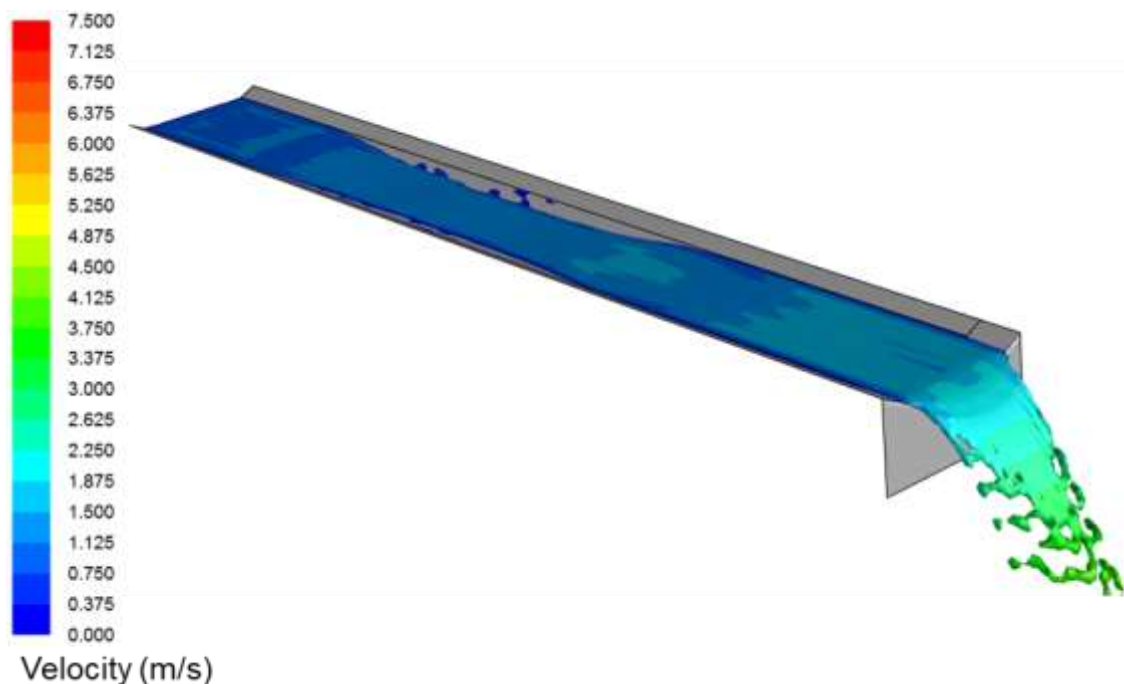


Figure 5.09 – Isosurface coloured by velocity of flow through the DWF channel of the platform at Level 1

Figure 5.10 shows flow streaklines at the water surface, coloured by velocity, of flow plunging from Level 1 and impacting on the Level 2 platform. Figure 5.11 shows flow streaklines adjacent to the surface of the platform. Flow impacted on the platform at a high velocity and spread radially across the platform, as would be expected. No recirculation regions occurred, which will prevent the accumulation of silts and settleable solids in any one location. As the inflow rate reduces and the water drains from the platform, stranding of a small number of solids will likely occur. This is to be expected and is not problematic for the performance of the drop structure. The highly turbulent nature of the flow will prevent a large accumulation of solids in any one location. CFD analysis found that almost all of the flow passed through the larger, central gap in the platform ledge, with only a small volume of flow passing through the smaller gaps in the ledge.

Figure 5.10 shows that some water overtops the man access ledge and spreads over a small region of the landing pad. The flow on the platform is transient and the flow onto the landing pad represents an intermittent wave. Only a small volume of solids would be transported onto the landing pad and the intermittent wave would aid in the clearance of any solids which were transported onto the landing pad. Therefore, it is not expected that there will be an issue with solids accumulation on the landing pad. Note that Figure 5.10 shows flow streaklines at only a small area of the landing pad, whereas Figure 5.11 shows streaklines across the entire ledge. This is because Figure 5.11 shows streaklines adjacent to the platform surface. In areas not occupied by water, such as on the landing pad, the streaklines represent the flow paths of the air within the model.

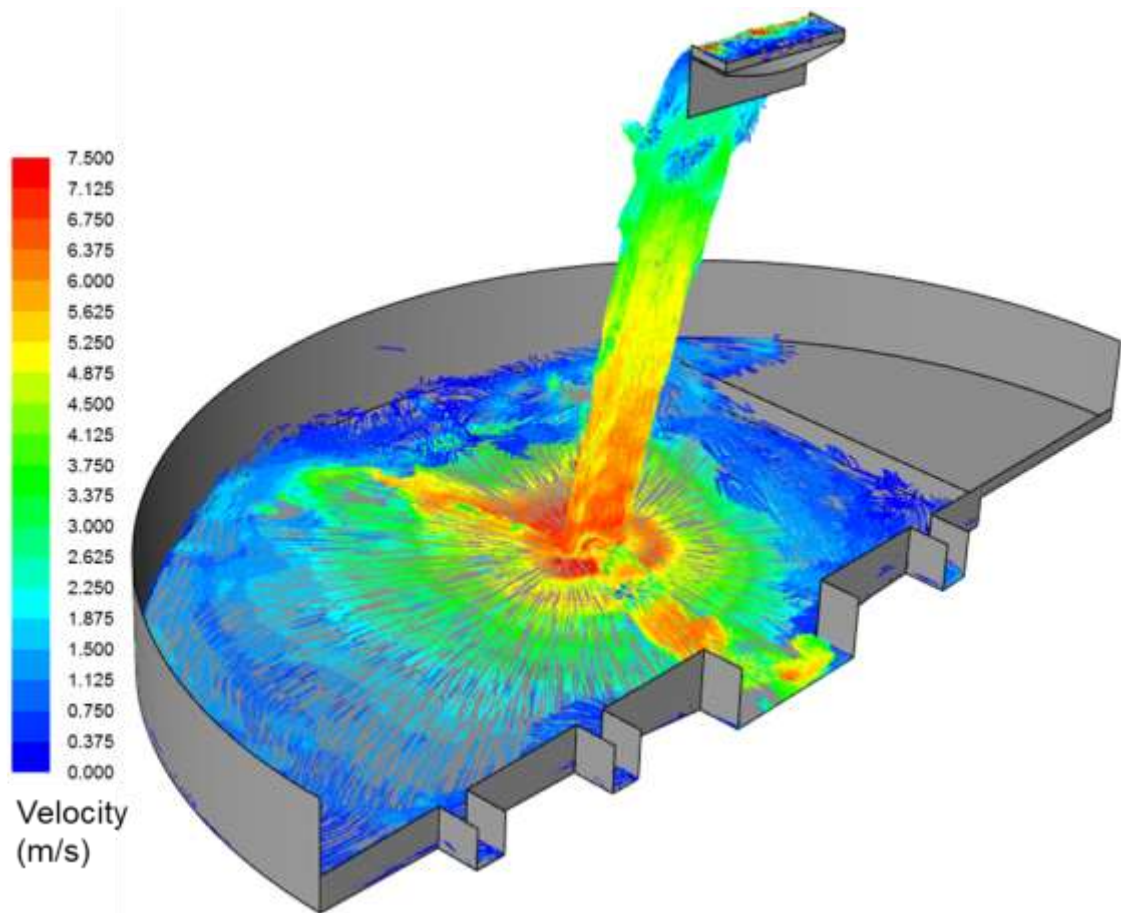


Figure 5.10 Flow streamlines at the water surface, coloured by velocity, at the Level 2 platform

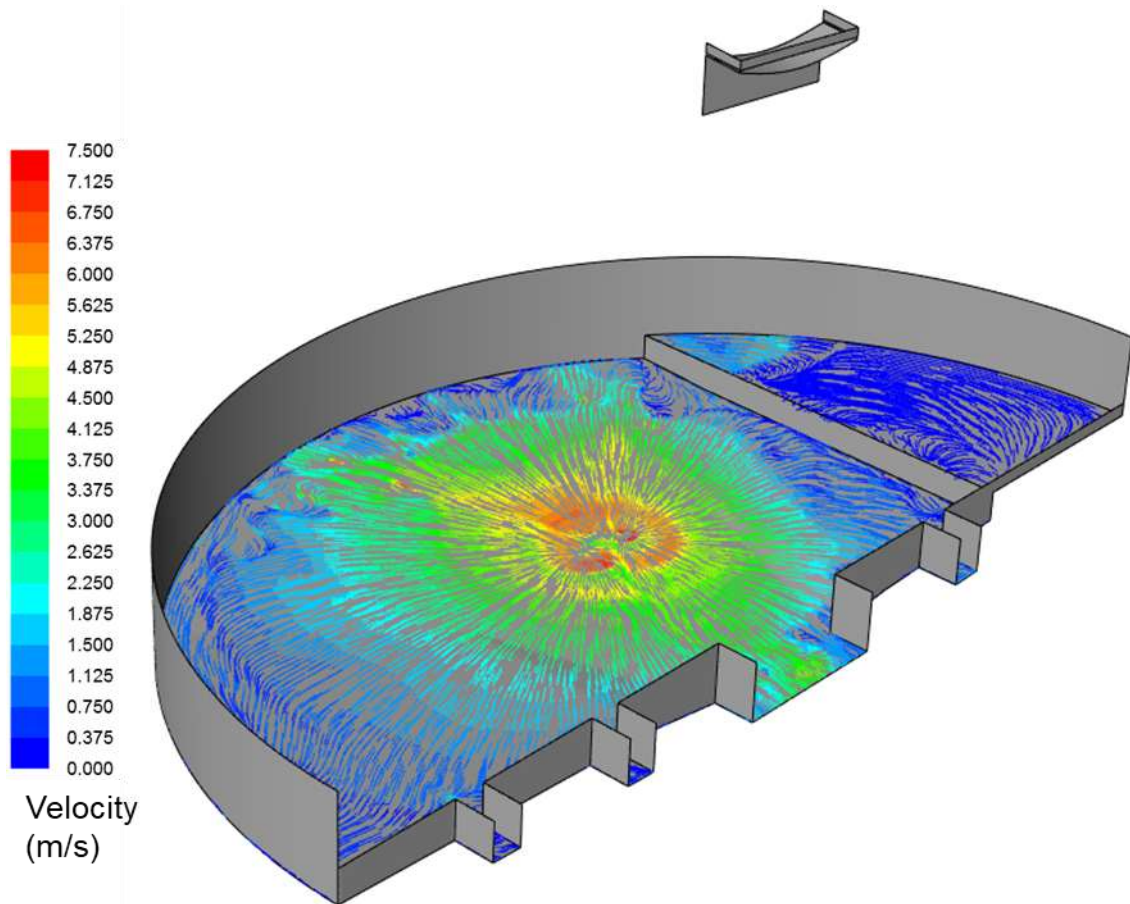


Figure 5.11 Flow streamlines at the platform surface, coloured by velocity, at the Level 2 platform

The proposed arrangements of the platforms at Levels 3 and 4 are identical to that of the platform at Level 2, so it is expected that the flow conditions at Levels 3 and 4 would be comparable to those at Level 2. Therefore, the platforms at Levels 3 and 4 were not modelled, as CFD modelling of these platforms would have been prohibitively time consuming and expensive and would have delayed the results significantly, only to produce findings which are comparable to those of Level 2. The outlet flow conditions at Level 2 were applied as the inlet conditions at Level 5.

Figure 5.12 shows streaklines at the water surface, coloured by velocity, at Level 5. As water from the platform above impacted the base of the drop structure and spread radially, with supercritical flow occurring. As the flow approached the opposing wall from the outlet channel, a hydraulic jump formed and flow was deflected laterally. The flow was transported around the supercritical flow region beneath the incoming jet, towards the outlet channel. A small volume of fluid was transported over the benching at the sides of the outlet channel, where the slope of the benching would aid the transport of flow and solids towards the outlet channel.

Figure 5.13 shows streaklines adjacent to the invert, also coloured by velocity, at Level 5. As at Level 2, streaklines are displayed across the entire surface of the invert, whereas in Figure 5.12, no streaklines are visible across parts of the benching. This is again due to the fact that, adjacent to the solid surface, streaklines are displayed for both air and water, however, in Figure 5.12, streaklines are shown at the water surface only. Figure 5.13 also shows that flow spread radially from the impact from the platform above, with higher velocity, supercritical flow occurring. The flow then deflected from the rear wall of the shaft, and was transported around the supercritical flow region, towards the outlet channel.

The velocities within the supercritical flow region were sufficient to scour silt and settleable solids from the invert. In the lower velocity regions, the gradient of the benching will promote the transport of solids towards the outlet channel. Similarly to at Level 2, stranding of solids will likely occur as the flow rate through the drop structure reduces. However, the velocities and turbulence at Level 5 are sufficiently high to prevent a permanent accumulation of solids in any one location. A small number of temporary stranded solids is to be expected, this would not be detrimental to the performance of the drop structure. The velocities through the outlet channel were also sufficiently high to clear silts and settleable solids. Therefore, CFD analysis has shown that this proposed arrangement would prevent the accumulation of silts and settleable solids at DWF conditions.

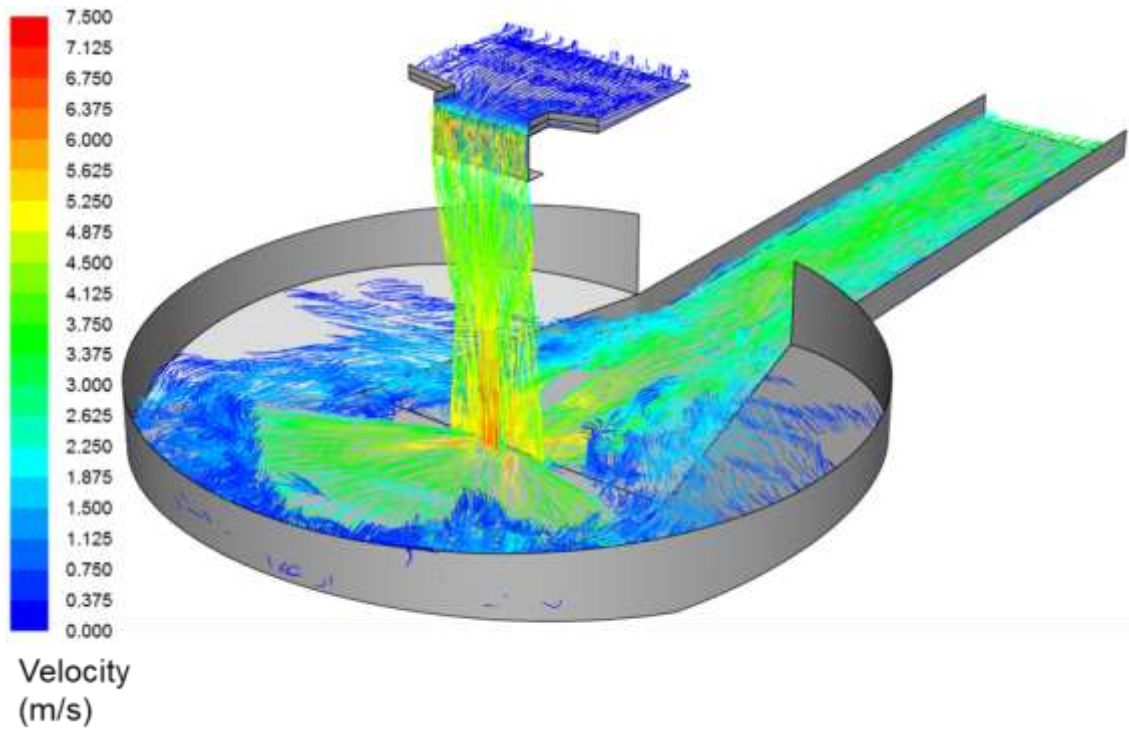


Figure 5.12 Flow streamlines at the water surface, coloured by velocity, at Level 5

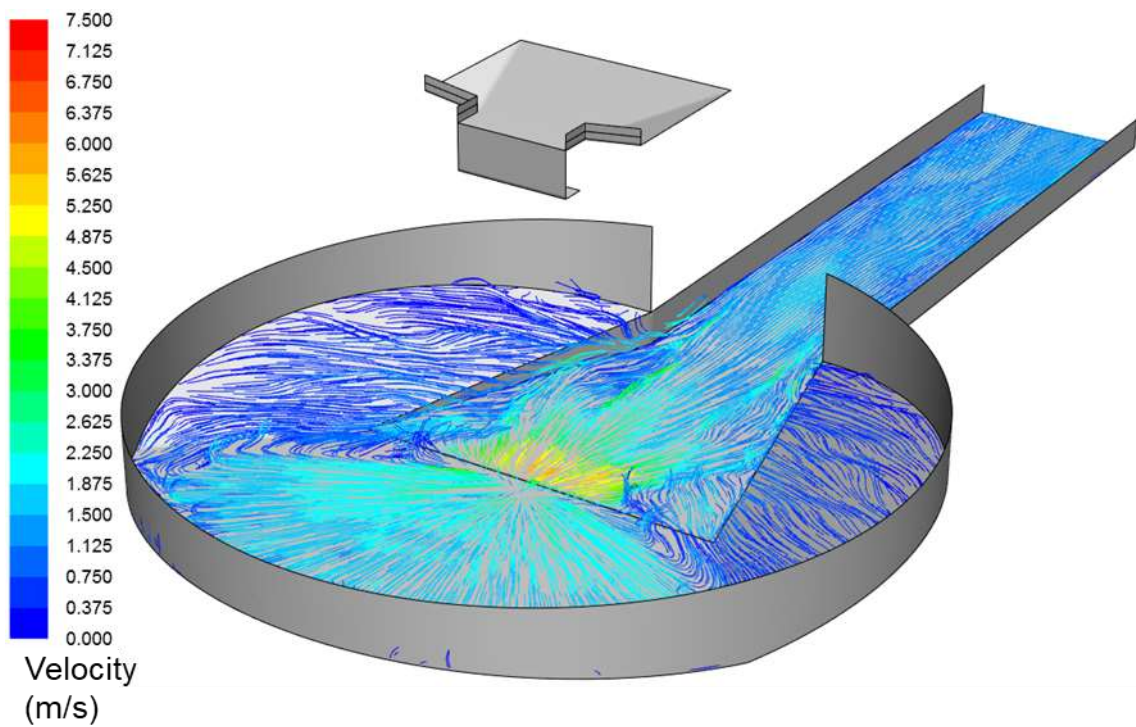


Figure 5.13 Flow streamlines at drop structure invert, coloured by velocity, at Level 5

5.4 Conclusions from CFD Modelling of Dry Weather Flow Conditions

Following physical hydraulic modelling of the storm flow conditions, analysis of the dry weather flow conditions (DWF) was conducted. Scaled physical hydraulic modelling of the DWF conditions was not appropriate, as the lower flow rates caused surface tension effects to dominate, preventing accurate analysis to be made. Therefore, Computational Fluid Dynamics (CFD) modelling of the DWF conditions was conducted at full scale. The main purpose of the DWF analysis was to determine whether silts and settleable solids would clear at the lower flow rates.

The arrangement of the drop structure was modified to aid the transport of solids. A number of additional slots were installed in the platform ledges in order to promote flow to spread and transport solids. Primary CFD analysis found that this arrangement resulted in low velocity, recirculation regions, which would be prone to solids accumulation. A revised arrangement was proposed in which a DWF flow channel was installed at Level 1 and benching was installed at Levels 2,3 and 4, in order to promote the transport of solids. CFD analysis of this revised arrangement found that at Level 1, all flow passed through the DWF channel with a high enough velocity to transport silts and settleable solid. At levels 2 - 5, CFD modelling found that temporary stranding of solids would likely occur. This is to be expected and is not detrimental to the performance of the drop structure. The velocities and turbulence throughout Levels 2 – 5 were sufficiently high to prevent long-term accumulation of solids in any one location. It should be noted that, the proposed modifications are small compared to the length scales of the storm flows. Therefore, the findings of the physical hydraulic model study of the storm flow conditions are applicable to the revised proposed DWF arrangement.

5.0 CONCLUSIONS

Initial testing

The Bury Drop Structure MH6 physical hydraulic model was tested at a number of flow rates representative of the specified flow range. Initial testing was conducted prior to confirmation of the tailwater levels by the Client, as such free discharge conditions were assumed for initial assessment purposes. Initial testing found that the Bury Drop Structure MH6 generally performed satisfactorily for the flow rates investigated, however, minor performance issues were identified that needed to be addressed. A continuous nappe formed at each cascade, which may prevent air from freely venting to atmosphere. Trapping air has the potential to locally pressurise the shaft, which may cause structural damage. The continuous nappes formed at each platform should be disrupted and broken up in order to allow air to be freely vented.

The proposed platform ledges acted to pool water on the platform which dissipated energy at each platform, however, at higher flow rates, a smaller proportion of the receiving flow was intercepted. This resulted in high energy flows cascading onto the platform below. The energy of these nappes formed at each platform of the cascade should be reduced, in so far as can reasonably be achieved.

Development

In order to mitigate the performance issues identified during initial testing, the following modifications to the Bury Drop Structure MH6 arrangement were made:

- At each platform, a flow deflection column was installed at the downstream ledge of each platform, to one side of the shaft, adjacent to the shaft wall. The purpose of the flow deflection columns was to deflect nappe flow away from the shaft wall, creating an air gap through which the entire chamber could be vented to atmosphere. The flow deflection columns were also intended to direct flow away from the shaft man access ladders located at each platform, in so far as was possible. This was proposed in order to reduce potential ragging of solids to the access ladders and to protect them from excessive hydraulic loading.

- The height of each platform ledge was increased by 50 % from 200 mm to 300 mm. The increase in ledge height was intended to intercept a higher proportion of the flow, which will increase the depth of pooled water retained at each platform. The increase in the retained water depth will dissipate a larger proportion of the energy of the receiving flow. It should be noted that, although the increase in ledge height will dissipate more energy, the flows through the drop structure are inherently high energy, especially at the higher flow rates. Therefore, although energy dissipation will be increased, the nappe flows produced at each platform will still exhibit significantly high energy and turbulence.

Final Testing

The modified Bury Drop Structure MH6 physical hydraulic model was tested at a range of flow rates and water levels provided by and agreed upon by the Client. Hydraulic assessment of the Bury Drop Structure MH6 physical hydraulic model found that the drop structure conveyed the flows safely and performed satisfactorily for all flow rates and tailwater levels investigated. Three different general flow conditions occurred within the drop structure, depending on the water level within the shaft:

1. At the higher water levels (100 years + 70% and 1000 year return periods), the drop structure was almost completely surcharged, i.e. up towards the shaft soffit. As flow passed across the shaft soffit at surcharged operation, air was entrained into the flow from the vent provided in the model soffit. This indicates that a vented soffit must also be installed in the prototype drop structure, as failure to do so may result in the chamber becoming depressurised, which could lead to structural damage and possible accelerated failure.
Submerged flow conditions occurred throughout the drop structure at the specified flow range of 100 years + 70% and above. At these conditions, flow paths were dictated by the arrangement of the platforms and the flow was conveyed through the drop structure and into the Outlet Culvert safely.
2. At a return period of 100 years + 35%, the water level within the drop structure was approximately at mid shaft height. In the upper section of the drop structure free-surface flow occurred and the flow cascaded from each platform to the next. This regime was characterised by high levels of air entrainment at each platform. The proposed increase in the height of the platform ledges increased the depth of the retained flow on each platform, which in turn increased energy dissipation.

The proposed vertical flow deflection columns at each platform ledge acted to deflect flow away from the wall of the drop structure, allowing air to freely vent to atmosphere. The flow deflection columns also deflected flow away from the man access ladders somewhat. At the lower depths, submerged flow occurred and the flow patterns were comparable to those at the higher water levels, but with slightly lower velocities and a higher volume of entrained air.

3. At the lower tailwater levels (return periods of 2 years, 20 years and 50 years) the water levels within the drop structure were closer to the invert of the shaft. Under these conditions, flow was also conveyed safely through the drop structure. Free surface cascading flow occurred at each landing level within the drop structure, characterised by high levels of splash, spray and air entrainment. Again, the proposed increased ledge heights increased the depth of retained water on the platforms, increasing energy dissipation, and the flow deflection columns diverted flow away from the wall of the chamber, creating an air gap through which air could be freely vented to atmosphere. The water levels at the base of the drop structure shaft, dictated by the receiving water course tailwater levels, also acted to cushion the flow and increase energy dissipation.

The solids handling of the drop structure was investigated for a range of operating scenarios. Solids were cleared from the drop structure quickly and efficiently at all flow rates investigated. At the higher water levels, solids were held in suspension and transported through the drop structure. Highly buoyant solids, representative of solids such as footballs and plastic bottles, were not drawn through the Inlet Culvert due to the water depth in the drop structure being significantly higher than the soffit of the Inlet Culvert. This is representative of the prototype flow conditions.

At a flow rate of 5670 l/s, the water level was at approximately the mid height of the drop structure. In the upper section of the drop structure free-surface cascade flow was exhibited and solids were either suspended in the flow or tracked along the base of each platform, prior to being transported over the platform ledge to the receiving platform below. In the lower section of the drop structure submerged flow conditions occurred and solids were cleared in a similar manner to those at the higher flow rates and tailwater levels. There was some temporary retention of highly buoyant solids within the submerged flow region within the shaft, however, the highly buoyant solids were cleared from the shaft towards the Outlet Culvert after a short duration of operation.

At the lowest specified flow rate of 1400 l/s, all solids were cleared throughout the drop structure. Cascading free-surface flow occurred throughout the structure and all model solids (floating, suspended and settleable), were transported across each platform, either in suspension, or across the base of the platform to pass over each platform ledge to the successive receiving platform below. Solids were directed towards the central channel formed at the invert of the shaft and out into the outlet culvert within the region of fastest moving flow.

At the lowest specified test flow rate of 1400 l/s, the lowest velocities occurred throughout the drop structure. This indicates that all solids will also be cleared forward to the Outlet Culvert at the higher flow rates of 2800 l/s and 3500 l/s, at which cascading flow also occurred and the flow conditions were comparable.

CFD Modelling of Dry Weather Flow Conditions

Following physical hydraulic modelling of the storm flow conditions, analysis of the dry weather flow conditions (DWF) was conducted. Scaled physical hydraulic modelling of the DWF conditions was not appropriate, as the lower flow rates caused surface tension effects to dominate, preventing accurate analysis to be made. Therefore, Computational Fluid Dynamics (CFD) modelling of the DWF conditions was conducted at full scale. The main purpose of the DWF analysis was to determine whether silts and settleable solids would clear at the lower flow rates.

The arrangement of the drop structure was modified to aid the transport of solids. A number of additional slots were installed in the platform ledges in order to promote flow to spread and transport solids. Primary CFD analysis found that this arrangement resulted in low velocity, recirculation regions, which would be prone to solids accumulation. A revised arrangement was proposed in which a DWF flow channel was installed at Level 1 and benching was installed at Levels 2,3 and 4, in order to promote the transport of solids. CFD analysis of this revised arrangement found that at Level 1, all flow passed through the DWF channel with a high enough velocity to transport silts and settleable solid. At levels 2 - 5, CFD modelling found that temporary stranding of solids would likely occur. This is to be expected and is not detrimental to the performance of the drop structure. The velocities and turbulence throughout Levels 2 – 5 were sufficiently high to prevent long-term accumulation of solids in any one location. It should be noted that, the proposed modifications are small compared to the length scales of the storm flows. Therefore, the findings of the physical hydraulic model study of the storm flow conditions are applicable to the revised proposed DWF arrangement.

APPENDIX 1 MODEL SIMILARITY

A1.1 Model Scale

The fundamental scale for any hydraulic model is the geometric scale, that is the ratio of a given length in the model to a given length in the full scale application. Practical considerations such as the circulating pump capacity, depth of model, the floor space utilised and ultimately cost, place limits on the maximum size of a model. Theoretical considerations regarding the correct representation of flow conditions places limits on the minimum size of a model.

The correct representation of flow conditions principally requires that the Reynolds Number to be above a critical value within the model to ensure that, where fully turbulent conditions are present in the prototype, similar turbulent conditions exist within the model. If the model is too small this condition will not be met.

Much has been written concerning the problem of scaling results from a physical model to predict full-size prototype performance. It is apparent that a scaling law for one arrangement may not be appropriate for another and thus no definitive, universally applicable law exists. In open channel studies it is generally accepted that the Reynolds Number should be at least 2,500 to ensure fully turbulent conditions are present. In studies where vortex formation may occur it is necessary to ensure that the Reynolds Number, at the outlet culvert, assuming full flow depth, has a value of at least 30 000 to 40 000 to eliminate viscous effects.

In this instance, at a nominal minimum flow of 1400 l/s the full scale Reynolds Number at the outlet culvert will be approximately 1148983. From practical considerations the model was constructed to a scale of 1/7 full size which will result in a Reynolds Number at the plane of the model outlet culvert of approximately 61336. This is in excess of quoted minimum values, and will ensure similarity between model and prototype at all flow rates investigated.

Following the selection of an appropriate geometric scale, the dynamic scale of for example, velocity and discharge, will be determined according to scaling criteria dictated by a particular model law. For free surface flows the governing forces are gravity and inertia, these forces are related by the Froude Scaling Law, which may be stated as:

Froude Number similarity will be achieved if the Froude Number in the prototype (F_p) and model (F_m) are equal, i.e. :

$$F_p = \frac{V_p}{\sqrt{g \cdot L_p}} = F_m = \frac{V_m}{\sqrt{g \cdot L_m}} \quad 1.1$$

Where V_p and V_m are the velocities in the prototype and model, L_p and L_m are characteristic lengths in the prototype and model and g is the gravitational constant. This will ensure dynamic similarity for flow regimes governed primarily by gravity. Rearrangement of equation 1.1 gives the velocity scale :

$$V_m = V_p \frac{\sqrt{g \cdot L_m}}{\sqrt{g \cdot L_p}} \quad 1.2$$

For a geometrically similar model the model scale factor may be defined by $S = L_m / L_p$ and the gravitational constant g will be equal, hence

$$V_m = V_p S^{1/2} \quad 1.3$$

The flow scale may be derived by continuity, whereby

$$Q \propto L^2 V \quad 1.4$$

Substitution into equation 1.2 gives

$$Q_m = Q_p S^{5/2} \quad 1.5$$

Similarly, transformation to take account of time is made using

$$V \propto L/T \quad 1.6$$

so that

$$T_m = T_p S^{1/2} \quad 1.7$$

Force scales are based on the ratio of measured force to inertial force and provided these are maintained in equal ratio model similarity of forces will also be achieved:

$$\frac{F_p}{\rho L_p^2 V_p^2} = \frac{F_m}{\rho L_m^2 V_m^2} \quad 1.8$$

so that assuming the model and full scale application are operated with the same fluid, substitution into equation 1.3 gives

$$F_m = F_p S^3$$

1.9

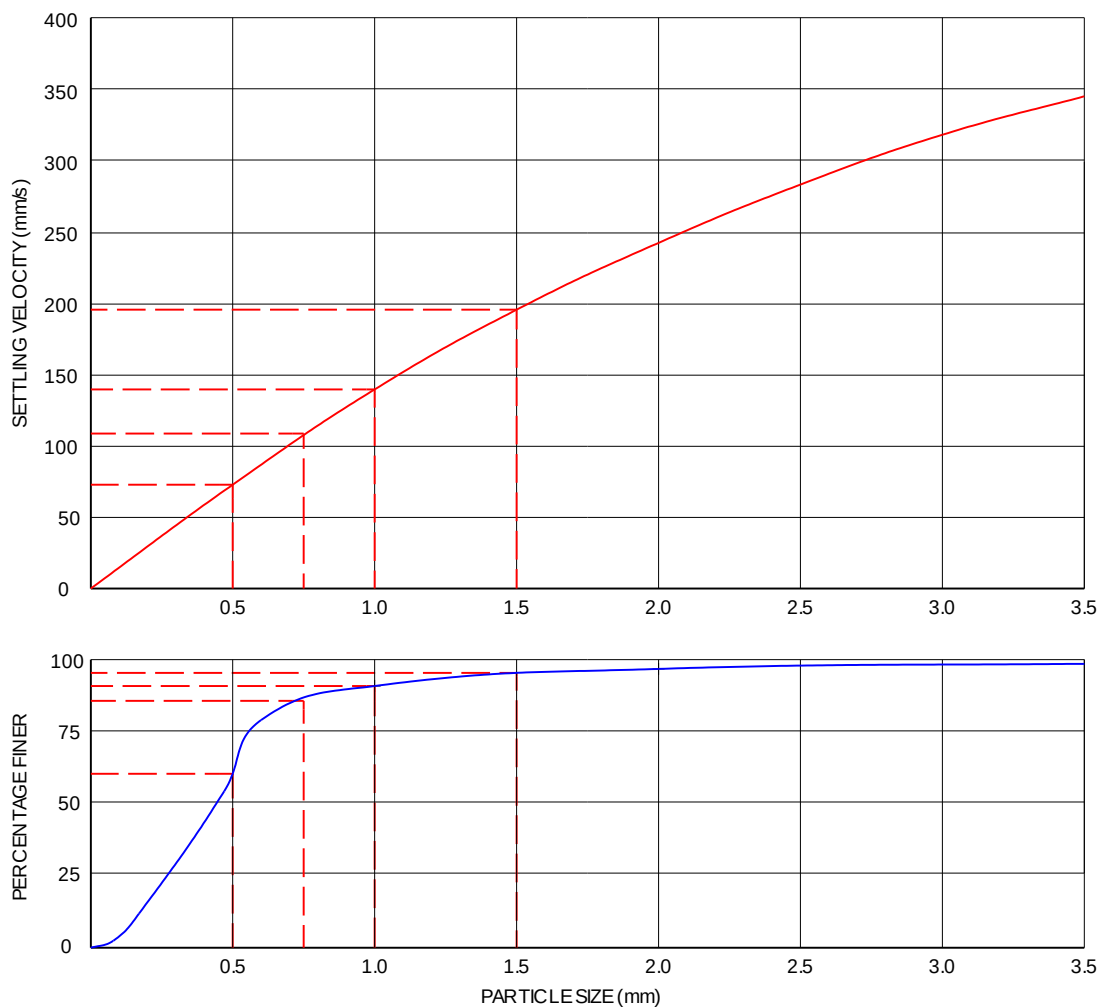
For the application in question a geometric scale of 1 to 7 was selected, hence the appropriate dynamic scales may be calculated as follows

Property	Scale Relationship	Model Value
Geometric	$S = L_m / L_p$	1/7
Velocity	$V_m = V_p S^{1/2}$	1/2.65
Flow	$Q_m = Q_p S^{5/2}$	1/130
Time	$T_m = T_p S^{1/2}$	1/2.65
Force	$F_m = F_p S^3$	1/343

A1.2 Modelling grit

It is important to represent the solid fraction of storm water and surface water in order to highlight potential problems of grit settlement and accumulation. Various methods of representing grit particles may be employed, in our experience grit transport and settlement can be successfully simulated using graded samples of plastic particles. Care must be taken to ensure the particles are of uniform density and free from air.

In accordance with scaling principles the plastic particles are selected on the basis of a settling velocity equal to the square root of the full scale particle settling velocity. The characteristic particle size distribution and settling velocity of a typical sewage sample are presented below.



The characteristic particle size distribution and settling velocity of a typical sewage sample

Interpretation of the model grit characteristics must be carried out as it is not possible to represent the cohesive or organic binding properties of the sewage solids. Also, value judgements have to be made with respect to disregarding particles in the sample which fail to become adequately wetted or to which air bubbles have attached. These techniques have however been shown in practice to be accurate in predicting grit transport and deposition.

A1.3 Grease

The definition of 'grease' usually refers to agglomerated particulate matter which is derived from a complex mixture of both the organic and inorganic constituents of sewage. The constituents are derived primarily from food sources and domestic and industrial oils, grease generally enters sewers in an emulsified form. De-emulsification of these source materials due to cooling results in separation of the solid constituents and an agglomeration with other non-greasy particulate matter. If left to accumulate, grease may create significant maintenance problems. Agglomeration with inorganic constituents such as rags may result in matting and, under extreme conditions, bridging to form a solid impenetrable barrier. Balling of grease and rags may also lead to maintenance problems further downstream.

The accurate representation of 'grease' properties within a physical model is not possible due to difficulties associated with scale similarity of the agglomeration process. However, indicative behaviour may be simulated using positive and neutral buoyancy solids such as chopped sponge, polystyrene and sawdust. Sawdust acts as an emulsion and also as a binding agent, to provide a good indicator of composite grease type properties.

A1.4 Scale effects

Scale errors occur whereby the laws of dimensional similarity cannot be satisfied simultaneously within a model operating on the basis of one particular law of affinity. This model was operated on the basis of Froude Number similarity, this effectively exaggerates the viscous influence defined by the Reynolds number (for non-turbulent conditions) and also exaggerates surface tensile forces defined by the Weber number. This condition is termed a 'scale effect'.

Under this model application, air was entrained in the flow at the cascade drops or at the soffit of the drop structure. Due to surface tension effects, air bubbles generated within a model exhibit the same absolute size as those generated within the prototype. As a result, air bubbles apparent in the model will be disproportionately large compared to their boundary scale.

The prototype, which operates at a higher Reynolds Number and therefore higher shear rate, will exhibit a larger number of proportionately smaller bubbles.

As a result of this condition, model bubbles will be more buoyant than their prototype full scale counterparts and will therefore exhibit a higher rise velocity. This results in model bubbles approaching the water surface quicker than their prototype counterparts, which will travel further. Hence the rise trajectory of model bubbles will be steeper.

It can be shown that within a Froude scale model, the depth of air bubble penetration is dictated by gravitational and inertial forces, provided flow is fully turbulent. Hence this characteristic will be correctly represented within the model provided the model is operated in excess of the Critical Reynolds Number (Re_c of approximately 2000), which will ensure that the flow regime is fully turbulent. Therefore, where prototype operation is critical in relation to the depth of air bubble penetration, conditions will be correctly represented within the model, provided the geometric scale is selected to satisfy Reynolds Number constraints.



The depth of air bubble penetration and subsequent rise trajectory is of importance where the full scale installation has a bearing on the passage of air to say, a pump or transfer pipe, whereby more air may enter the pump or pipe within the prototype than within the model. Hence, where considered critical, an assessment of the relative characteristics of the prototype bubble flow paths and model bubble flow paths must be made. The deviation in bubble rise trajectory cannot be eliminated within a small scale Froude model, however the effect is reduced as the model scale is reduced.

APPENDIX 2 DRAWINGS

The following drawings form part of this report:-

<i>Title</i>	<i>Drawing No.</i>
General Arrangement Drawings	
Drop Shaft Design – Former Tetrosyl Site, Bevis Green, Bury Sections A-A, B-B and C-C	01
Drop Shaft Design – Former Tetrosyl Site, Bevis Green, Bury Plans A-A and B-B	02
Drop Shaft Design – Former Tetrosyl Site, Bevis Green, Bury Plans C-C and D-D	03
Drop Shaft Design – Former Tetrosyl Site, Bevis Green, Bury Plan E-E	04
Details of Modifications	
Bury Drop Shaft - Details of Modifications - Plan at A-A	05
Bury Drop Shaft - Details of Modifications - Plan at B-B	06
Bury Drop Shaft - Details of Modifications - Plan at C-C	07
Bury Drop Shaft - Details of Modifications - Section A-A (of Plan at A-A)	08
Bury Drop Shaft - Details of Modifications - Section A-A (of Plan at B-B)	09
Bury Drop Shaft - Details of Modifications - Section A-A (of Plan at C-C)	10
Bury Drop Shaft - Details of Modifications - Section B-B (of Plan at A-A)	11
Bury Drop Shaft - Details of Modifications - Section B-B (of Plan at B-B)	12
Bury Drop Shaft - Details of Modifications - Section B-B (of Plan at C-C)	13

Bury Drop Shaft - Details of Modifications - Section C-C (of Plan at A-A)	14
Bury Drop Shaft - Details of Modifications - Section C-C (of Plan at B-B)	15
Bury Drop Shaft - Details of Modifications - Section C-C (of Plan at C-C)	16



Section B - B

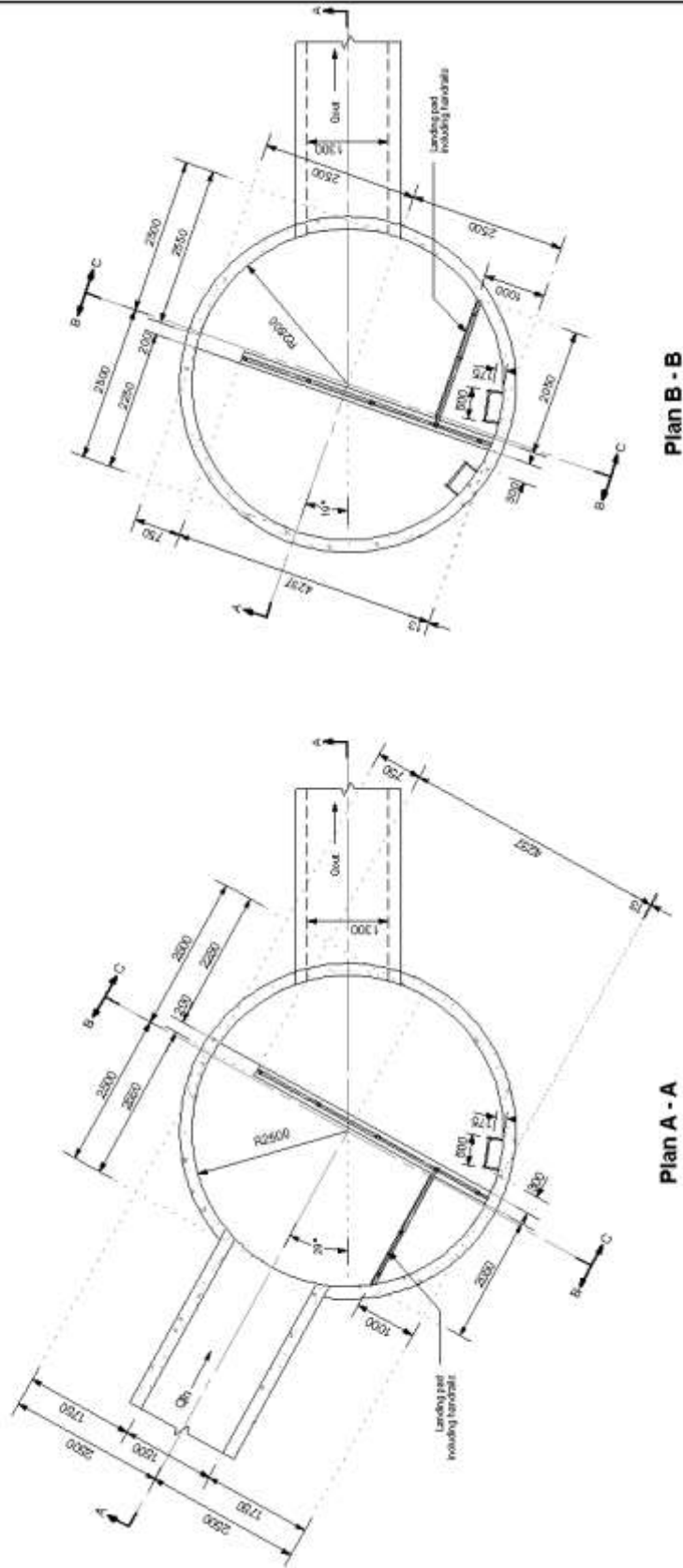
Section A - A

**Drop Shaft Design - Former Tetrosyl Site, Bevis Green, Bury
Sections A - A, B - B and C - C**



Drawing 01

20-28GEN-02



Drop Shaft Design - Former Tetrosyl Site, Bevis Green, Bury
Plans A - A and B - B

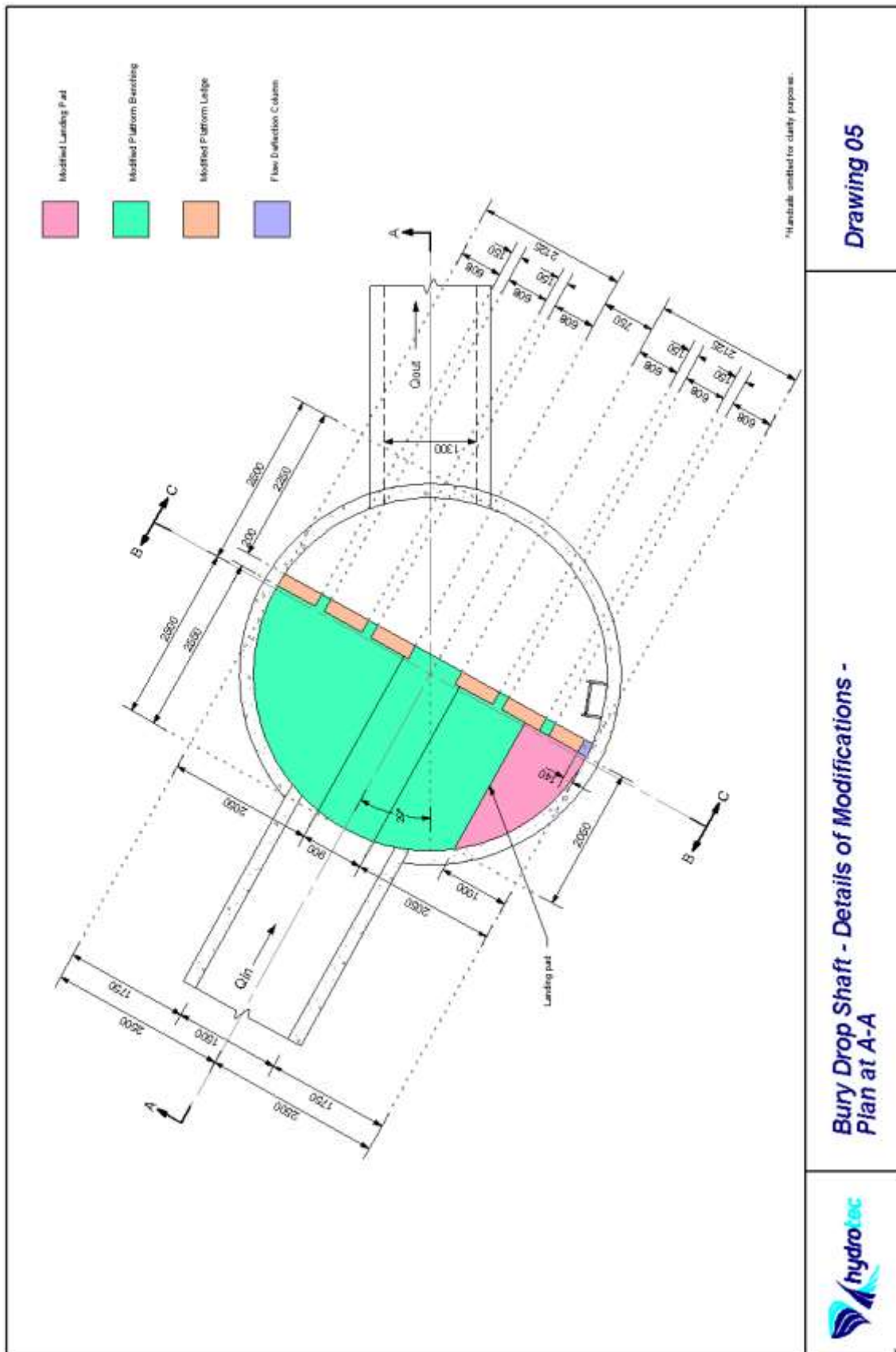
Drawing 02

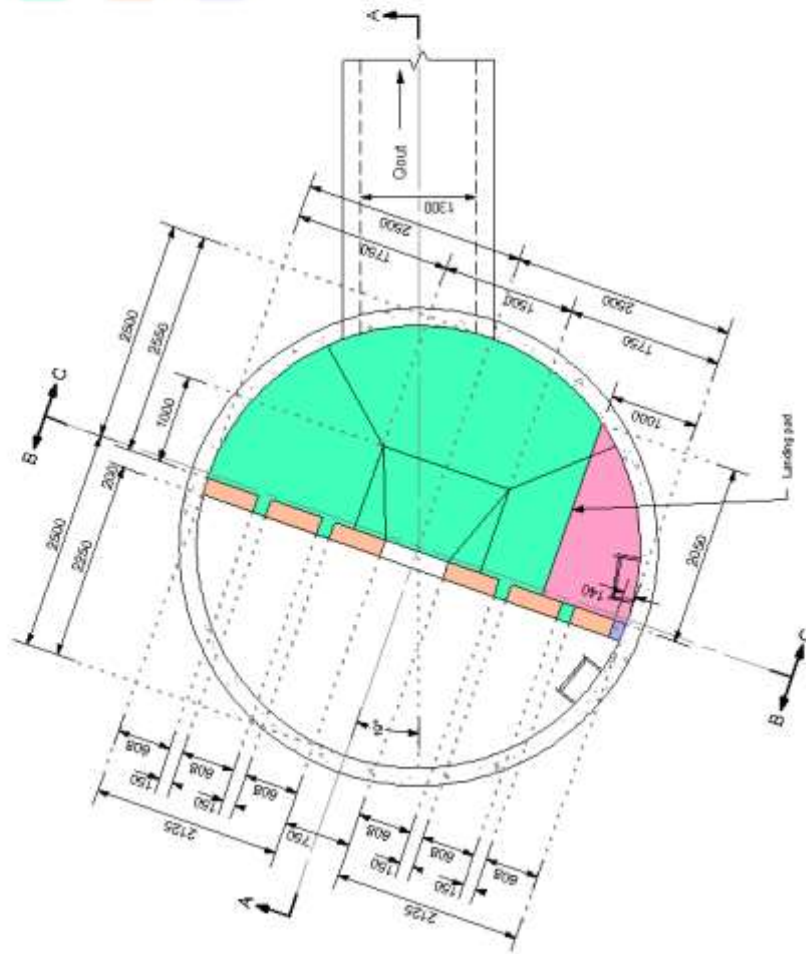
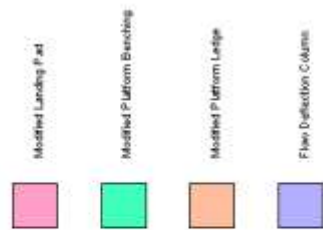


**Drop Shaft Design - Former Tetrosyl Site, Bevis Green, Bury
Plans C - C and D - D**



**Drop Shaft Design - Former Tetrosyl Site, Bevis Green, Bury
Plan E - E**

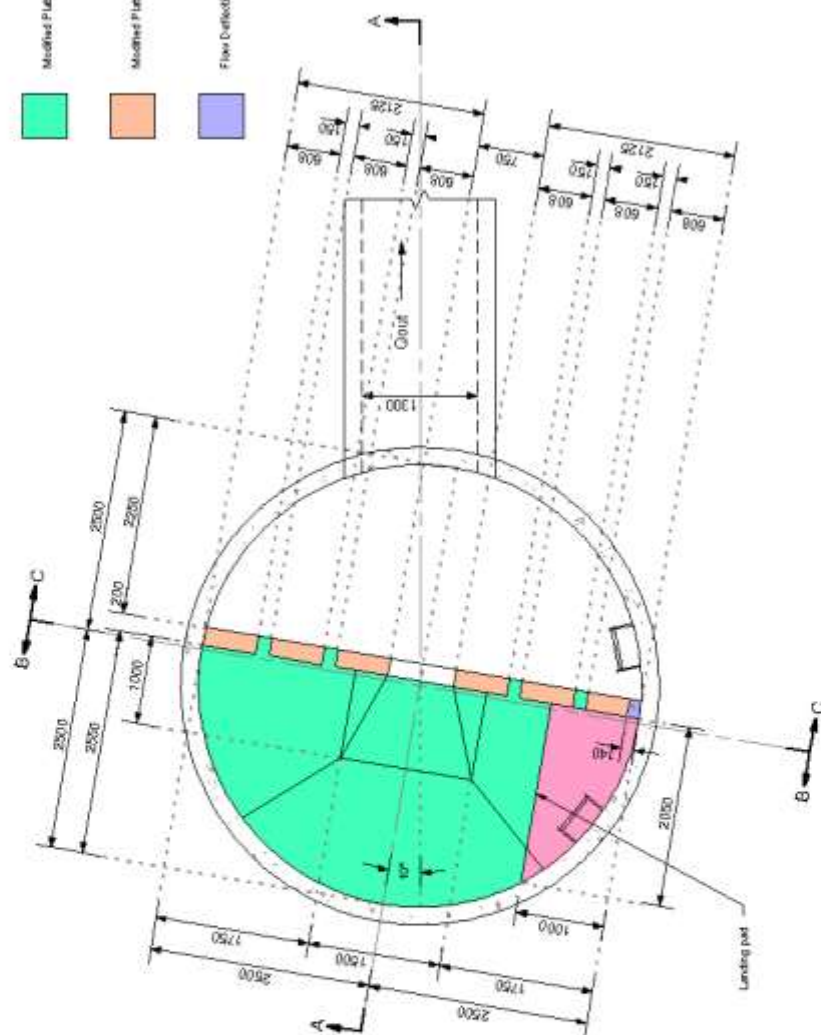


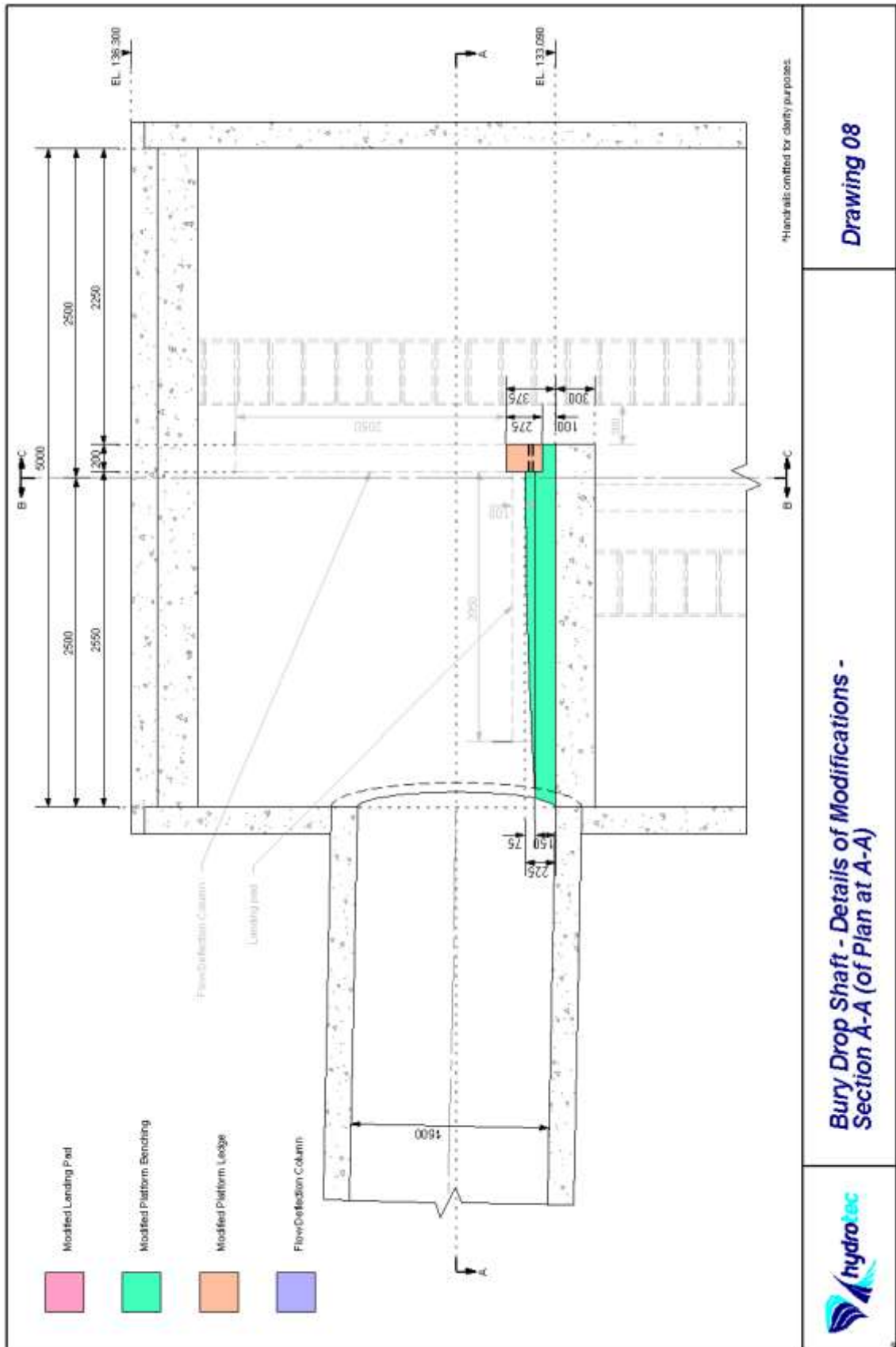


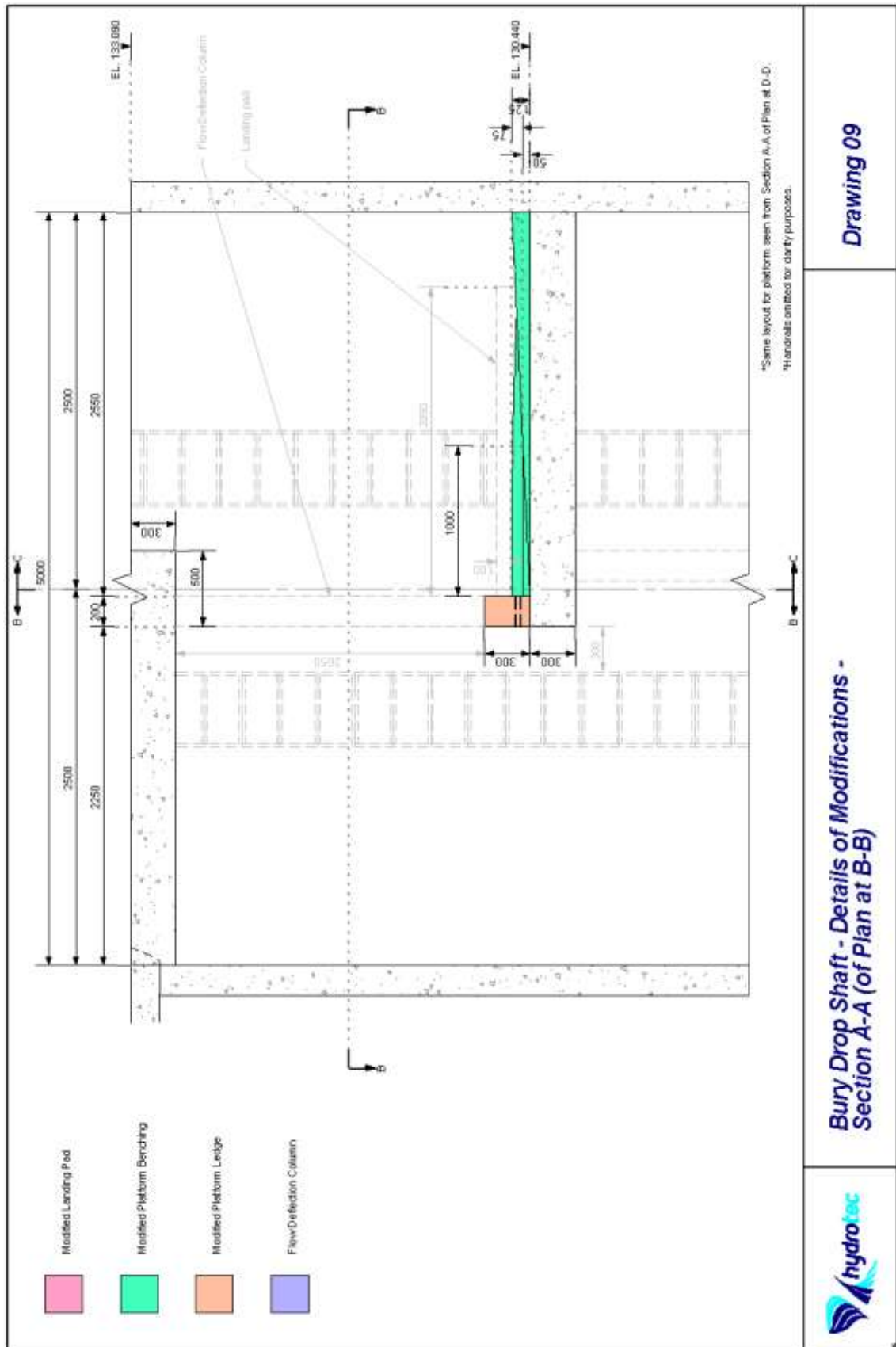
* Same layout for platform seen from Plan D-D.

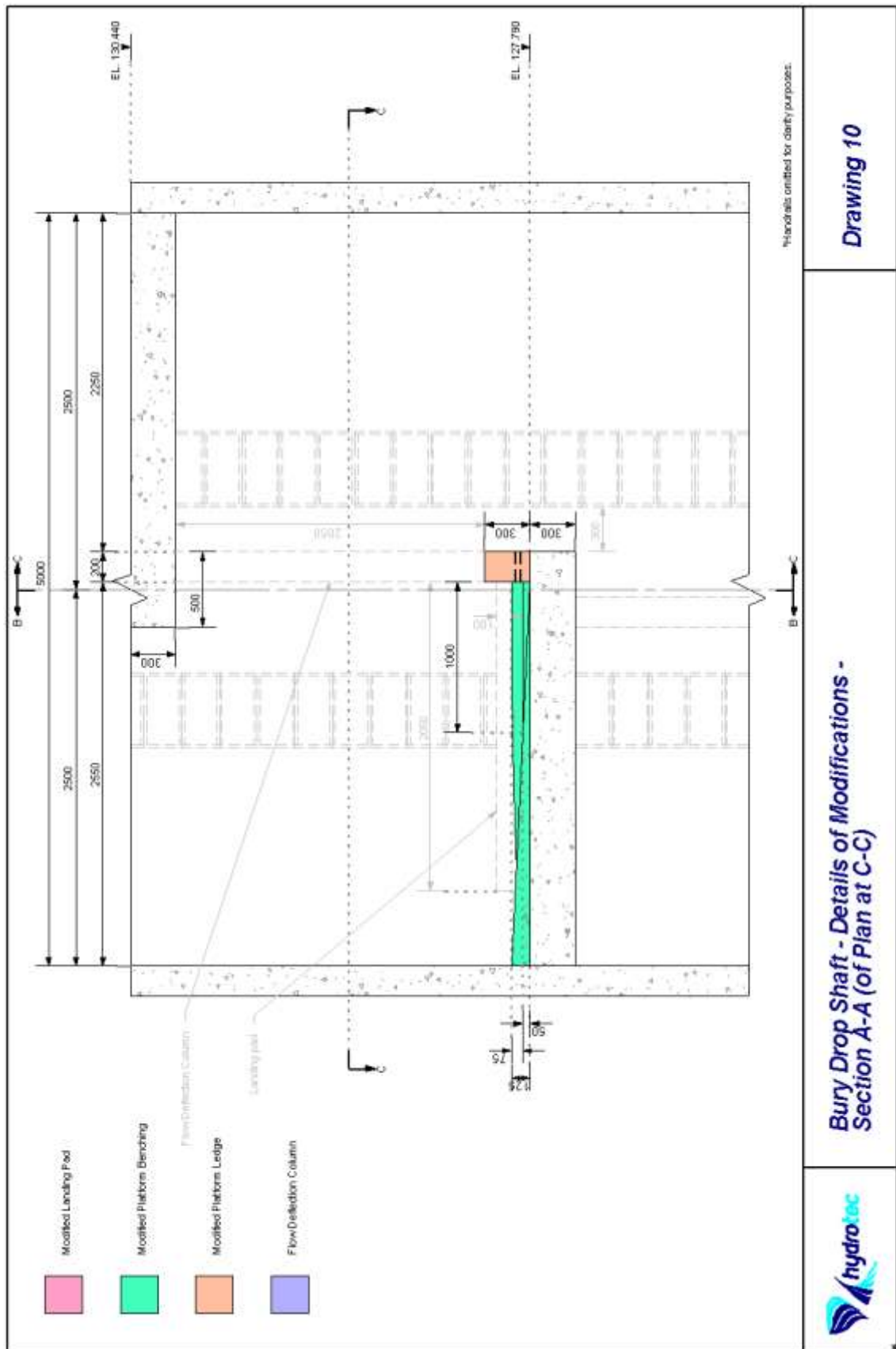
**Bury Drop Shaft - Details of Modifications -
Plan at B-B**

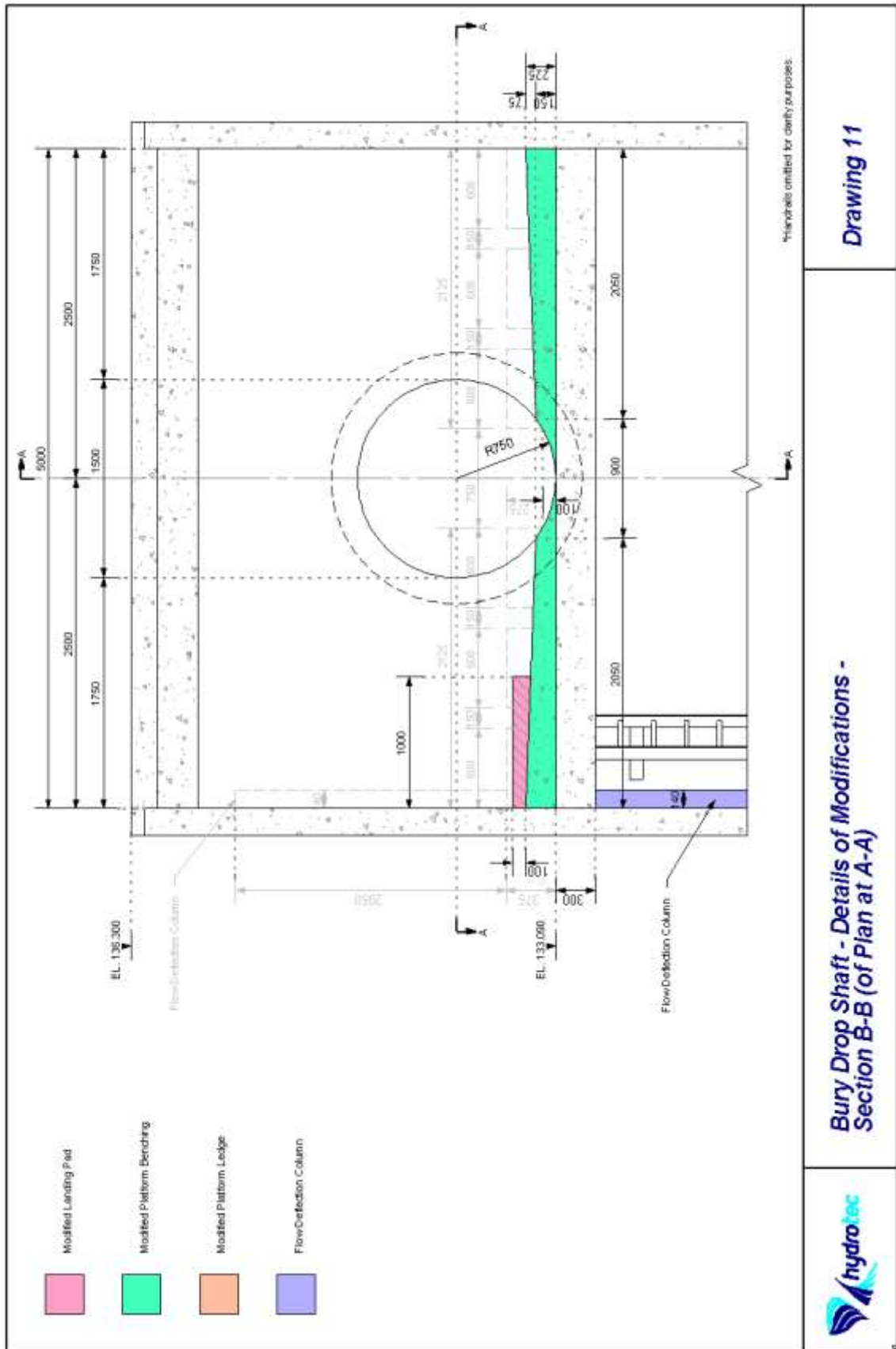
Drawing 06

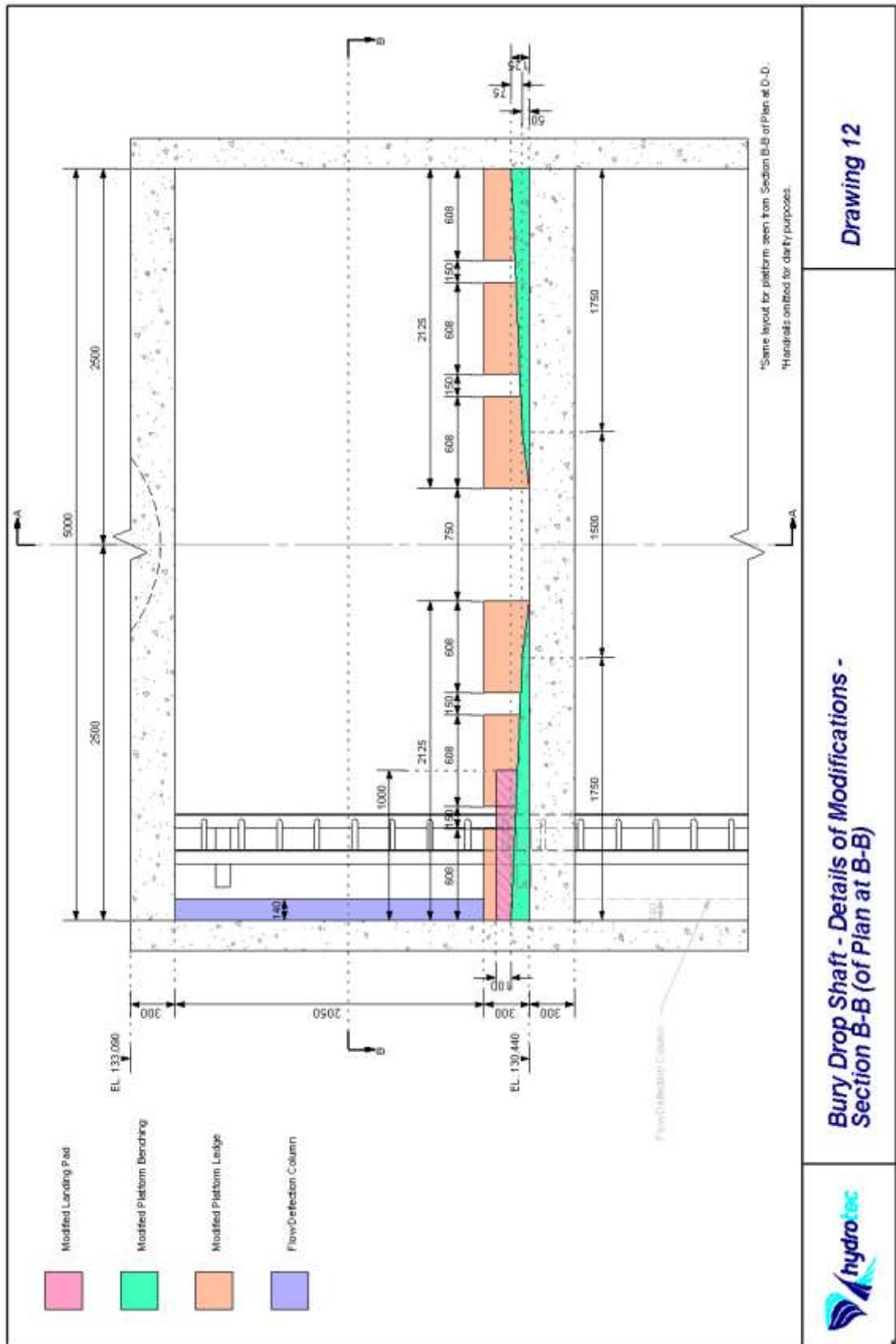


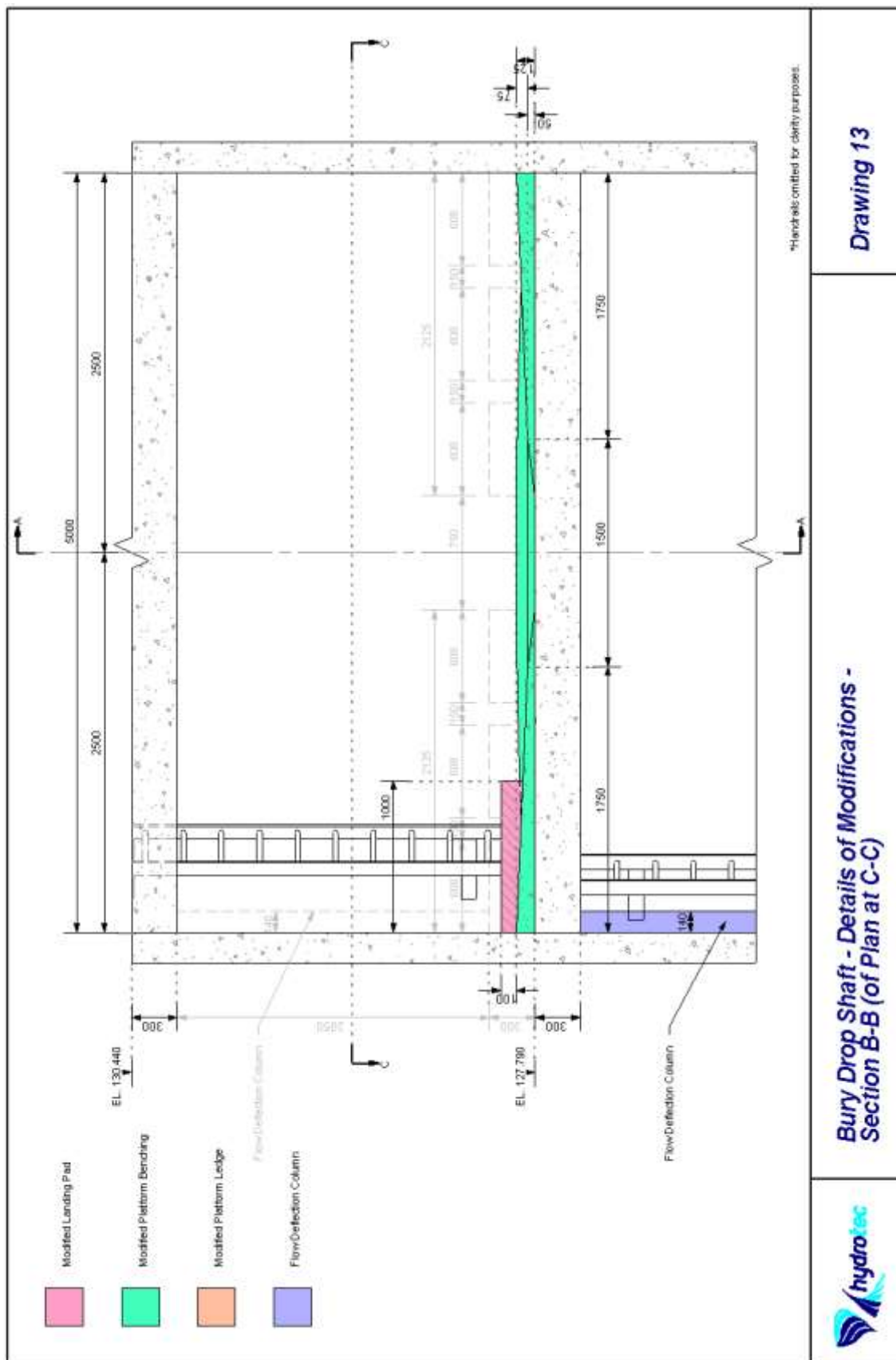


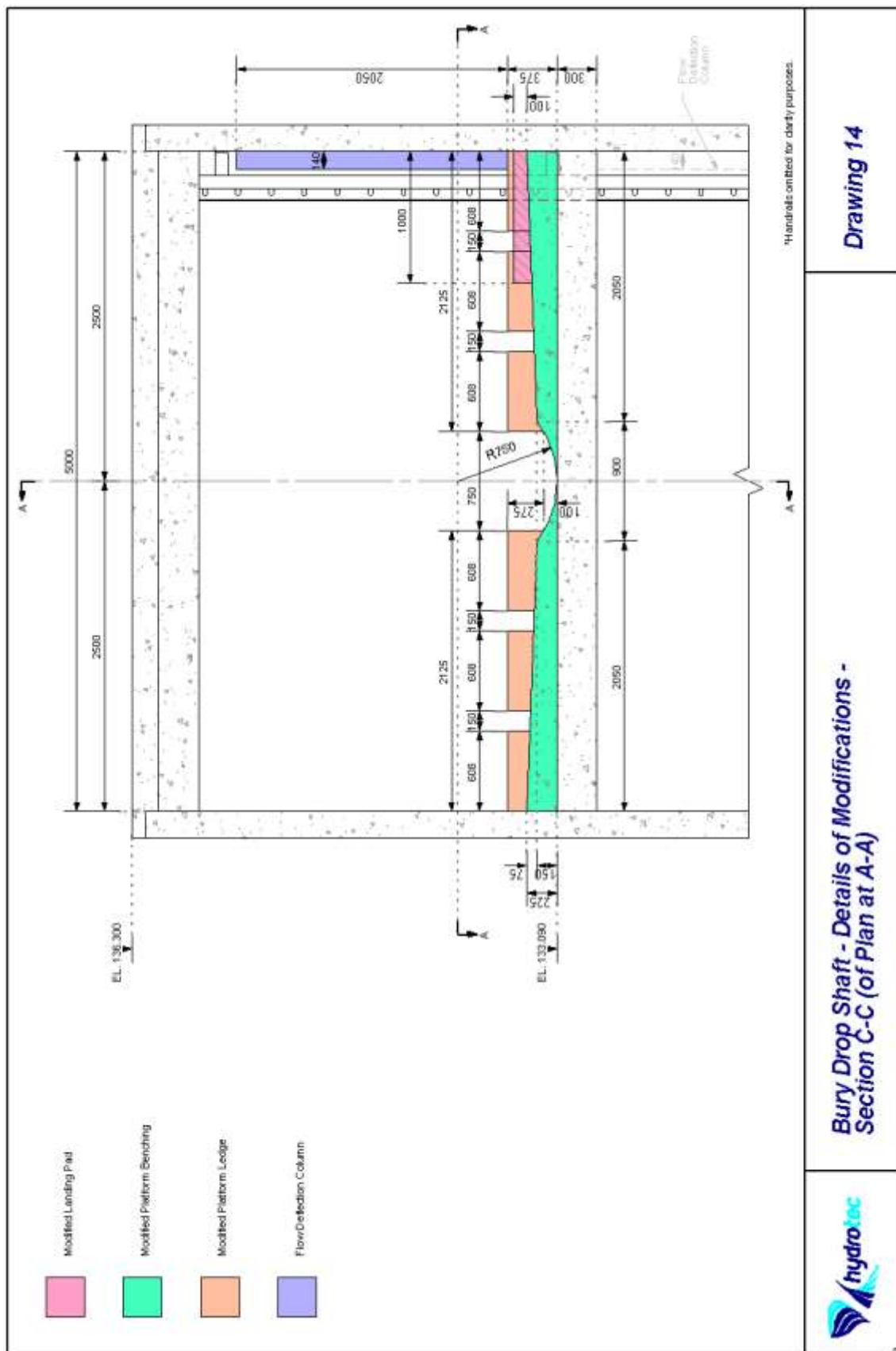


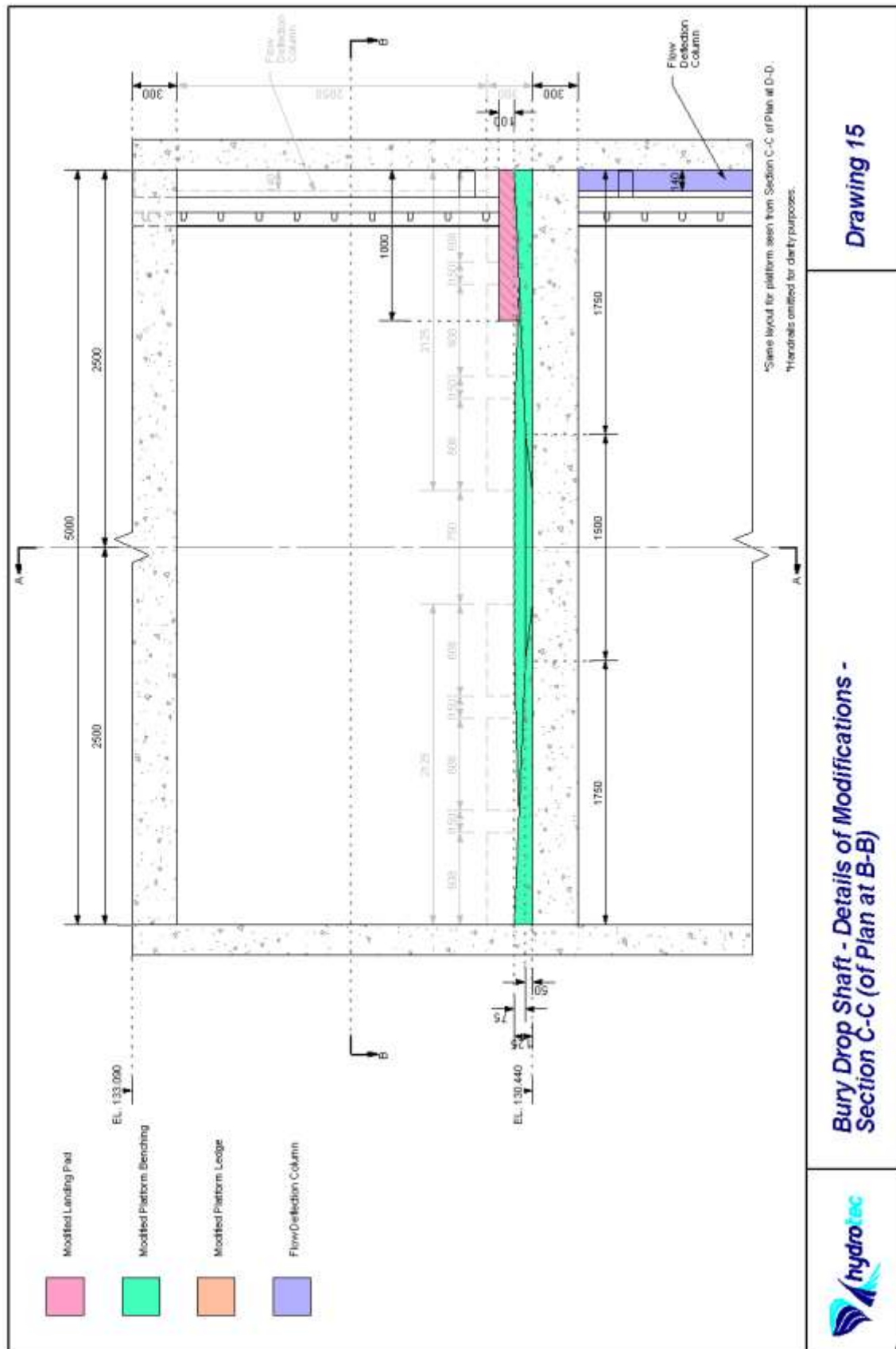


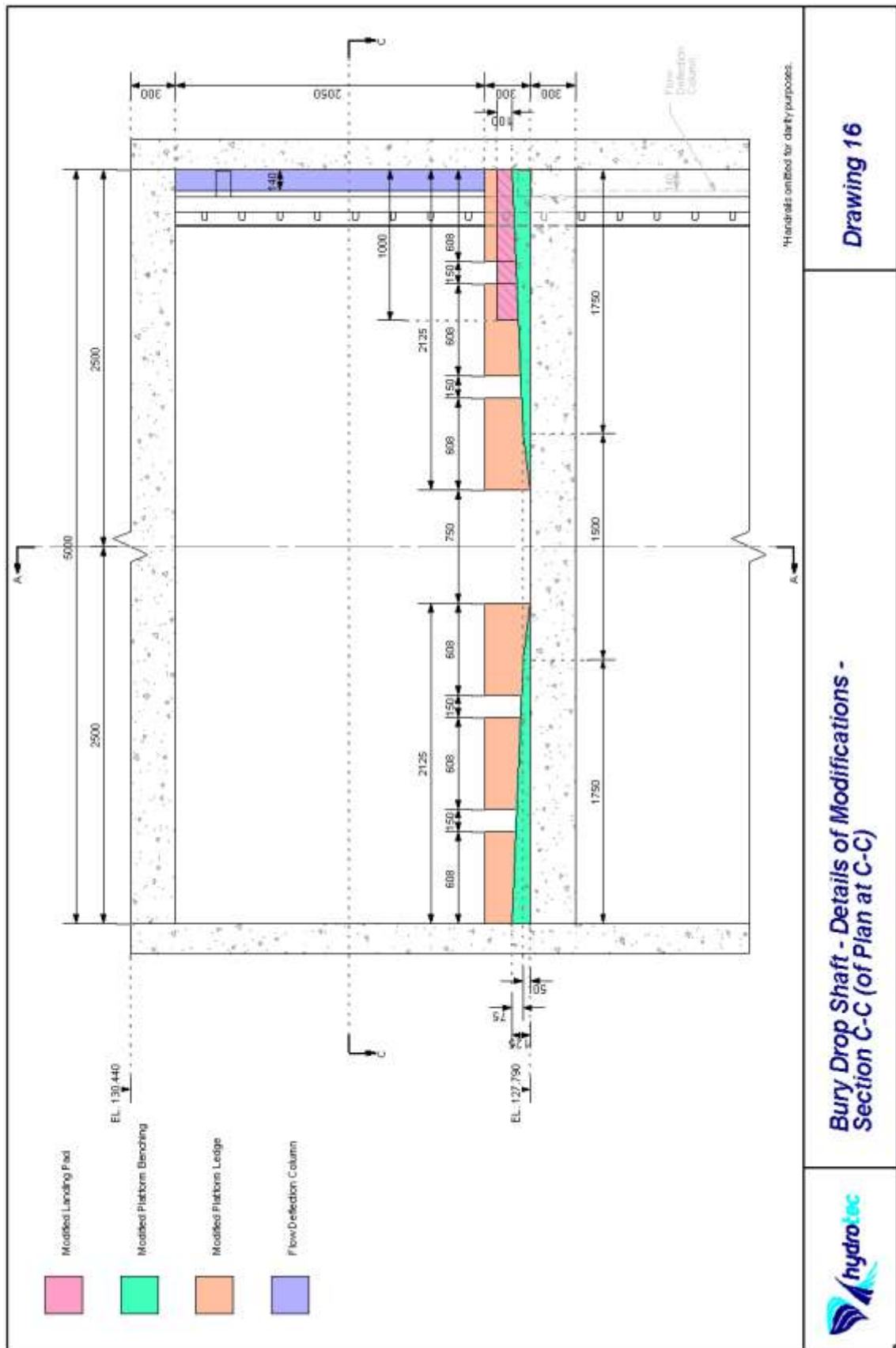












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Contract	Bevis Green
Client	Barratt Homes
Day, Date and Time	27/11/2020
Author	Emma Wright
Reviewer / Sign-off	Matt Hemsworth
Subject	Technical Note

1 Introduction

JBA Consulting was commissioned by Barrett Homes in 2020 to undertake a hydromorphology assessment and conceptual design for the watercourse at the Former Tetrosyle site, Bevis Green, Bury.

1.1 Purpose and scope of study

1.2 Study area

Figure 1-1 displays the site location and the catchment area of the watercourse, which is unnamed and is a tributary of Pigs Lee Brook. The watercourse rises on the southern edge of Harden Moor near Nangreaves and flows primarily south towards the site. The outflow from the reservoir is controlled by a weir in which the channel is culverted and then flows into open channel before discharging into Pigs Lee Brook.



Figure 1-1: Site Location

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2 Desk Based Assessment

2.1 Overview

A desk-based assessment has been undertaken for the catchment of the watercourse. This incorporates nearby main rivers and aims to provide understanding on catchment controls that influence hydromorphology on a local scale. The assessment has utilised available information including historic maps, aerial imagery, and LiDAR data.

2.2 Geology and land use

According to the British Geological Survey (BGS) mapping the watercourse and its upstream catchment is underlain by sandstone and are overlain by till superficial deposits, which are glaciogenic in origin.

The land surrounding to the south-west of the site is heavily urban with residential and commercial buildings. The land use to the north and within the watercourse's catchment is rural with localised areas of small woodland and agricultural land.

2.3 Flow regime

There are no flow gauges present on the watercourse, the closest gauge available is located approximately 3.0km downstream on the River Irwell. The difference in catchment characteristics, namely size, mean the flow gauge (69044 – Irwell at Bury ground) cannot be used to inform the flow regime of the unnamed watercourse.

The watercourse is not gauged; however, in the upper catchment it is characterised with steep topography, which is likely to produce rapid delivery of storm runoff to the watercourse. The urbanised nature of the catchment around the site will further contribute to this. During heavy rainfall events there is the potential for energy to be concentrated over a relatively short time frame, leading to elevated erosion rates in certain reaches.

2.4 Water Framework Directive Status and Water Quality

The unnamed watercourse falls within The Irwell (Rossendale STW to Roch) (ID GB112069064620) which is located within the Croal Irwell Operational Catchment. The waterbody is designated as 'heavily modified'. In the 2016 Cycle 2 assessment, the waterbody was classified as Moderate. There are several reasons why the waterbody is not achieving 'Good' status including both diffuse and point source pollution impacting upon phosphate in the watercourse and local ecology (macrophytes and phytobenthos (combined) and invertebrates).

2.5 Flood Risk

Figure 2-1 displays the Environment Agency Flood Zone Mapping across the site, illustrating that the site is located within Flood Zone 1. Downstream of the site where the watercourse converges with the River Irwell is in Flood Zone 3 which follows the route of the existing watercourse. To the north of the site the Cheesden Brook is within Flood Zone 2 and 3 which follows the route of the watercourse except to the north of the site the is flooding of the floodplain, however, this does not impact upon the unnamed watercourse of the site.

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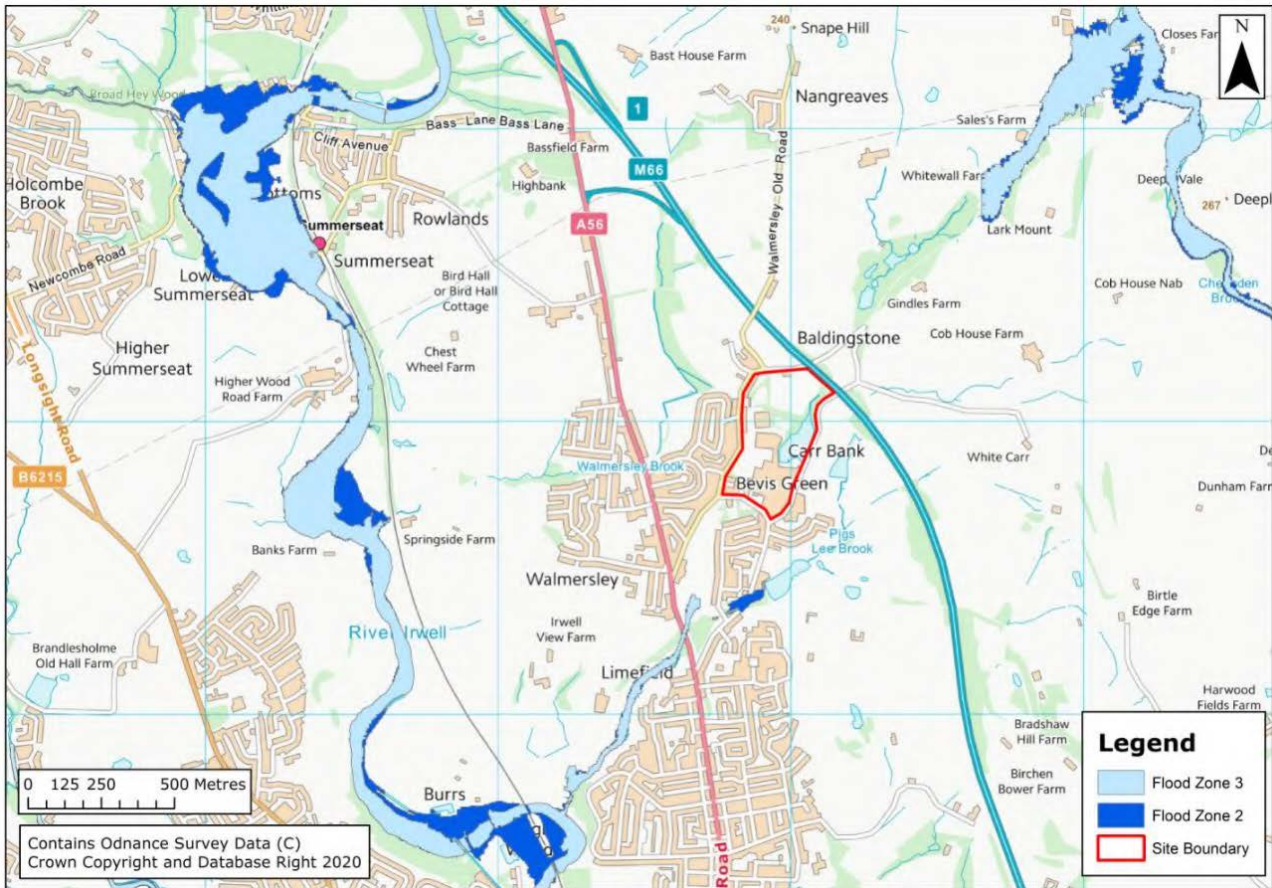


Figure 2-1: Current Flood Risk

2.6 Historic Trend Analysis

A desk-based review of historic maps has been undertaken to develop a historic chronology of the watercourse in response to channel modification over time. This information informs the current state of the channel through the identification of current and historic pressures influencing the watercourse. The assessment can therefore be utilised to predict how the riverine system will potentially develop under different scenarios in the future.

By reviewing the historic Ordnance Survey mapping it is apparent that the watercourse has been altered over time. The first mapping available from 1800, shows the reservoir and several weirs on the watercourse. It is thought the reservoir was originally constructed to supply water to the cotton mill.

Prior to the construction of the M66 the watercourse displayed some sinuosity upstream of the reservoir. The motorway in Bury was constructed between 1973 to 1978, during which the watercourse was straightened. The downstream extent of the watercourse connecting to Pig Lees Brook is entirely culverted, the exact date of which the culverting took place is unknown due to unclear mapping. Upstream of the site the watercourse was modified between 1930 and 1955 with the installation of three ponds controlled by weirs to maintain the water depth. These weirs are still present today.

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3 Walkover Assessment

3.1 Introduction

A walkover assessment was completed on the 15th July 2020 by Rebecca Thrower (geomorphologist) and Emma Wright (geomorphologist) of the watercourse through the site and upstream of the site boundary. The purpose of surveying the watercourse upstream of the site was to identify analogue reaches to gain an understanding of the geomorphic conditions of the river when not confined to culverts.

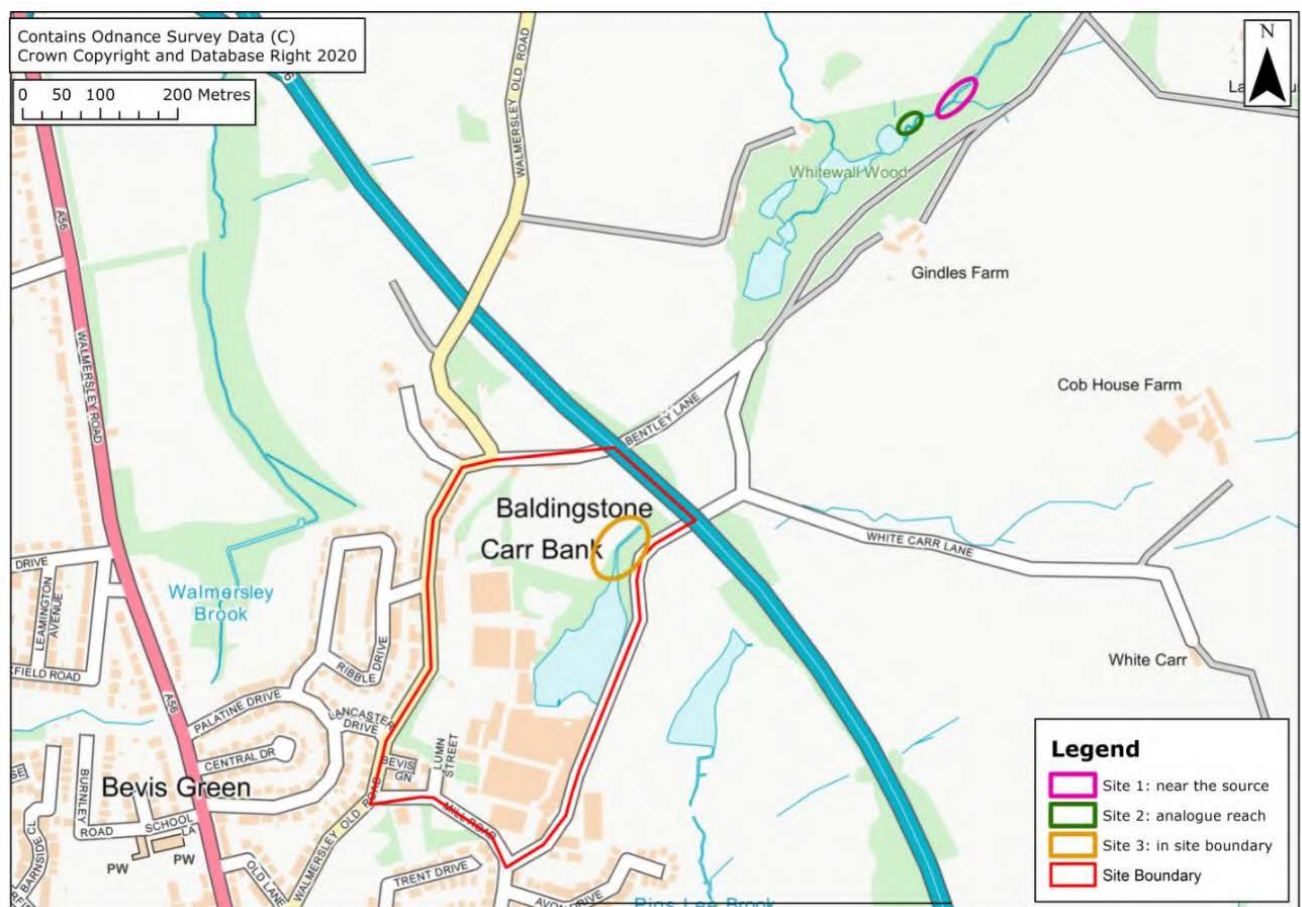


Figure 3-1: Survey Locations

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3.1.1 Upstream of the site: near the source

Upstream of the site (approximately 0.6km) the watercourse is a single thread river that flows through an alluvial channel. At this point in the catchment the watercourse is confined to a steep valley and displays both boulder cascade and cobble step pool morphologies (Figure 3-2). The channel is very narrow, approximately 0.5m wide, and steep with coarse bed material (gravels, cobbles, boulders) indicating an active supply of sediment to the reach. Some bank erosion was evident immediately downstream of the confluence between the two watercourses, where the channel gradient reduces and sediment has been deposited (Figure 3-4). The river is confined by the steep valley and therefore limited floodplain is evident; however, downstream the valley floor widens and this creates some floodplain. The confinement of the valley and lack of floodplain for flows to spill onto concentrates energy in channel and increases the size of the material that can be transported.

The riparian zone consists of a broadleaf woodland providing a large supply woody debris and leaf litter into the channel (Figure 3-4).



Figure 3-2: Upstream steep morphology

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This reach could be used to define characteristics of the new channel if the gradient is to be of similar steepness. Figure 3-3 depicts the long section throughout the site but specifically the step-pool sequences that occur within the steeper gradient. This can be used as an analogue to determine the in-channel features within an area of steeper gradient.

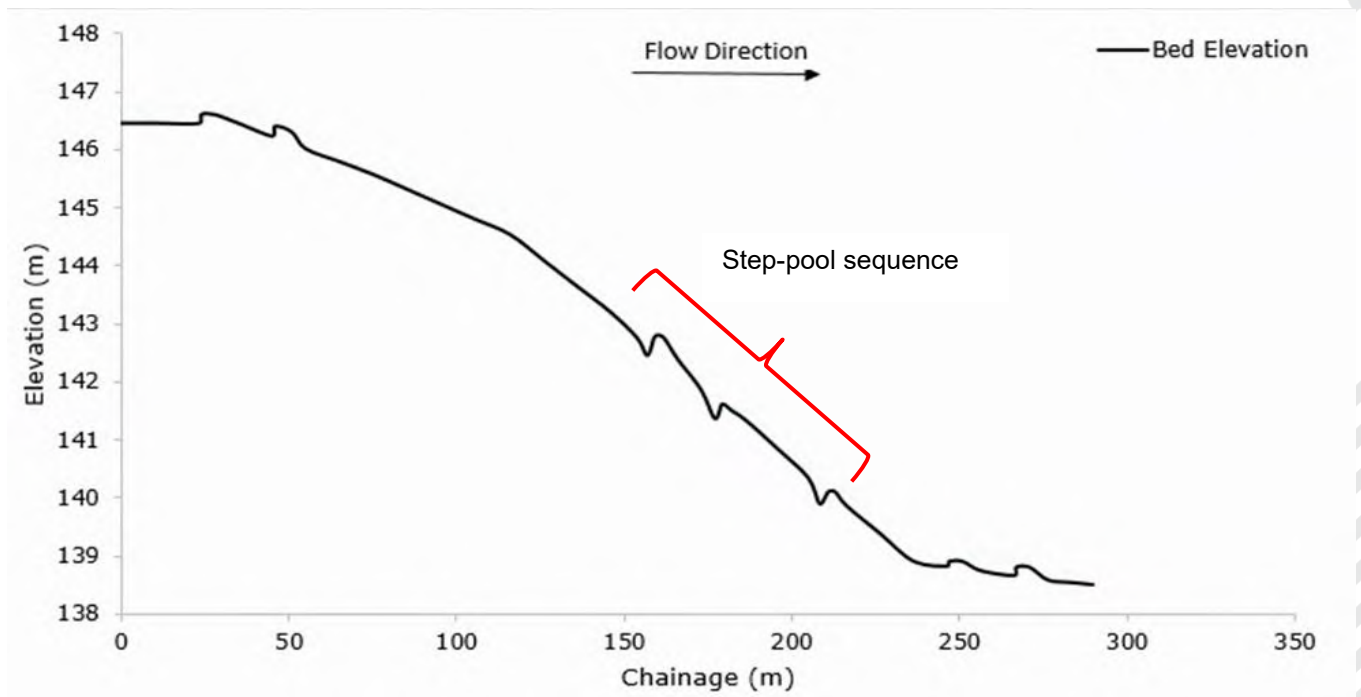


Figure 3-3: Long section to demonstrate the step-pool (step-pool) sequence

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Figure 3-4: Large woody debris supplied into the channel with bank erosion

3.1.2 Upstream of site: Reduced channel gradient

This reach displayed a distinct change in the morphology from the previous reach due to a notable change in gradient and slight widening of the valley floor. The reduced gradient has allowed for a meandering pool riffle morphology. In-channel velocities have reduced as a result of a shallower gradient and enabled greater sediment deposition. Smaller sized coarse sediment is seen throughout with the presence of fine gravels and small cobbles. Small gravels and cobbles are stored in short exposed gravel bars on alternative sides of the channel. These features are typically 0.5m wide and range in length from 1m to 8m. Immediately downstream of the bars, riffles have formed over the submerged part of the bar. Riffles are associated with shallower water depths, locally steeper gradient and increased in-channel velocities due to increased turbulence. These contrast to the deeper areas of channel that are characterised by slower moving pools, which during low flow conditions are typically low energy areas of the channel. Within this reach the river is well connected to its floodplain with shallow bank gradients and heights, which were typically 0.3-0.5m above bed level and at the time of the walkover 0.2-0.25m above the water level (Table 3-2).

Figure 3-5 shows a basic site sketch with the key hydromorphic processes of the reach illustrated. It was determined that this reach and the upstream reach would be used as analogue sites to inform the key hydromorphic features within the new channel construction at the site, therefore incorporating both the higher and lower gradient channel morphologies.

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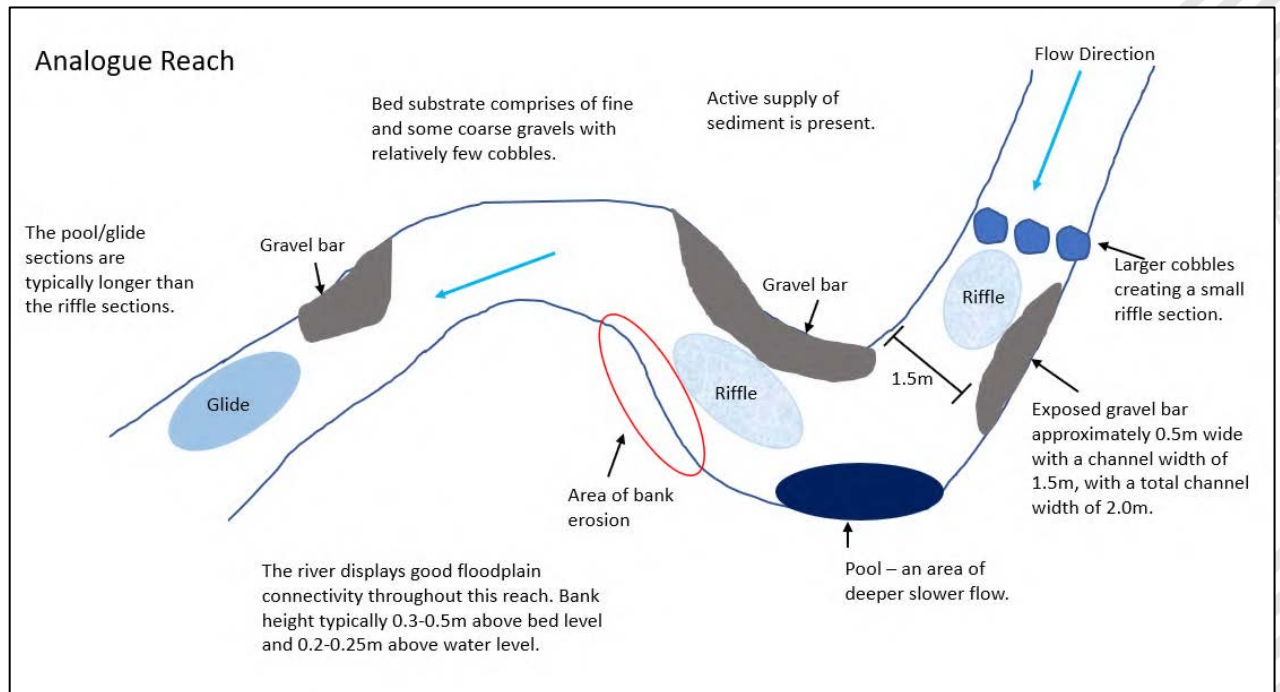


Figure 3-5: Site sketch of the reach to be used as the analogue reach (for lower gradient reaches)

Table 3-1 shows the approximate channel widths, widths and lengths of bar features estimated from the analogue reach, based upon site photographs from Table 3-2 below.

Site Photo	Channel Width (m)	Channel Width (m)	Bar width (m)	Bar length (m)	Riffle length (m)	Bank height from bed (cm)	Height of Bar feature	Connected to floodplain
1	2	0.5	1.5	5	2	0.4	0.2	Yes, particularly on the left-hand bank
2	1	0.5	0.5	1.5	1.5	0.4	0.2	Yes
3	3	0.5	2.5	8	2	0.5	0.3	Yes
4	1	0.5	0.5	1	-	0.7	0.3	No
5	2	0.5	1.5	2	0.75	0.5	0.2	Yes

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Table 3-2: Site photographs to be used as an analogue reach (for the lower gradient reaches)

Photograph 1



Photograph 2



Photograph 3



Photograph 4



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3.2 Indicative bar features

A series of indicative cross sections were developed from using the channel measurements from the site visit. These sections provide a guide to the development of the cross sections.

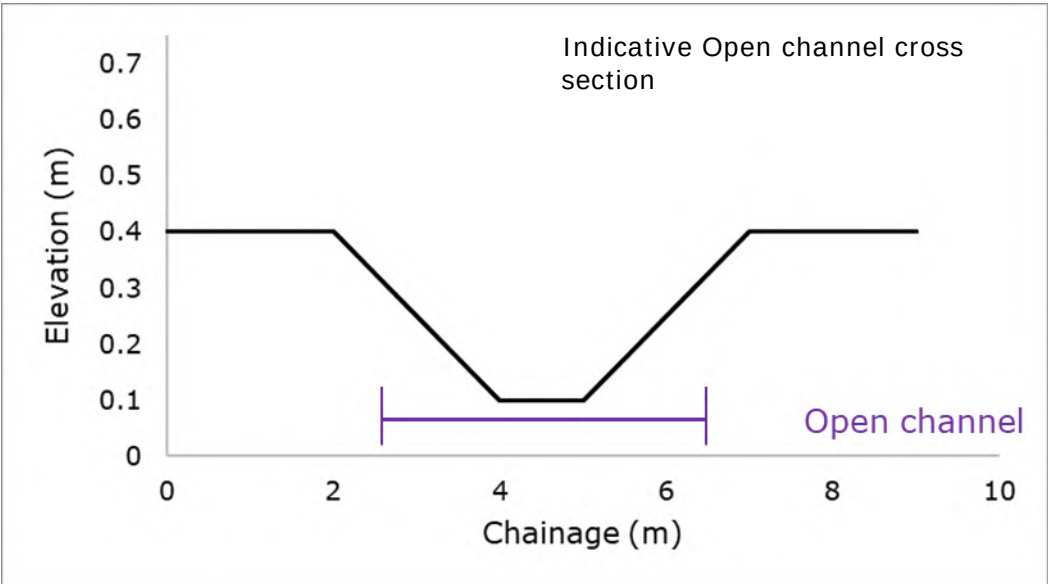


Figure 3-6: Indicative open channel section

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Table 3-3: Indicative open channel section			
Open channel - Option 1		Open channel - Option 2	
Chainage (m)	Elevation (m)	Chainage (m)	Elevation (m)
0	0.4	0	0.4
2	0.4	3	0.4
4	0.1	4.25	0.1
5	0.1	5	0.1
7	0.4	6.25	0.4
9	0.4	9.25	0.4

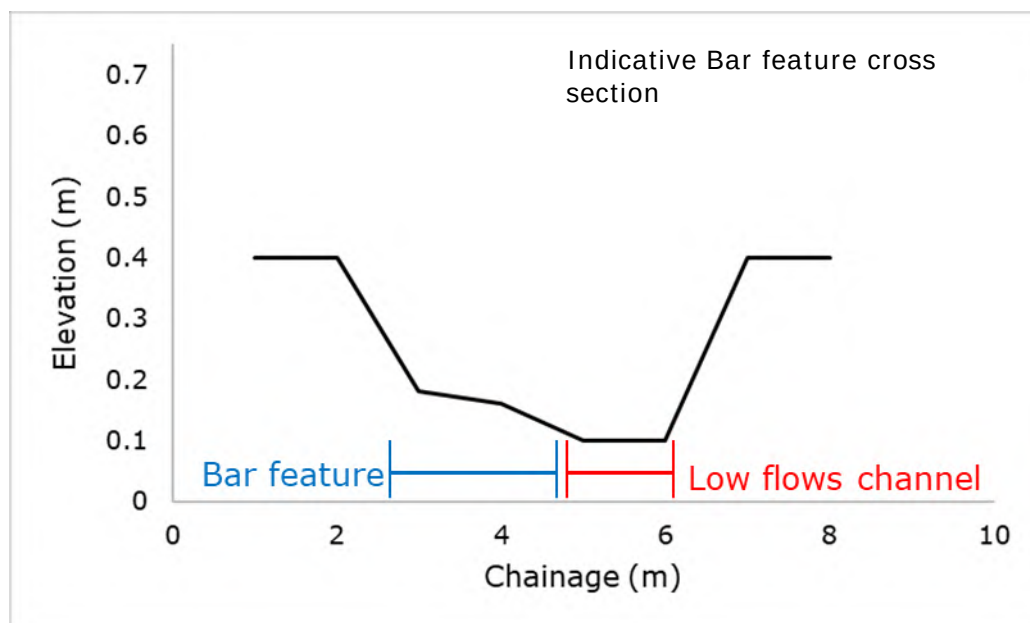


Figure 3-7: Indicative low flows channel section

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Table 3-4: Indicative open channel section

Bar feature - Option 1		Bar feature - Option 2	
Chainage (m)	Elevation (m)	Chainage (m)	Elevation (m)
0	0.4	0	0.4
2	0.4	2	0.4
3	0.18	3	0.2
4.5	0.16	5.5	0.15
5.5	0.1	6	0.1
6	0.1	6.75	0.1
7	0.4	7.75	0.4
10	0.4	9.75	0.4

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4 Low Flows Modelling

4.1 Introduction

This section of the report describes the modelling approach that was undertaken to represent the in-channel features for the new open channel watercourse. The purpose of this modelling was to determine the function of the morphological features and to understand what influence they will have on sediment transport processes, to inform the concept design for the new channel.

4.2 Methodology

4.2.1 Flow Duration Curve

A catchment for the unnamed watercourse has been delineated from the FEH CD-ROM and run through LowFlows 2 software, developed by Wallingford HydroSolutions (WHS). This software estimated the natural flow regime at the location based on a database of hydrologically similar catchments. A Flow Duration Curve (FDC) was developed and used to determine the low flows for the model, key exceedances are shown in Table 4-1.

Table 4-1: Summary of Flow Duration Curve	
Exceedance	Discharge (m ³ /s)
Q10	0.104
Q20	0.065
Q50	0.027
Q70	0.017
Q95	0.007

4.2.2 Baseline Scenario

An existing hydraulic model, supplied by Cole Easdon, was used to develop the concept design for the new channel. The supplied hydraulic model consists of one of the existing configurations with the watercourse culverted underneath the buildings and a post development model with proposed ideas for an open channel.

4.2.3 Post Development Scenario

The low flows model was developed through using the Cole Easdon post development model. To adapt the model for us, several cross sections were modified to incorporate bar and step-pool sequence to represent similar widths and attributes as observed within the analogue reach.

4.3 Model representation

Table 4-2 below depicts how the features were represented in the model, including bar and step-pool sequences.

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Table 4-2: Model Representation	
Feature	Representation
Bar feature	The channel geometry was altered to incorporate bar features into the channel. The length of the bars was determined using similar lengths as observed in the 2nd, pool riffle, analogue reach. The height of the bar features was determined by the water levels, allowing the majority of the bar to be exposed at Q70 whilst submerged at higher flow exceedances, for example Q10.
Step -Pool feature	The channel geometry was altered to incorporate a deeper channel to act as a pool. The depth of the section was increased from the original however the banks remained the same. Whilst immediately downstream of the pool the channel geometry was altered to incorporate a cross section to represent a step, in order to represent some of the features observed in the 1st analogue reach where the gradient is steep. The bed level was raised within this section to provide increased velocities through this section.

4.4 Assumptions and limitations

The hydraulic modelling undertaken for this study is accompanied by some assumptions and limitations, which are discussed below:

- The in-channel roughness values were all set at 0.05, which was considered appropriate to hydraulic conditions observed in the analogue reaches.
- The model was truncated at the weir at Mill Road downstream of the site, as it was anticipated that this structure would not instigate any backflow (due to an 8m drop in bed levels either side of the structure) which may influence on upstream water levels. This was therefore seen as an appropriate location to truncate the model. The model was additionally truncated upstream of the attenuation pond as it was anticipated that this would control the flows downstream into the new channel.

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5 Results

5.1 Introduction

Within this section of the report the results outputted from the post developed hydraulic model are discussed. Table 5-1 reports the model node name and whether it is classified as an open channel section or a bar feature (the difference of which are shown in the next section). These node names are used in the sections below to discuss the velocity, entrainment and shear stress.

Table 5-1: Model node labels	
Cross section type	Mode Node Label
PLB352	Open channel
PLB387	Bar feature
PLB427	Open channel
PLB468	Open channel
PLB508	Open channel
PLB508_PO1	Pool (lowered bed level)
PLB508_PO2	Open channel
PLB508_R1	Step (raised bed level)
PLB508_R2	Open channel
PLB548	Open channel
PLB588	Open channel
PLB628	Open channel
PLB641	Open channel

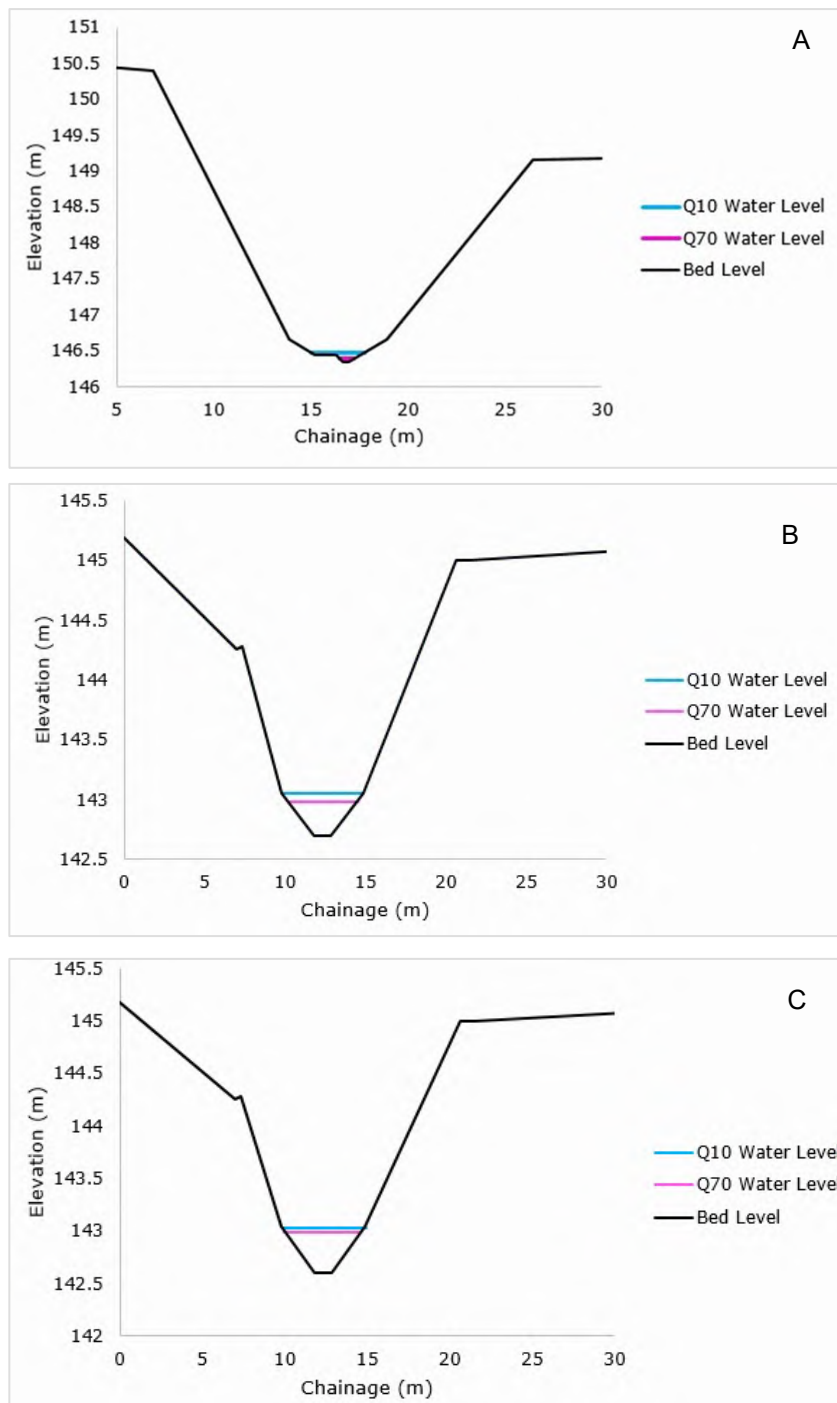
5.2 Water Levels

Table 5-2 depicts a cross sections within the model including both the open channel, with the bar features and in areas of step-pool sequences. The water levels within the bar cross section (A) shows that within a higher flow exceedance (Q10) the bar features are fully submerged, whilst at a lower exceedance they are exposed as per the design criteria. The cross section that contains a pool (B) shows the water levels are greater because the channel is narrower. Cross section (C) depicts the step has raised water levels compared to the open channel sections.

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Table 5-2: Model water levels



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5.3 Channel Velocities

Velocities throughout the site have been extracted from the hydraulic model. Figure 5-1 shows the channel velocities with the open river channel sections without the presence of any in-channel features. The data shows that the gradient steepens, as shown in the long section in Figure 5-2, there is a correlated increase in the in-channel velocities. As the bed gradient become shallower these velocities reduce. It is anticipated to observe erosional process occurring in the steeper sections where the velocities are high, whilst where the gradient becomes shallower it is anticipated to observe a more depositional environment. The Hjulström curve depicts the relationship between the size of the sediment and the velocity that is required to either transport it, erode or deposit it. Within the Q10 flows the velocities range from 0.3-0.7m/s. The calculated entrainment values for the same event would be capable of entraining, transporting and deposition of small gravels between 6-38mm.

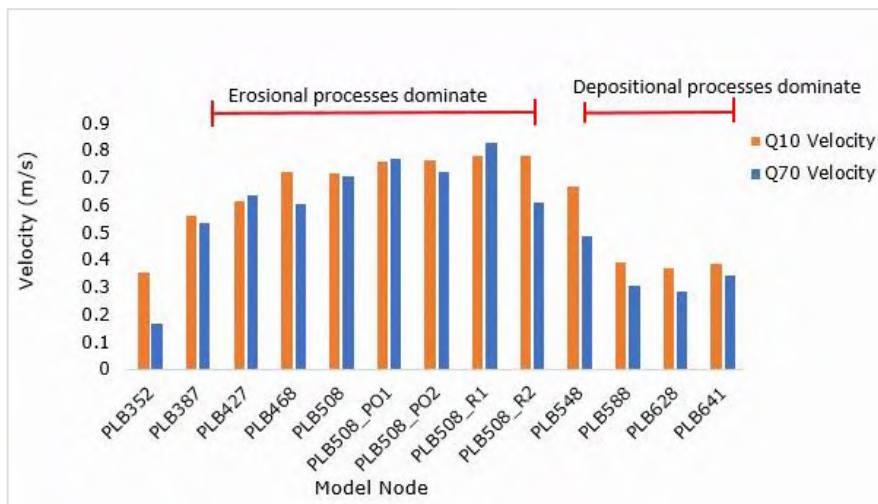
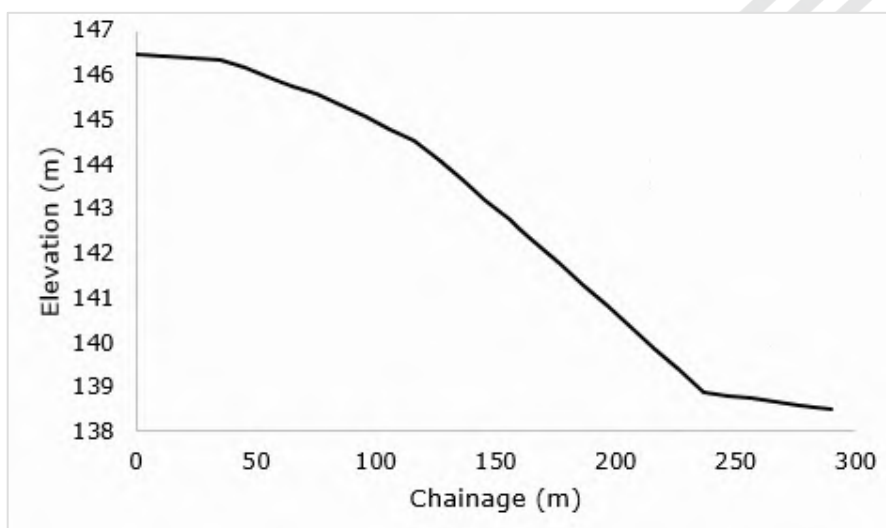


Figure 5-1: Velocities across the site (Baseline) sequence*

* *Note not to scale, total model length is 289m and the distance between the model nodes vary.



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Figure 5-2: Long section

5.3.1 In channel features present

The hydraulic model was developed to include the addition of a step-pool sequence within the stepper sections of the long profile to represent the bed conditions observed upstream from the site. These features have also been introduced to manage the high velocities shown in Figure 5-1, which could lead to a highly erosive channel if left designed as a uniform channel. In the shallower gradient areas at the start and end of the reach, a bar feature has been included to determine the influence this would have on the in-channel velocities.

Figure 5-3 shows the results of this scenario in terms of modelled velocities.

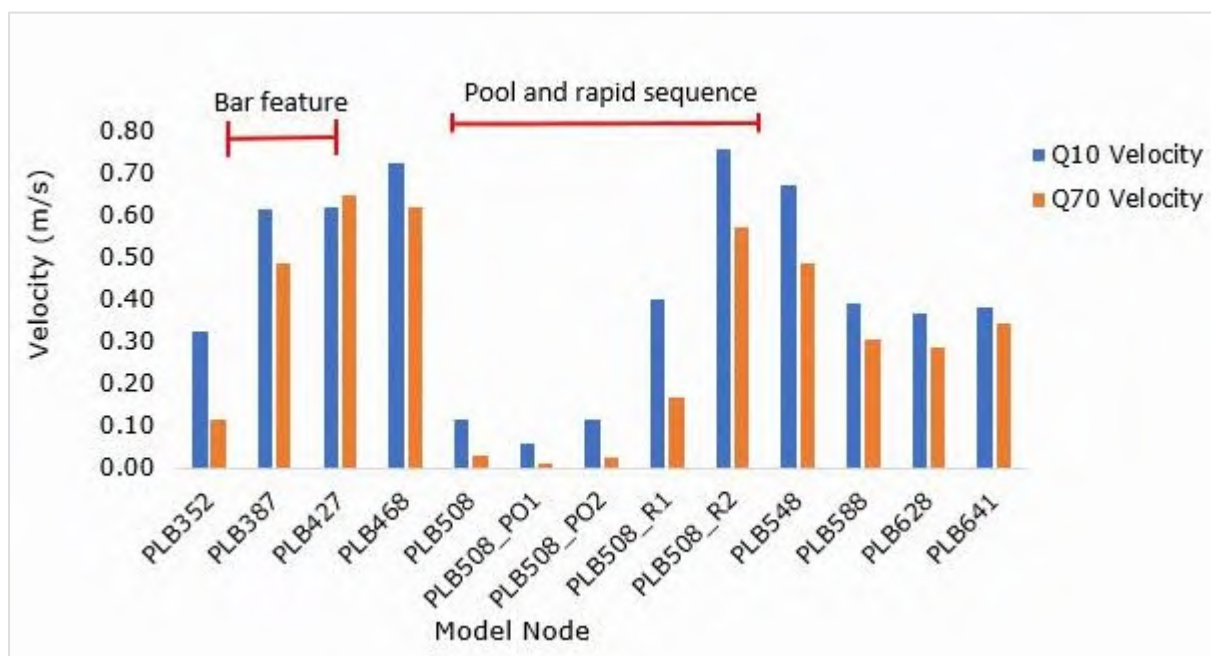


Figure 5-3: Velocities across the site (post development) sequence *

*Note not to scale, total model length is 289m and the distance between the model nodes vary.

Table 5-3: Comparison of velocity's		
	Q10 Velocity	
Model Node	Baseline	Post Development
PLB352	0.35	0.32
PLB387	0.56	0.61
PLB427	0.62	0.62
PLB468	0.72	0.72
PLB508	0.72	0.11
PLB508_PO1	0.76	0.06

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PLB508_PO2	0.76	0.11
PLB508_R1	0.78	0.4
PLB508_R2	0.78	0.75
PLB548	0.67	0.67
PLB588	0.39	0.39
PLB628	0.37	0.37
PLB641	0.38	0.38

Velocities throughout the site have been extracted from the hydraulic model. Table 5-3 (red text highlights an in-channel feature) shows a comparison of velocities between the baseline and the post development results during Q10 flows. It shows within the steeper gradient where the pool has been modelled there is a decrease in the in-channel velocities as a result of deeper water, with a substantial increase in velocities through the step-pool due to the steeper gradient associated with these features. For the Q10 flow results around the bar feature in the lower gradient area, the velocities are higher which can be attributed to the way the model calculates an average velocity for the cross section and in bar features, the flow depths over the bar are shallower than then open channel. In addition, there is a narrower flow channel which increases water depth attributing to higher velocities.

The variation in velocity between sections and between different flow events will help to maintain and provide diverse geomorphic conditions, significantly improving upon the existing culverted watercourse.

5.4 Sediment transport

The shear stress and entrainment values were determined for the modelled cross sections for the site, using the Shields equation. Entrainment is the value in which the maximum size of sediment can be transported during a particular flood or low flow event. Figure 5-4 depicts the entrainment values at both Q10 and Q70 flow exceedances for the post development. The graph shows that the typical grain size that is mobilised and entrained during Q10 flows ranges from 6-38mm, which represent flows that are exceeded only 10% of the time and are therefore relatively high flows (compared to other low flows), suggests the new channel would be capable of sustaining bedform development. This is regarded as a positive outcome as bedform development leads to diverse morphology and creates in-channel habitat, such as those observed in the analogue reaches upstream.

Velocity is used in the calculation of shear stress and so the results share similar trends. Figure 5-4 shows that typically there is a greater size of sediment which is entrained within the open channel compared to those with a bar feature present. This indicates that at Q10 flows, larger sediment would be entrained and transported through the open channel sections, which are deeper and slightly narrower than the overall channel with the bar features. Whereas at bar feature cross-section, the slight reduction in maximum grain size entrainment would mean coarser sediment is deposited. This enables bars to form as they are created through depositional processes. Immediately downstream of the bar features there is the potential for riffles to form as an extension of the bar and which would provide flow diversity within the channel.

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Immediately upstream of the modelled step-pool on the steeper section, there is a drop in the shear stress and entrainment associated with the pool feature, as a result in the reduced flow velocity. The modelled step-pool sequence is shown to reduce velocities overall when compared to the results without them in. This will help manage velocities in the reach which is seeking to mimic the high gradient slope observed in the headwater streams.

The recorded entrainment values outputted from the hydraulic model, is comparable to observations made on the walkover assessment of the analogue reach. Small gravels were observed typically within the open channel, with larger, coarser gravels identified on the exposed bar features. Whilst, further upstream in areas of steeper gradients step-pool sequences dominated observing very coarse bed material (gravels, cobbles, boulders) indicating an active sediment supply.

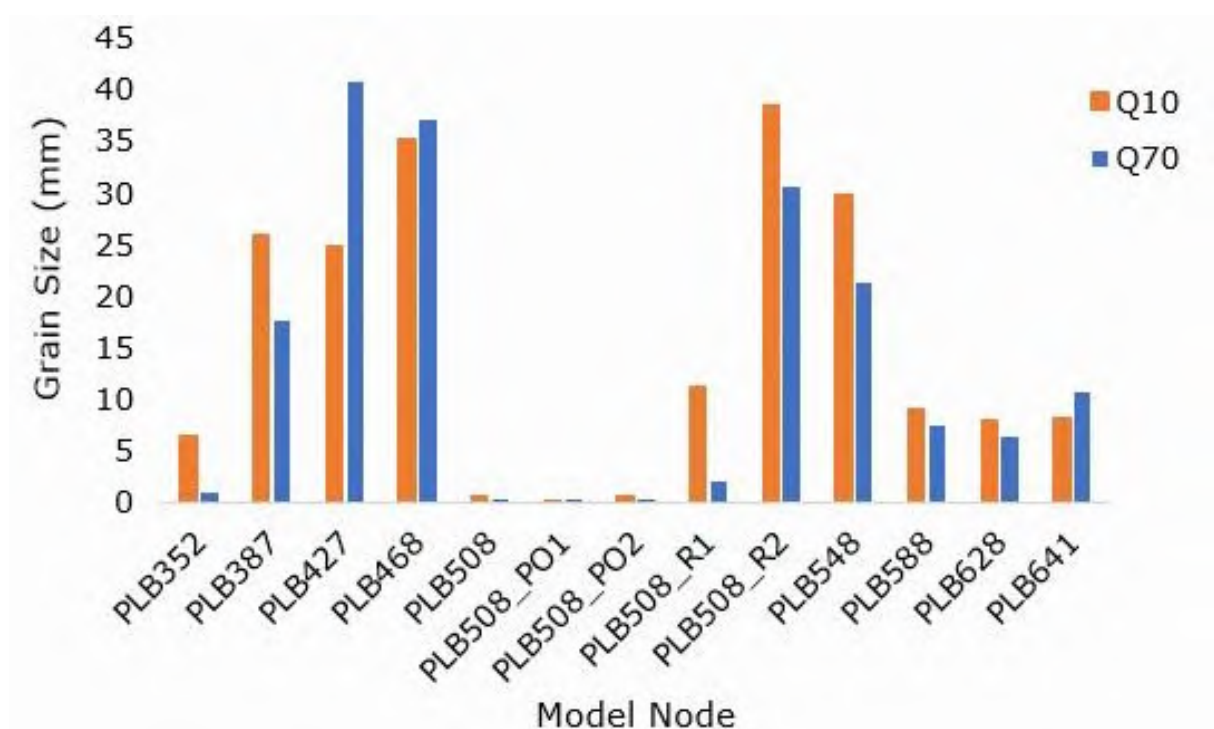


Figure 5-4: Entrainment values across the site*

*Note not to scale, total model length is 289m and the distance between the model nodes vary.

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5.5 Design risks

When designing a new channel or when considering modifications to an existing channel, there are a number of risks that need to be considered to develop an appropriate design. The following risks have been identified:

- Inappropriate sizing of the channel which could lead to a degraded system that does not support hydromorphic processes: To mitigate this risk analogue sites have been identified and used to inform the shape of the new channel. Hydraulic modelling has been used to test the appropriateness of the channel shape and size under low flow conditions.
- Uncontrolled or excessive erosion that could impact infrastructure or destabilise the bed: The steeper channel profile required by the client has the potential to generate high energy that could lead to instability in the reach, particularly if sediment supply to the reach is limited by the reservoir and modifications upstream. To mitigate this, features are recommended in the concept design to control erosion and mimic morphological units found in steep channels (i.e. rapid and step pools).
- Uncontrolled or excessive erosion that could impact infrastructure through bank erosion: Bank erosion is a residual risk that will need to be considered in future design work. The conceptual design has involved working with natural processes to produce a functioning channel, within the constraints of the site. Some bank erosion may therefore occur on the outside of small bends within the channel; monitoring will be required to ensure bank erosion is not excessive and does not threaten the stability of infrastructure within the development.

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6 Indicative Drawings

6.1 Introduction

Following on from the hydraulic modelling a series of indicative drawings were developed to demonstrate the in-channel bar features geometry, which met the design criteria of the bars being fully submerged at Q10 and exposed during lower flows for example at Q70, and developing slower in-channel velocities through the pool, but increased velocities over the step-pool sections.

The indicative cross sections are provided within DRV-JBAU-XX-00-DR-EN-0001-S3-P02-Indicative_Cross_Sections. Within the indicative drawings it outlines the long section marked with the locations for the different in-channel features, such as step-pool sequences and bar features. The step-pool sequences are located within areas of steeper bed gradient as these processes are anticipated to be dominated by erosional processes. Whilst in shallower bed gradients it is anticipated to have more depositional environments allowing the formation of bars. River sections where the bar is present the channel is typically wider which encourages deposition during higher flows and develops a varied channel morphology. It additionally displays how the cross sections developed for the hydraulic modelling compare to the original cross sections within the model provided.

6.2 CDM Regulation 2015

This report provides conceptual design parameters and information for the new channel associated with the Bevis Green site. If these are taken forward to detailed design and implementation all work will need to follow the framework and fulfil the obligations outline in the Construction (Design and Management) Regulation 2015 (CDM Regulations). At this early stage, JBA Consulting has not been designated as the Principal Designer but have acted as a designer. When progressing the designs further, the client should assess where the nature and scale of the works are likely to be notifiable under CDM regulations (the criteria for it being notifiable is that the construction phase will last more than 30 working days or shifts and gas more than 20 persons on site, or will involve more than 500 person days or shifts).

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7 Conclusions

A hydraulic model has been developed to include in-channel features to improve the geomorphic conditions of the watercourse from its existing culvert. A geomorphology assessment was undertaken to understand hydromorphic processes operating in the watercourse and an analogue site was identified upstream. The concept design for the new channel has been based on this analogue site.

Bar features were hydraulically modelled to narrow the channel width and create an area of deeper flow during low flow conditions. Water levels were shown to submerge the bar feature during a Q10 event whilst at a lower flow extent, such as Q70, the bars were exposed with all the flow contained within the low flows channel. The calculated entrainment values showed a Q10 event would be capable of entraining and transporting small gravels (6-38mm), whilst smaller sediment sizes could be transported during a Q70 event.

This sediment mobilisation is similar to observations made during the site walkover. Velocities and shear stress were found to be lower in areas where bar features were present in the model, indicating higher rates of deposition in these parts of the channel. Immediately downstream of the bars, an increase in velocities and entrainment was shown, indicating the potential for riffle features to develop which would encourage greater geomorphic diversity within the channel.

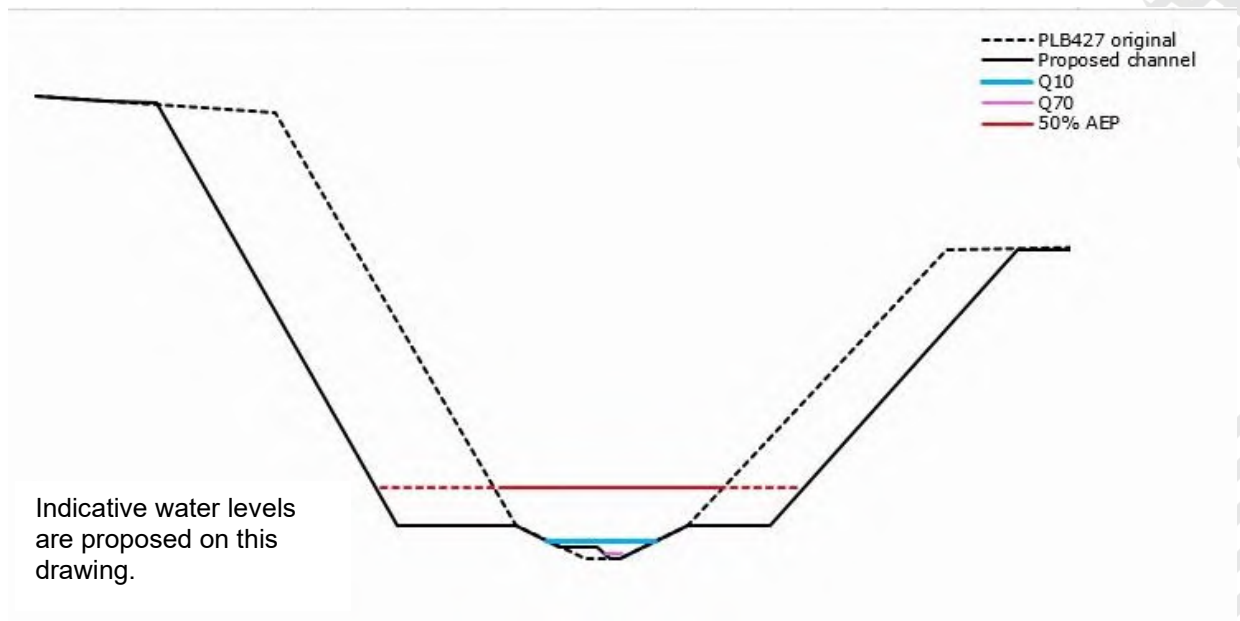
8 Recommendations

It is recommended that the following should be completed at the next stage:

- Advancement of the concept proposals during the detailed design stage - it is assumed this will be undertaken by Barratt Homes.
- It may be required to conduct further flood risk modelling of the new design to ensure the proposed channel does not increase flood risk to adjacent properties.
- A Water Framework Directive assessment would be required to obtain the relevant permits.
- Larger flows (such as the 50% AEP) should not be contained within the channel as this disconnects the channel from its floodplain and concentrates high energy within the channel, which can lead to exacerbated erosion. It is understood that space is limited; however, by designing the channel so that the upper bank profiles are set back and larger flows are able to spill onto a second stage of the channel, would allow for some riparian connectivity within the limited river corridor. This would also provide additional flood storage capacity during flood flows without impacting upon the gradient of slopes on both banks, as the figure below demonstrates.

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Legend

In-channel feature

- Bar
- Step/rapid
- Pool
- Riffle
- Channel

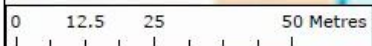
Pool riffle morphology
Shallow gradient

PLB352

PLB427

Flow Direction

Steep gradient:
Cobble rapids and/or
step pools to control
channel gradient and
high flow energy.



Legend

In-channel feature

- Bar
- Step/rapid
- Pool
- Riffle
- Channel

Steep gradient:
Cobble rapids and/or
step pools to control
channel gradient and
high flow energy.

PLB427

Flow Direction

Pool riffle morphology
Shallow gradient

PLB641



Bevis Green – Conceptual Plan view

This document presents a concept design comprising of an indicative plan for the new channel with example placement of morphological features. Further hydraulic modelling and analysis will be needed as part of a detailed design stage to determine the placement and size of in-channel features (including size of sediment) to progress the design for implementation.

Revision Ref/Date	Amendments
P02 -2 nd December 2020	Draft – Issued for Comment

Drawn: E. Wright
27/11/2020

Checked: R Thrower
02/12/2020

Approved: M Hemsworth
02/12/2020

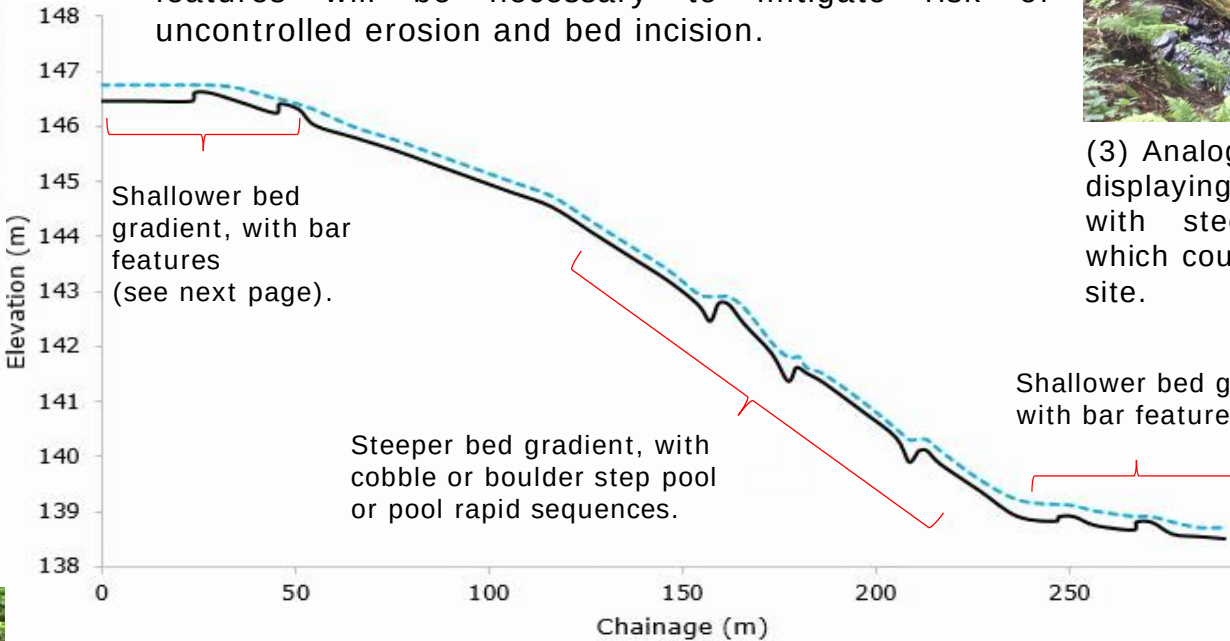
Project Reference:
2020s0819



(1) Analogue reach displaying a step-pool morphology with steeper bed gradient. A similar concept could be achieved at the site, but with a wider channel as required.



The shallower bed gradient at either end of the new channel will reduce in-channel velocities and allow for depositional processes to dominate. It is anticipated that bar features will form (2), reducing the channel width and introducing morphological diversity.



Within areas of steeper bed gradient, features mimicking either step pool or cascade morphology should be created along this reach. Coarser bed material will form the step or rapid (areas of raised bed level) whilst material upstream of the rapid will be scoured out to form a pool, with increased water depths and reduced in-channel velocities. It is expected to see sediment deposition within the pool as a result. Erosional processes will dominate throughout this reach. Appropriate placement and sizing of features will be necessary to mitigate risk of uncontrolled erosion and bed incision.



(3) Analogue reach photograph displaying a cascade channel with steeper bed gradient, which could be achieved at the site.



(4) Typical sediment identified downstream of the step pool/ cascade reaches, predominately consisting of coarse gravels.

Bevis Green – Long Section Plan

Indicative long-section plot with example placement of morphological features to control gradient and limit high velocities. To be developed further as part of a detailed design stage.

Revision Ref/Date	Amendments
P02 -2 nd December 2020	Draft – Issued for Comment
Drawn: E. Wright 27/11/2020	
Checked: R Thrower 02/12/2020	
Approved: M Hemsworth 02/12/2020	
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Bevis Green – Indicative Plan of Pool Riffle sections

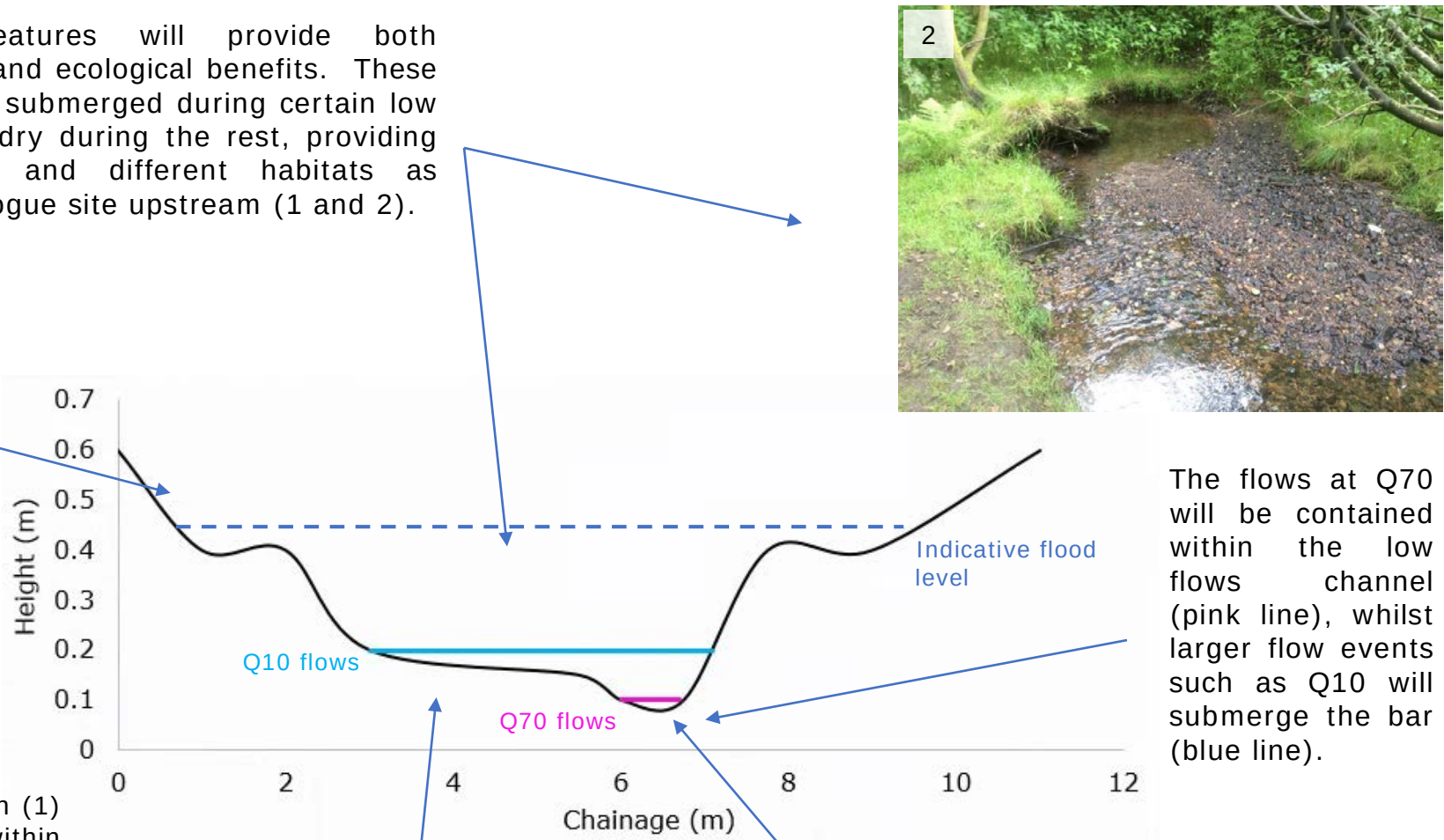
Indicative cross-section showing the typical channel shape within the shallower gradient sections of channel. To be developed further as part of a detailed design stage.

Revision Ref/Date	Amendments
P02 -2 nd December 2020	Draft – Issued for Comment

Drawn: E. Wright 27/11/2020
Checked: R Thrower 02/12/2020
Approved: M Hemsworth 02/12/2020
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In-channel bar features will provide both hydromorphological and ecological benefits. These features will only be submerged during certain low flow conditions and dry during the rest, providing channel complexity and different habitats as observed in the analogue site upstream (1 and 2).

The gradient of this slope will depend on the difference between the desired bed level and proposed site ground levels.



The flows at Q70 will be contained within the low flows channel (pink line), whilst larger flow events such as Q10 will submerge the bar (blue line).

Analogue reach photograph (1) displaying a bar feature within an area of gentler gradient.

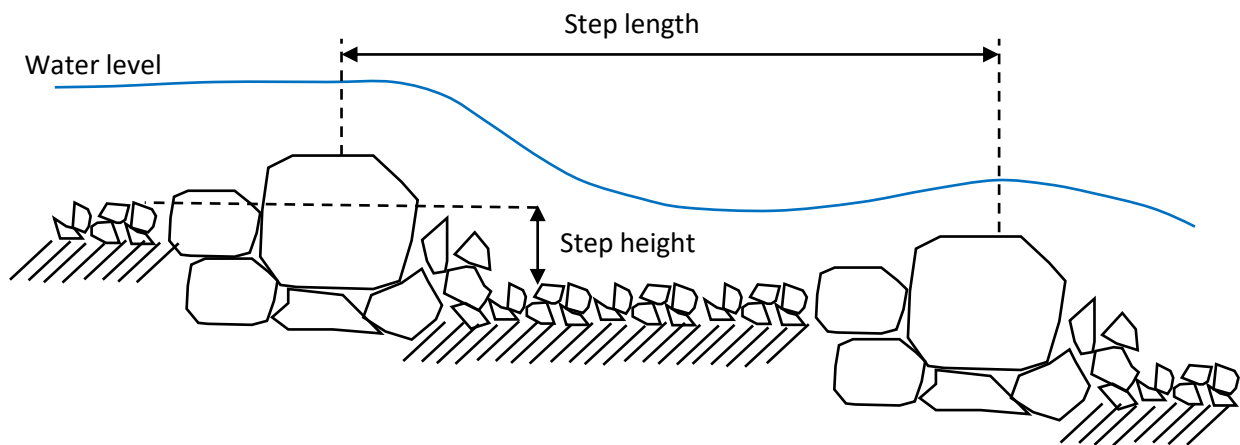


Bank heights are relatively shallow which is representative of the analogue reach and is important for maintaining a hydraulic connection to the riparian zone during larger flows.

Narrower low flows channel adjacent to the bar to encourage fine sediment to be transported downstream, as seen in the analogue reach. Generally, the channel is wider with bar features (compared with 'open channel' section) to encourage deposition during higher flows and varied channel morphology.

Bevis Green – Indicative Step Pool

This document presents a concept design comprising of an indicative plan for the new channel. Further hydraulic modelling and analysis will be needed as part of a detailed design stage to determine the placement and size of in channel features including size of sediment) to progress the design for implementation



The spacing and size of step pools should be variable, in order to maintain a natural sediment flux, particularly during flood flows. Designs should avoid uniform dimensions for the step pools.

Revision Ref/Date	Amendments
VO – December 2020	Draft – Issued for Comment
Drawn: E. Wright	
Checked: R. Thrower	
Approved: M. Hemsworth	
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Cross section Reference	Parameter	Return Event								
		1:2yr	1:10yr	1:30yr	1:50yr	1:75yr	1:100yr	1:100yr +35%	1:100yr + 70%	1:1000 yr
CH352										
Pool riffle										
	Peak Water Level (mAOD)	147.06	147.2	147.273	147.304	147.34	147.366	147.473	147.584	147.608
	Peak Water Depth (m)	0.61	0.75	0.823	0.854	0.89	0.916	1.023	1.134	1.158
	Wetted Perimeter (m)	6.358	7.243	7.705	7.901	8.129	8.293	8.97	9.672	9.824
	Wetted Area (m2)	2.641	3.562	4.089	4.322	4.601	4.806	5.696	6.692	6.917
	Peak Velocity (m/s)	1.016	1.242	1.356	1.443	1.469	1.505	1.669	1.829	1.868
	Shear Stress	33.92973539	47.92848828	55.69893688	62.44161272	63.98049136	66.62818489	79.48085132	92.75910412	96.19510047
	Critical Shear (entrainment mm)	46.58166496	65.8003594	76.46830093	85.72522743	87.83793265	91.47291452	109.1181627	127.3476925	132.0649244
	Sediment type which can be entrained	All sediment up to Very coarse gravel	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Boulders
CH387										
Pool riffle	Peak Water Level (mAOD)	146.718	146.815	146.865	146.89	146.918	146.938	147.018	147.107	147.126
	Peak Water Depth (m)	0.368	0.465	0.515	0.54	0.568	0.588	0.668	0.757	0.776
	Wetted Perimeter (m)	5.4	5.916	6.183	6.316	6.465	6.572	6.998	7.472	7.573
	Wetted Area (m2)	1.245	1.785	2.082	2.235	2.409	2.537	3.065	3.69	3.829
	Peak Velocity (m/s)	1.055	1.245	1.348	1.398	1.432	1.465	1.609	1.743	1.775
	Shear Stress	44.51660863	56.67754545	64.05626924	67.76595403	69.88848535	72.28973421	83.60564288	94.26286222	96.99140735
	Critical Shear (entrainment mm)	61.11623696	77.81181911	87.94196705	93.03494205	95.94893598	99.24557736	114.781032	129.4121812	133.1581631
	Sediment type which can be entrained	All sediment up to Very coarse gravel	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Boulders	All sediment up to Boulders
CH427										
Cobble rapid / step pool	Peak Water Level (mAOD)	145.92	146.009	146.056	146.08	146.106	146.123	146.199	146.28	146.299
	Peak Water Depth (m)	0.35	0.439	0.486	0.51	0.538	0.553	0.629	0.71	0.729
	Wetted Perimeter (m)	5.322	5.829	6.097	6.234	6.393	6.479	6.923	7.373	7.481
	Wetted Area (m2)	1.51	1.64	1.915	2.06	2.234	2.328	2.825	3.389	3.527
	Peak Velocity (m/s)	1.139	1.346	1.457	1.504	1.545	1.583	1.731	1.874	1.907
	Shear Stress	48.41973838	67.80825032	76.59091511	80.24251789	83.11380444	86.44618801	99.07447469	111.6021113	114.5939793
	Critical Shear (entrainment mm)	66.47478987	93.09301004	105.1506092	110.1638442	114.1057938	118.6807772	136.0179775	153.2169968	157.3244909
	Sediment type which can be entrained	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Cobbles	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders
CH468										
Cobble rapid / step pool	Peak Water Level (mAOD)	144.843	144.919	144.958	144.978	145	145.016	145.082	145.153	145.17
	Peak Water Depth (m)	0.313	0.389	0.428	0.448	0.47	0.486	0.552	0.623	0.64
	Wetted Perimeter (m)	5.122	5.522	5.727	5.832	5.948	6.032	6.379	6.752	6.842
	Wetted Area (m2)	0.964	1.364	1.58	1.693	1.829	1.915	2.315	2.769	2.883
	Peak Velocity (m/s)	1.362	1.625	1.769	1.83	1.888	1.931	2.124	2.312	2.352
	Shear Stress	79.38724753	103.2150651	117.8930076	124.0408446	129.5185819	134.051316	155.1143259	176.4497991	180.9657804
	Critical Shear (entrainment mm)	108.9896553	141.7025368	161.8536814	170.2939618	177.8142717	184.0371997	212.954315	242.2454914	248.4454197
	Sediment type which can be entrained	All sediment up to Cobbles	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders
CH508										
Cobble rapid / step pool	Peak Water Level (mAOD)	143.059	143.131	143.172	143.19	143.211	143.226	143.292	143.364	143.379
	Peak Water Depth (m)	0.309	0.381	0.422	0.44	0.461	0.476	0.542	0.614	0.629
	Wetted Perimeter (m)	5.093	5.482	5.703	5.801	5.914	5.995	6.351	6.74	6.821
	Wetted Area (m2)	0.943	1.32	1.545	1.648	1.769	1.856	2.255	2.715	2.814
	Peak Velocity (m/s)	1.389	1.669	1.808	1.873	1.938	1.981	2.178	2.364	2.409
	Shear Stress	83.01708924	109.8103787	123.8979624	130.8779419	137.7318531	142.2705593	164.2945999	185.5812012	191.18652
	Critical Shear (entrainment mm)	113.9730149	150.7571519	170.0978009	179.6805182	189.0901582	195.3212854	225.5577863	254.7818671	262.4773319
	Sediment type which can be entrained	All sediment up to Cobbles	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders
CH548										
Cobble rapid / step pool	Peak Water Level (mAOD)	141.131	141.197	141.24	141.258	141.277	141.291	141.356	141.422	141.438
	Peak Water Depth (m)	0.311	0.377	0.42	0.438	0.457	0.471	0.536	0.602	0.618
	Wetted Perimeter (m)	5.113	5.531	5.802	5.916	6.036	6.125	6.536	6.953	7.054
	Wetted Area (m2)	0.954	1.301	1.541	1.646	1.757	1.841	2.246	2.681	2.791
	Peak Velocity (m/s)	1.382	1.668	1.788	1.839	1.902	1.945	2.095	2.244	2.272
	Shear Stress	81.97238569	110.5375657	121.9745076	127.0491955	133.8718293	138.5050672	153.6778117	169.6731775	172.4451071
	Critical Shear (entrainment mm)	112.5387558	151.7554967	167.4571163	174.4240852	183.7907849	190.1516932	210.9821446	232.9419612	236.7475051
	Sediment type which can be entrained	All sediment up to Cobbles	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders	All sediment up to Boulders
CH588										
Pool riffle	Peak Water Level (mAOD)	139.365	139.548	139.675	139.734	139.811	139.866	140.116	140.41	140.481
	Peak Water Depth (m)	0.465	0.648	0.775	0.834	0.911	0.966	1.216	1.51	1.581
	Wetted Perimeter (m)	6.095	7.25	8.051	8.424	8.91	9.257	10.835	12.691	13.139
	Wetted Area (m2)	1.806	3.004	3.953	4.427	5.076	5.562	7.998	11.342	12.227
	Peak Velocity (m/s)	0.745	0.77	0.773	0.773	0.774	0.774	0.775	0.774	0.775
	Shear Stress	20.41770303	19.50457504	18.57557822	18.15951656	17.72323384	17.41154717	16.29902377	15.25315457	15.0878221
	Critical Shear (entrainment mm)	28.03118241	26.7775616	25.5021547	24.9309494	24.33198288	23.90407256	22.37670455	20.94084517	20.71386251
	Sediment type which can be entrained	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels
CH628										
Pool riffle	Peak Water Level (mAOD)	139.197	139.457	139.61	139.677	139.762	139.822	140.086	140.389	140.461
	Peak Water Depth (m)	0.617	0.877	1.03	1.097	1.182	1.241	1.506	1.809	1.881
	Wetted Perimeter (m)	6.866	8.359	9.238	9.623	10.112	10.451	11.957	13.355	13.685
	Wetted Area (m2)	2.754	4.679	5.983	6.593	7.402	7.984	10.838	14.502	15.427
	Peak Velocity (m/s)	0.493	0.494	0.494	0.494	0.495	0.495	0.494	0.494	0.495
	Shear Stress	8.082570021	7.262118656	6.917501026	6.78899336	6.667792465	6.573528049	6.184253375	5.822840729	5.773961113
	Critical Shear (entrainment mm)	11.09644871	9.970062371	9.496941589	9.320515189	9.154120156	9.024705841	8.490276019	7.99409759	7.926991441
	Sediment type which can be entrained	All sediment up to Medium gravels	All sediment up to Medium gravels	All sediment up to Medium gravels	All sediment up to Medium gravels	All sediment up to Medium gravels	All sediment up to Medium gravels	All sediment up to Medium gravels	All sediment up to Medium gravels	All sediment up to Medium gravels
CH641										
	Peak Water Level (mAOD)	139.151	139.423	139.582	139.65	139.737	139.798	140.067	140.373	140.446
	Peak Water Depth (m)	0.651	0.923	1.082	1.15	1.237	1.298	1.567	1.873	1.946
	Wetted Perimeter (m)	4.866	6.206	6.99	7.325	7.754	8.055	9.381	10.889	11.249
	Wetted Area (m2)	2.032	3.446	4.427	4.881	5.493	5.942	8.122	10.998	11.476
	Peak Velocity (m/s)	0.664	0.665	0.666	0.665	0.665	0.665	0.666	0.667	0.666
	Shear Stress	14.46642795	13.19525557	12.66718244	12.4170543	12.16627716	12.00318793	11.41351791	10.87473739	10.80600755
	Critical Shear (entrainment mm)	19.86075907	18.11558406	17.39059977	17.0472023	16.70291383	16.47901088	15.66946105	14.92977672	14.83541848
	Sediment type which can be entrained	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Coarse gravels	All sediment up to Medium gravels	All sediment up to Medium gravels	All sediment up to Medium gravels

Appendix 4



GRID COORDINATES			
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100	000000.00	010000.00	100.00

SURVEY GRID AND LEVELS ARE BASED ON THE ORIGINAL SITE SURVEY CARRIED OUT BY OTHERS, DRAWING No 122018/001. THIS DRAWING HAS BEEN PRODUCED BY A COMBINATION OF OS DIGITAL SUPERPLAN AND STANDARD TOPOGRAPHICAL DIGITAL SURVEYING. CARE SHOULD BE TAKEN WHEN MEASURING FROM ONE SYSTEM TO ANOTHER. THE OS INFORMATION IS FOR GUIDANCE ONLY. ONLY MANHOLES FOUND DURING THE SITE SURVEY ARE INDICATED ON THIS PLAN. FURTHER INVESTIGATION MAY BE REQUIRED TO DETERMINE THE FULL EXTENT OF THE SITE DRAINAGE.

ISSUE	REVISION	DATE

PROJECT
TETROSYL,
WALMERSLEY OLD ROAD,
BURY.

DRAWING TITLE
EXTENSION TO TOPO SURVEY.

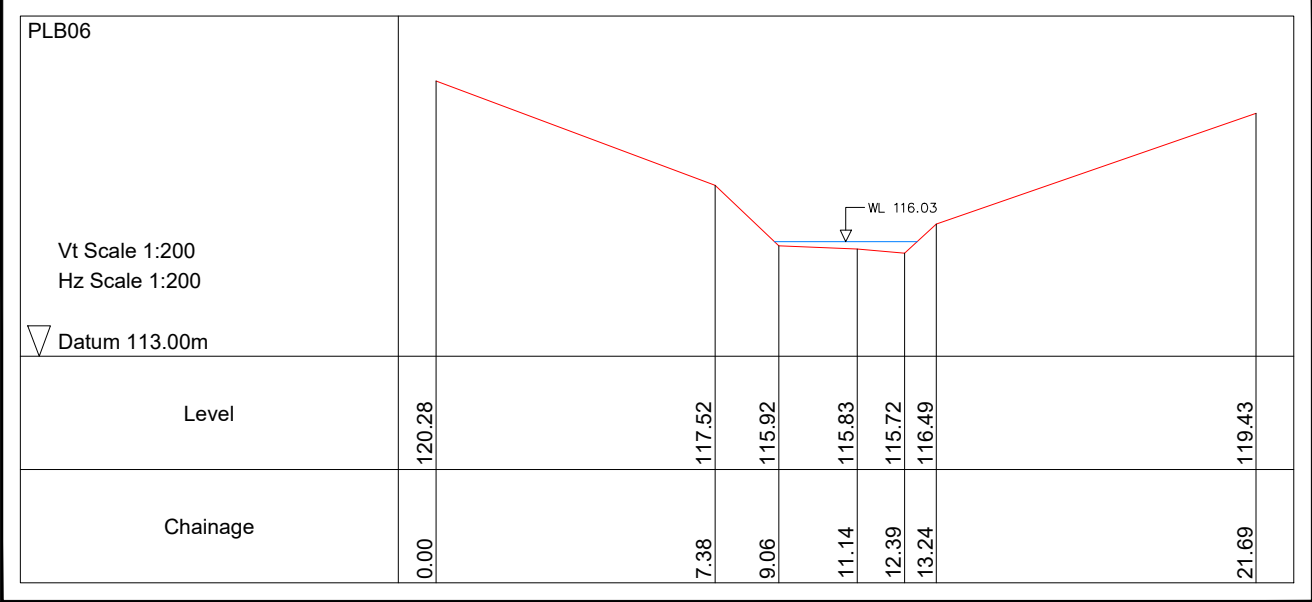
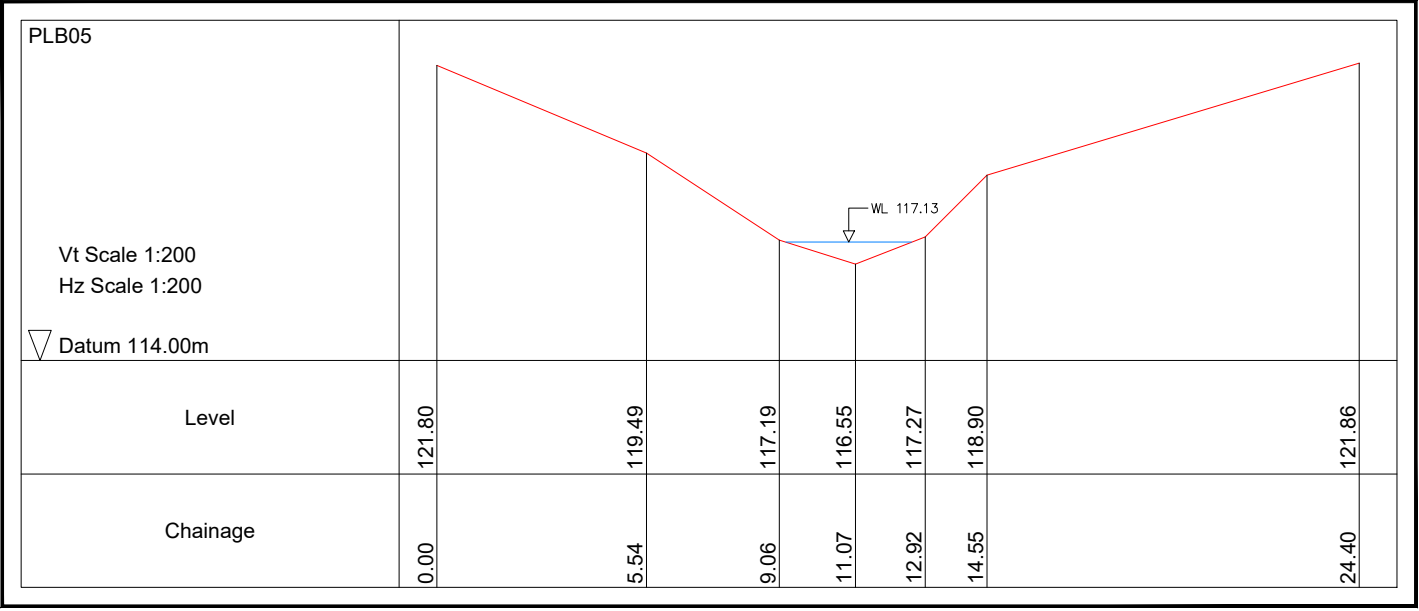
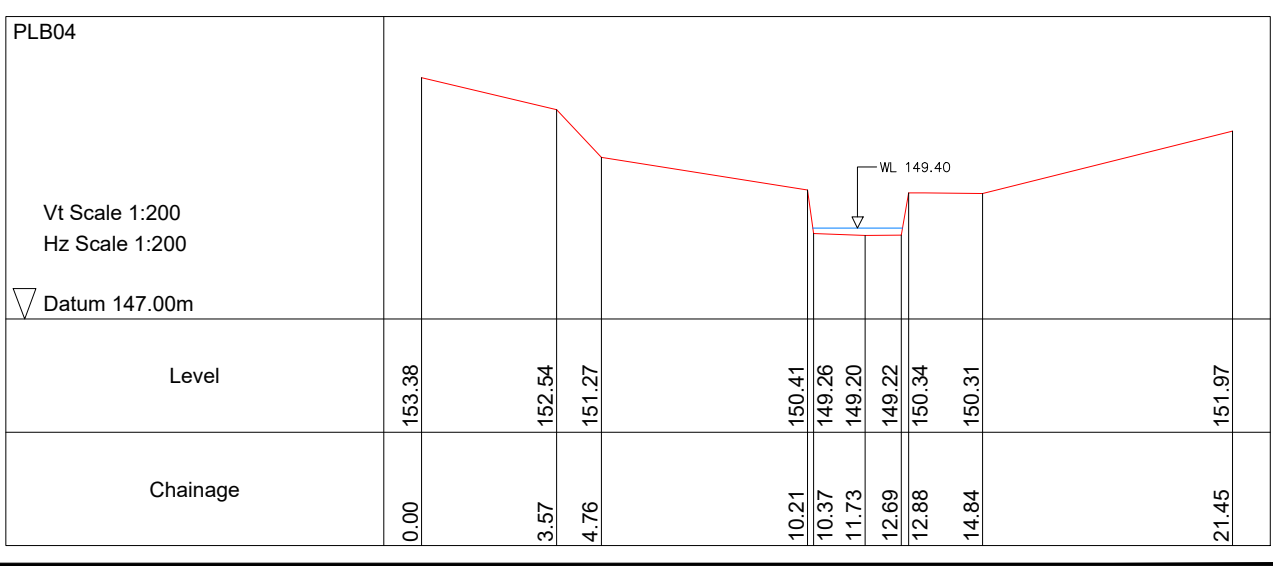
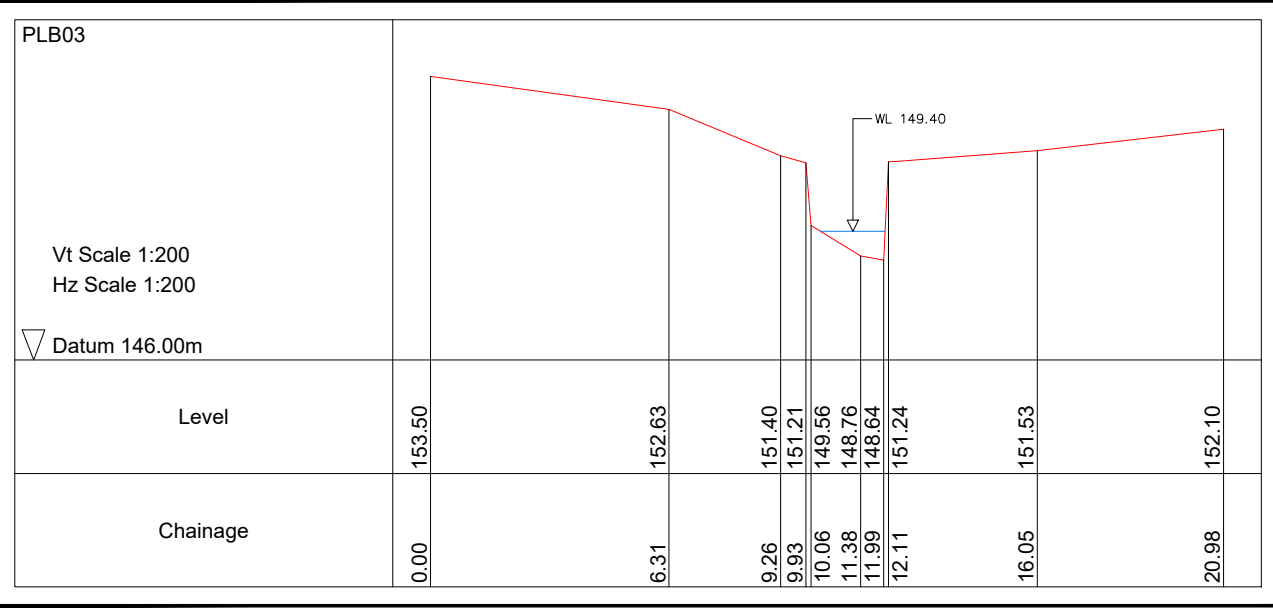
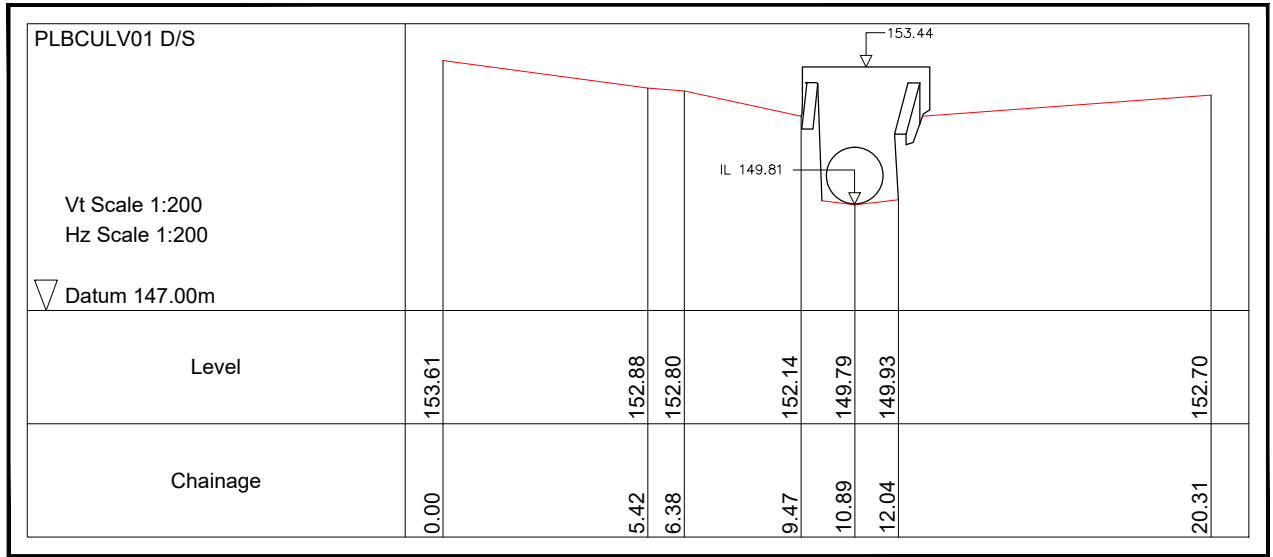
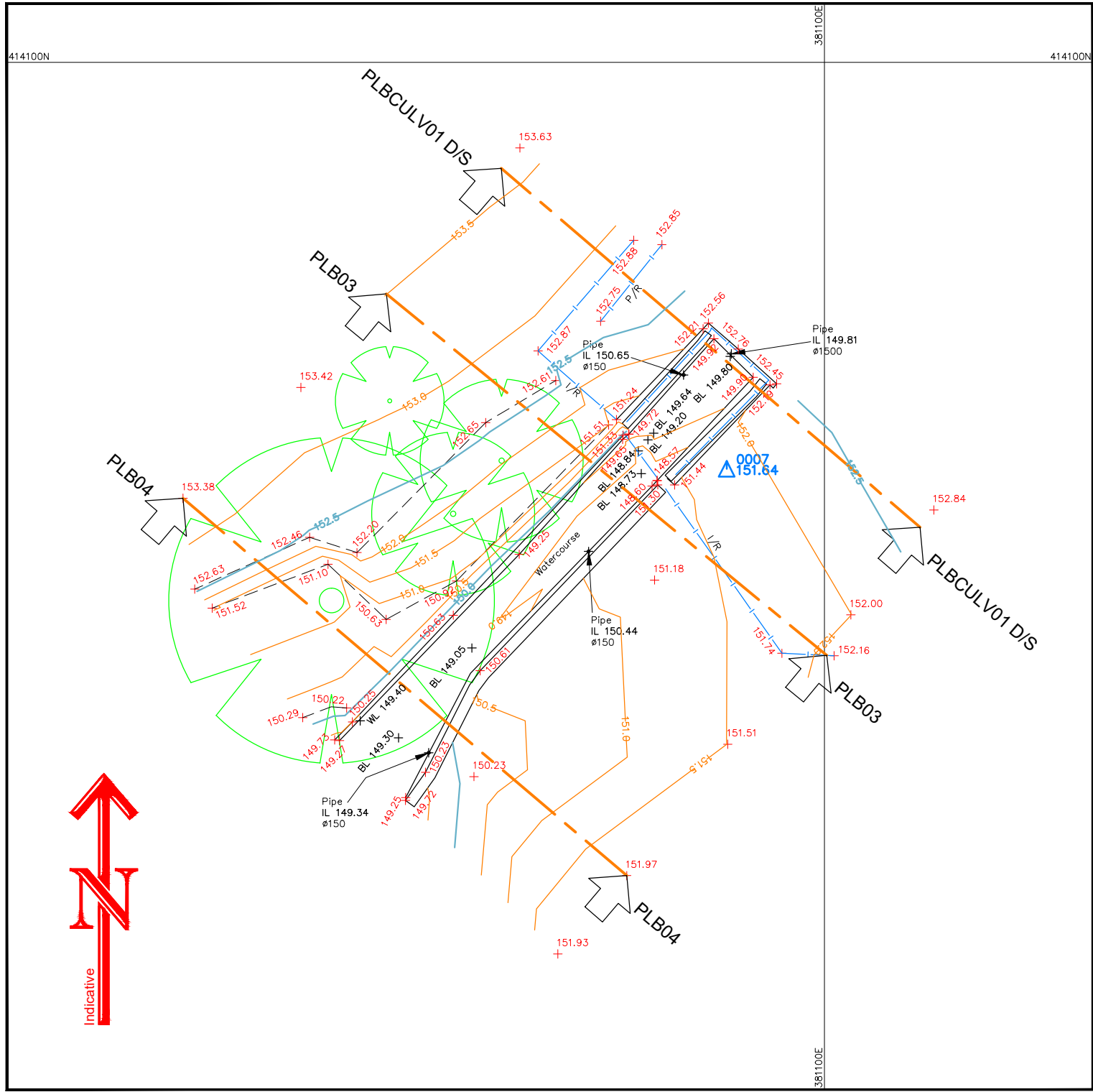
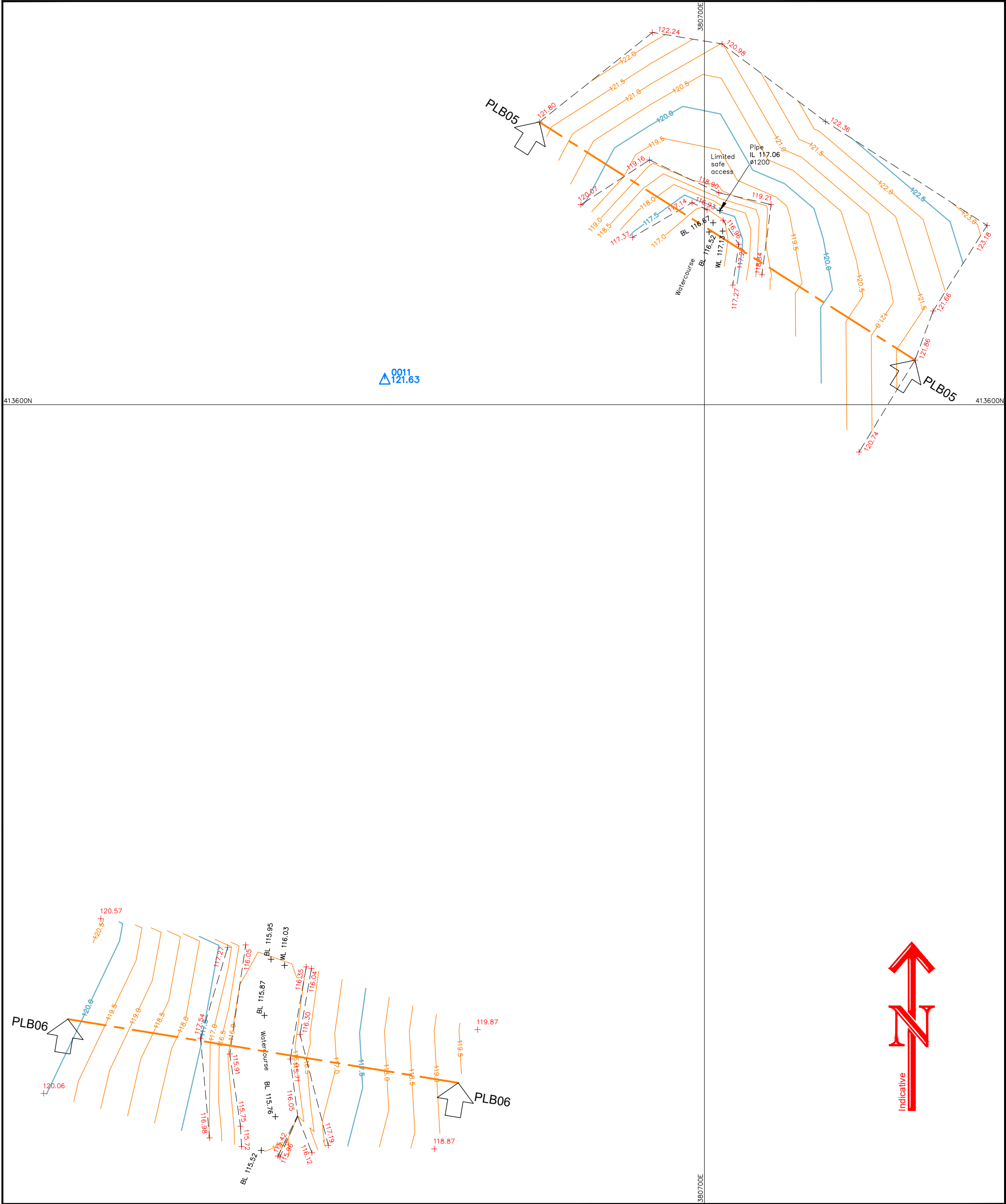
Scale 1:500 Date 12.03.15

Drawing Number SDL 2140/1. Rev. - Drawn by BU

CLIENT BARRATT HOMES MANCHESTER, CITY PARK, 4 BRINDLEY ROAD, MANCHESTER, M16 8HQ. TEL: 0161 872 0161.

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LAND SURVEYING • DESIGN CONSULTANTS
CIVIL • ARCHITECTURAL • PLANNING





Topographical Survey Legend

B Bollard	h Height	RWP Rain Water Pipe	Level Prefix Descriptions
BS Bus Stop	Hyd Hydrant	SP Soil Pipe	AL Arch Level
BT Telecommunications IC	IC Inspection Cover	SFS Stone Faving Slabs	BL Bed Level
CPS Concrete Paving Slabs	JB Junction Box	ST Stop Tap	CL Cover Level
CTV Cable Television Cover	KO Koth Outlet	SLS Stone Laying Slab	EL Eaves Level
DP Downpipe	LB Letter Box	SV Stop Valve	FL Floor Level
DK Drain	LP Lamp Post	TCS Telephone Call Box	IL Invert Level
ELC Electrical IC	Mx Utility Marker	TL Traffic Light	OL Overhead Cable Level
EP Elec. Pole	MP Mile Post	TP Telegraph Pole	PL Parapet Level
FR Earth Road	NP Name Plate	TFS Tactile Faving Slabs	RL Roofridge Level
FP Flagpole	P Post	VP Vent Pipe	SL Soffit Level
FM Flagpole	PM Parking Meter	WM Water Meter	Spr Arch Springing Level
G Gully	RE Rodding Eye	WO Water Outlet	TF Top Of Fence Level
GV Gate Post	RS Road Sign	UTL Unable To Lift	TL Threshold Level
GV Gas Valve	RTW Retaining Wall		WL Water Level

Fence Annotation
B/W Barbed Wire
C/B Closed Boarded
C/A Corrugated Iron
C/L Chain Link
C/P Concrete Panel
H/R Hand Rail
I/R Iron Railings
O/B Open Boarded
P/C Post & Chain
P/R Post & Rail
P/W Post & Wire
W/M Wire Mesh

Fence
Safety Barrier
Gate
Site
Hedge
Limit of Foliage
& Tree Canopy

Spot Level
Survey Control Station (coordinated point)
Tree
MT Multi Trunk

Levels defining edge of carriageway are observed at channel (bottom of kerb), Unless otherwise stated.

Survey Station Coordinates

Stn	Easting	Northing	Level
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0003	381172.64	414173.37	154.83
0004	381228.42	414163.07	161.41
0005	381214.12	414127.29	158.89
0006	381120.81	414072.82	157.65
0007	381096.40	414085.03	151.64
0008	380659.11	413616.23	130.59
0009	380664.06	413638.74	131.29
0010	380679.55	413640.40	131.64
0011	380682.47	413601.42	121.63

Please note:
Any discrepancy found between an Ordnance Survey benchmark and an OSGB15 computed orthometric height is likely to be due to bench mark subsidence or uplift and, assuming precise GPS survey has been carefully carried out, the orthometric height given by OSGB15 should be considered correct in preference to archive benchmark heights. OSGB15 converts GPS ellipsoid heights to orthometric heights above mean sea level.

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Levels are related to OSGB15
Value n/a
Tied to OS datum by GPS, see note.

Coordinates are related to National Grid (GPS)

Scale
0 2 4 6 8 10 12 14 16 18
metres

1:200

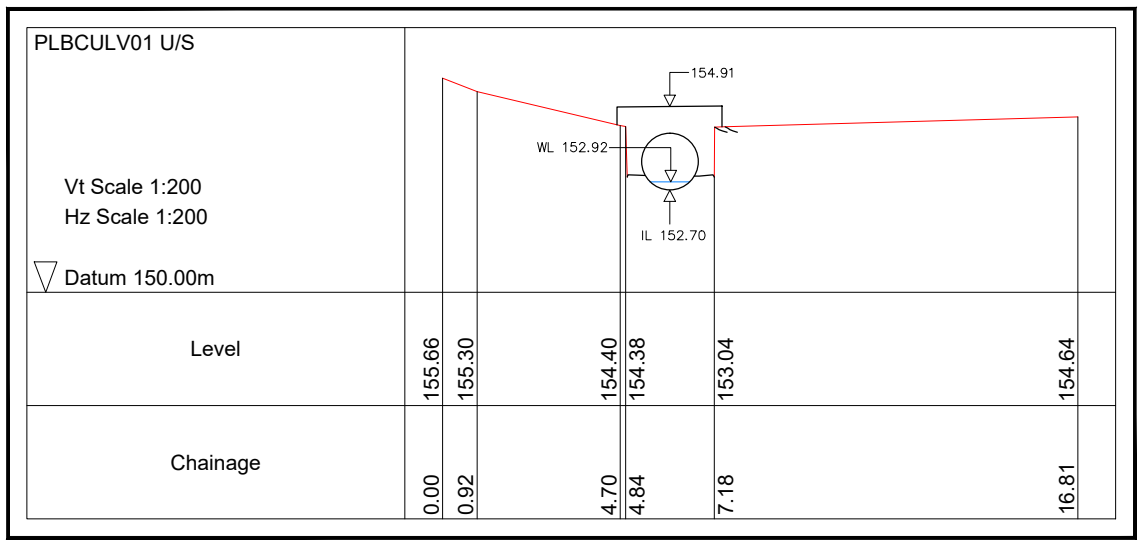
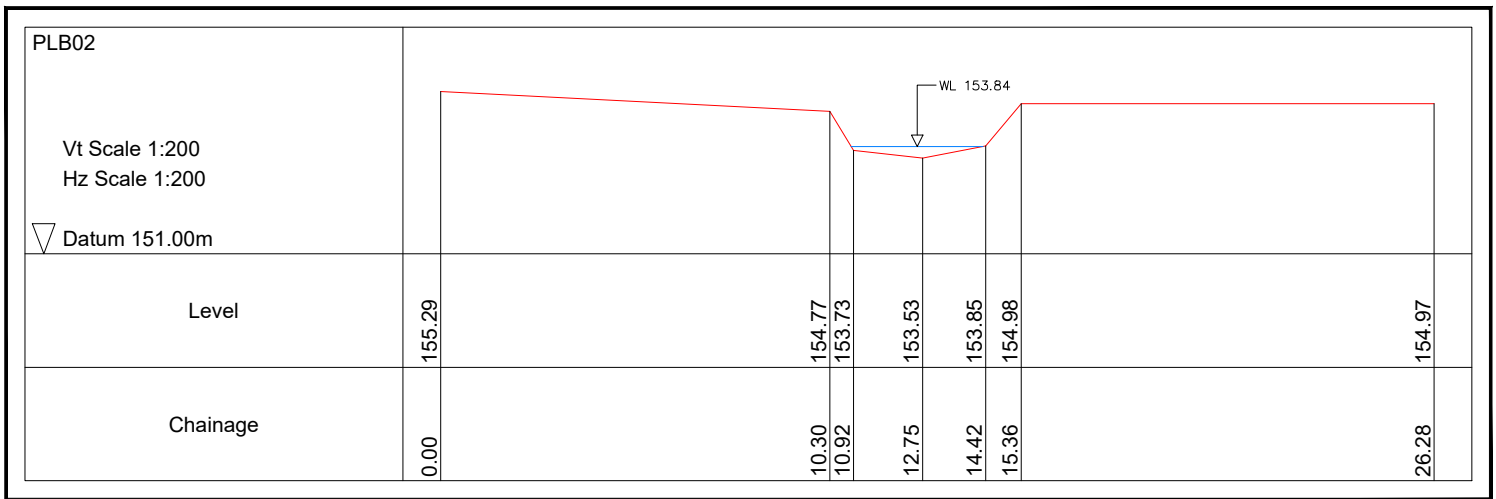
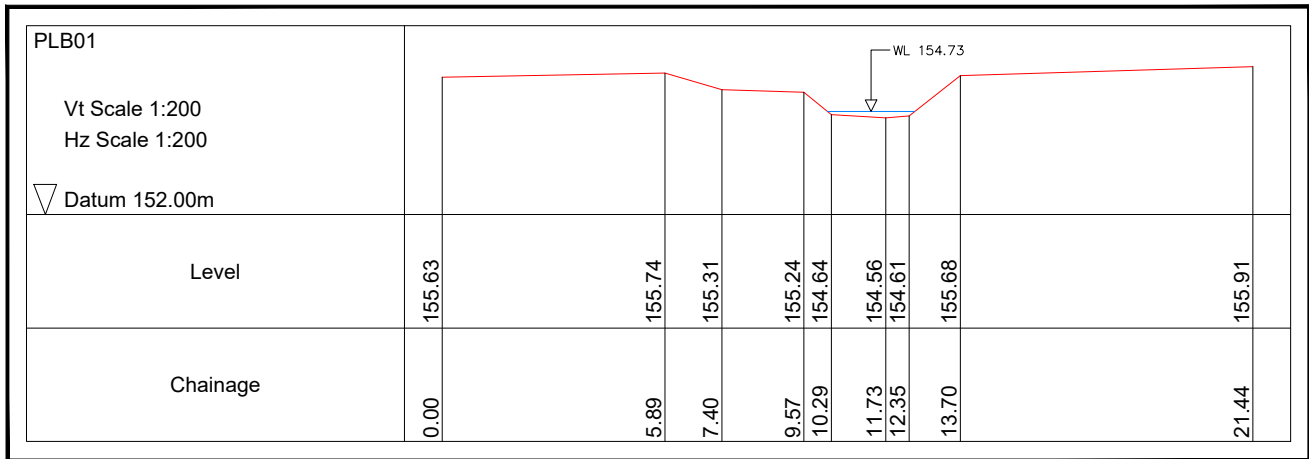
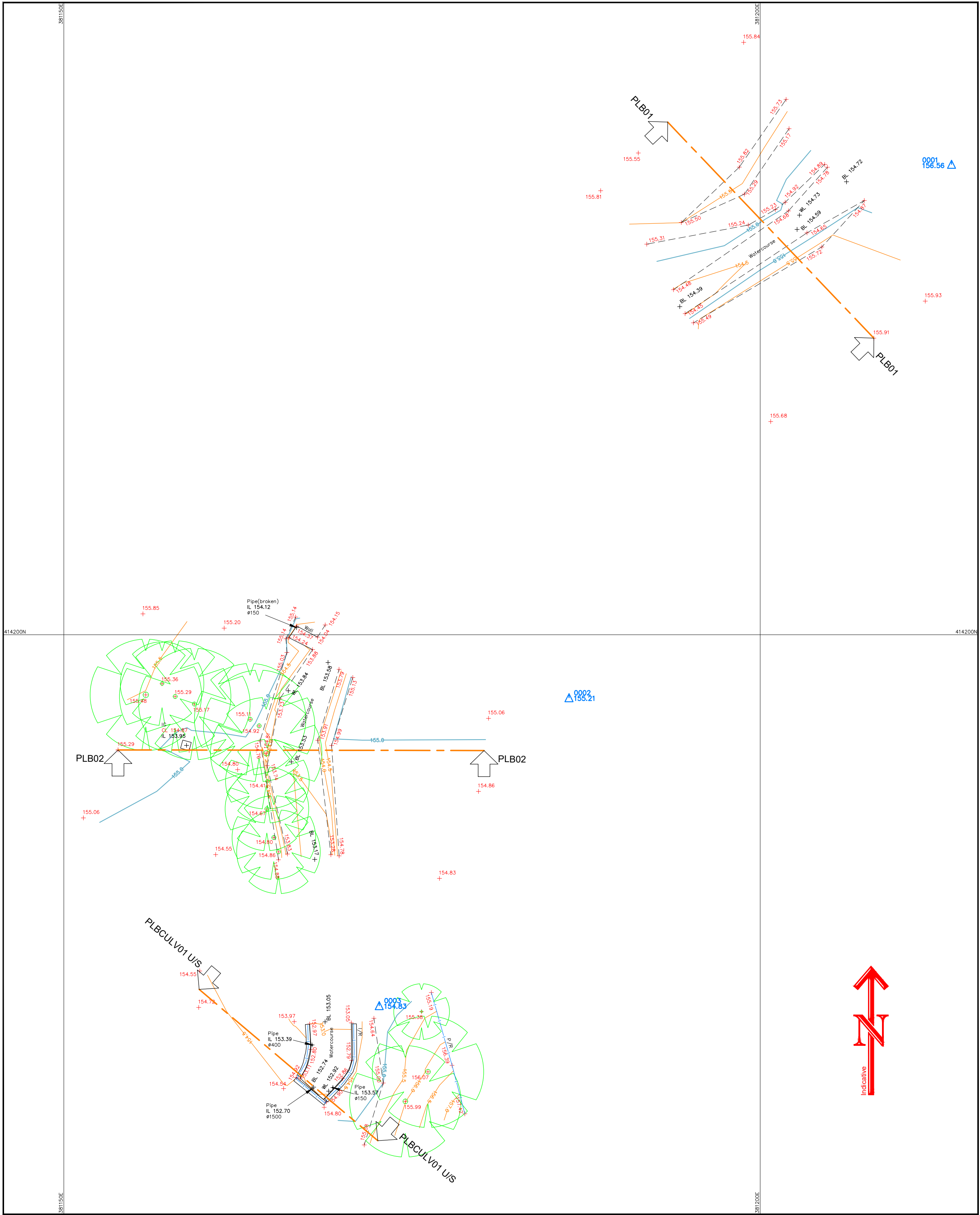
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Client **Barratt Homes Manchester**

Topographical & Sectional Survey
Tetrosyl Ltd & Bevis Green
Walmersley Old Rd BL9 6RE

Surveyed **AL AF** Drawn **ME dlp**
Date **February 2018** Checked **AL**
File name **18583_2D** Revision No. Sheet size **A1**
Drawing No **SSL:18583:200:2:2**

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Topographical Survey Legend			
B	Bollard	h	Height
BS	Bus Stop	Hyd	Hydrant
BT	Telecommunications IC	IC	Inspection Cover
CPS	Concrete Paving Slabs	JB	Junction Box
CTV	Cable Television Cover	KO	Kerf Outlet
DP	Downpipe	LB	Letter Box
DK	Drain	LP	Lamp Post
ELC	Electrical IC	Mx	Utility Marker
EP	Elec. Pole	MP	Mile Post
EP	Earth Road	NP	Name Plate
FP	Flagpole	P	Post
FT	Floodlight	PM	Parking Meter
G	Gully	RE	Rodding Eye
GV	Gate Post	RS	Road Sign
GV	Gas Valve	RTW	Retaining Wall
Fence Annotation		C/L	Fence
B/W	Barbed Wire	Z	Safety Barrier
C/B	Closed Boarded		Gate
C/A	Corrugated Iron		Site
C/L	Chain Link		Hedge
C/P	Concrete Panel		Limit of Foliage & Tree Canopy
H/R	Hard Rail		
I/R	Iron Railings		
O/B	Open Boarded		
P/C	Post & Chain		
P/R	Post & Rail		
P/W	Post & Wire		
W/M	Wire Mesh		
RWP	Rain Water Pipe		
SP	Soil Pipe		
SPS	Stone Paving Slabs		
ST	Stop Tap		
SV	Stop Valve		
TCS	Telephone Call Box		
TL	Traffic Light		
TP	Telegraph Pole		
TFS	Tactile Paving Slabs		
VP	Vent Pipe		
WM	Water Meter		
WO	Water Outlet		
UTL	Unable To Lift		
Thr	Threshold		
WL	Water Level		
AL	Arch Level		
BL	Bed Level		
CL	Cover Level		
EL	Enrse Level		
FL	Floor Level		
IL	Invert Level		
OL	Overhead Cable Level		
PL	Parapet Level		
RL	Roadbridge Level		
SL	Soft Level		
Spr	Arch Springing Level		
TF	Top Of Fence Level		
TL	Top Of Wall Level		
Thr	Threshold Level		
WL	Water Level		
50.00	Spot Level		
SSL 50.00	Survey Control Station (coordinated point)		
Tree MT	Multi Trunk		

Survey Station Coordinates

Stn	Easting	Northing	Level
0001	381213.74	414233.78	156.56
0002	381186.27	414195.48	155.21
0003	381172.64	414173.37	154.83
0004	381228.42	414163.07	161.41
0005	381214.12	414127.29	158.89
0006	381120.81	414072.82	157.65
0007	381096.40	414085.03	151.64
0008	380659.11	413616.23	130.59
0009	380664.06	413638.74	131.29
0010	380679.55	413640.40	131.64
0011	380682.47	413601.42	121.63

Please note:

Any discrepancy found between an Ordnance Survey benchmark and an OSGB15 computed orthometric height is likely to be due to bench mark subsidence or uplift and, assuming precise GPS survey has been carefully carried out, the orthometric height given by OSGB15 should be considered correct in preference to archive benchmark heights. OSGB15 converts GPS ellipsoid heights to orthometric heights above mean sea level.

Levels are related to OSGB15
Value n/a
Tied to OS datum by GPS, see note.

Coordinates are related to National Grid (GPS)



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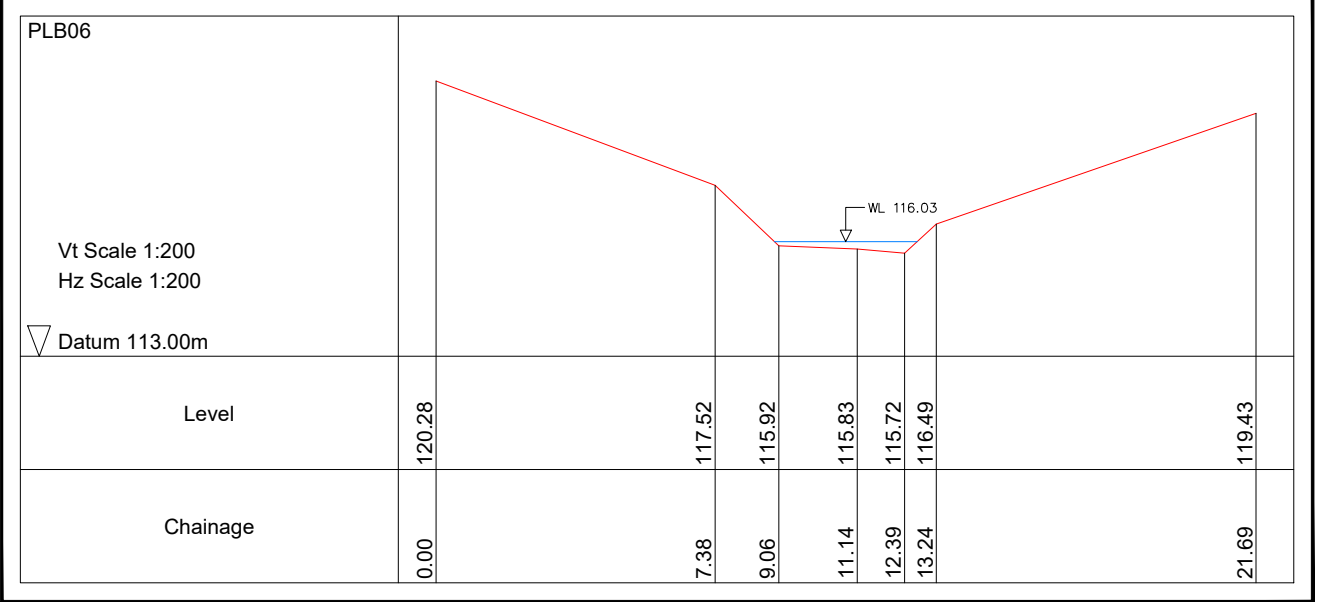
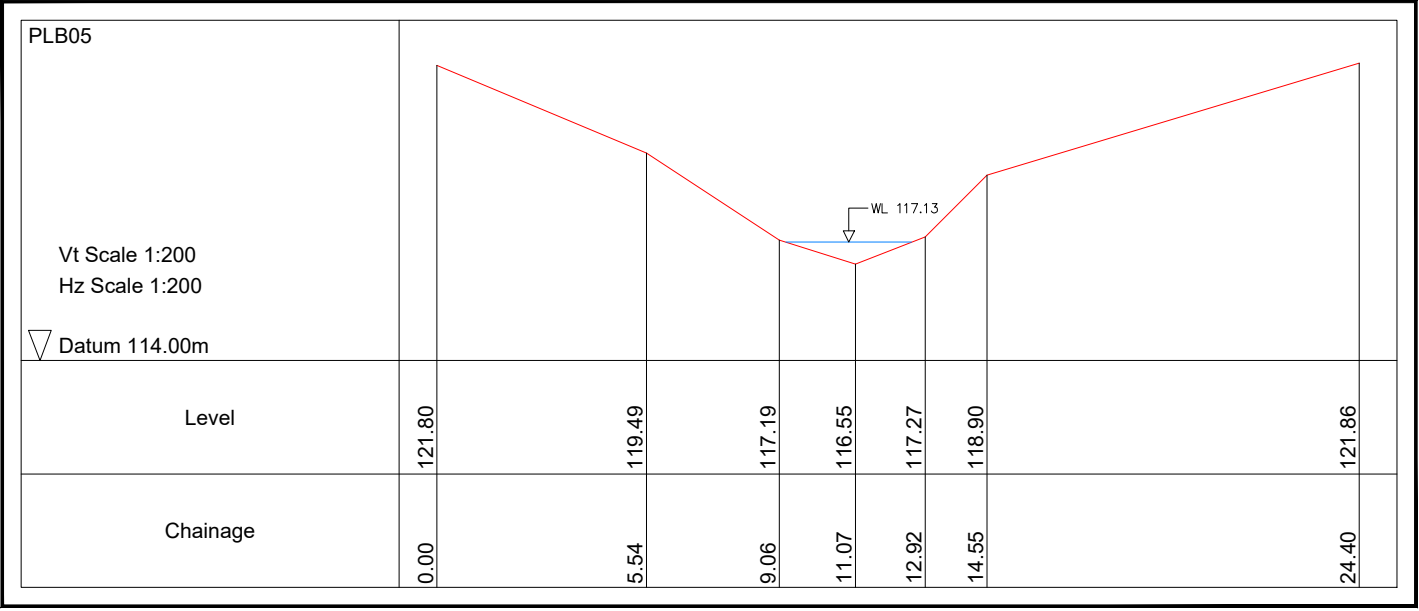
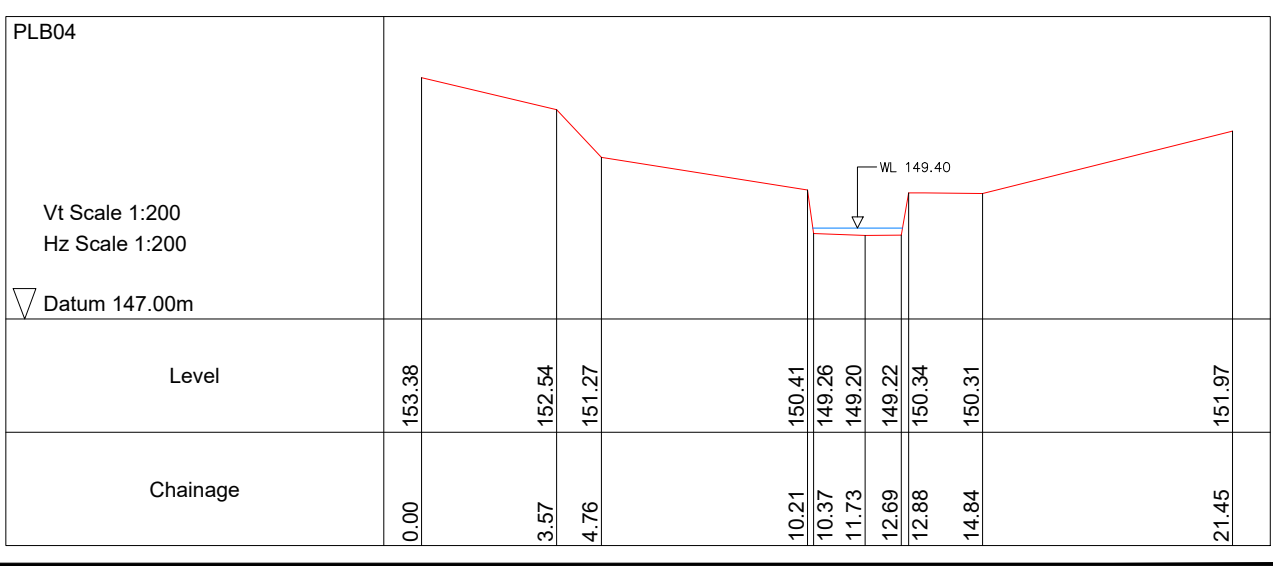
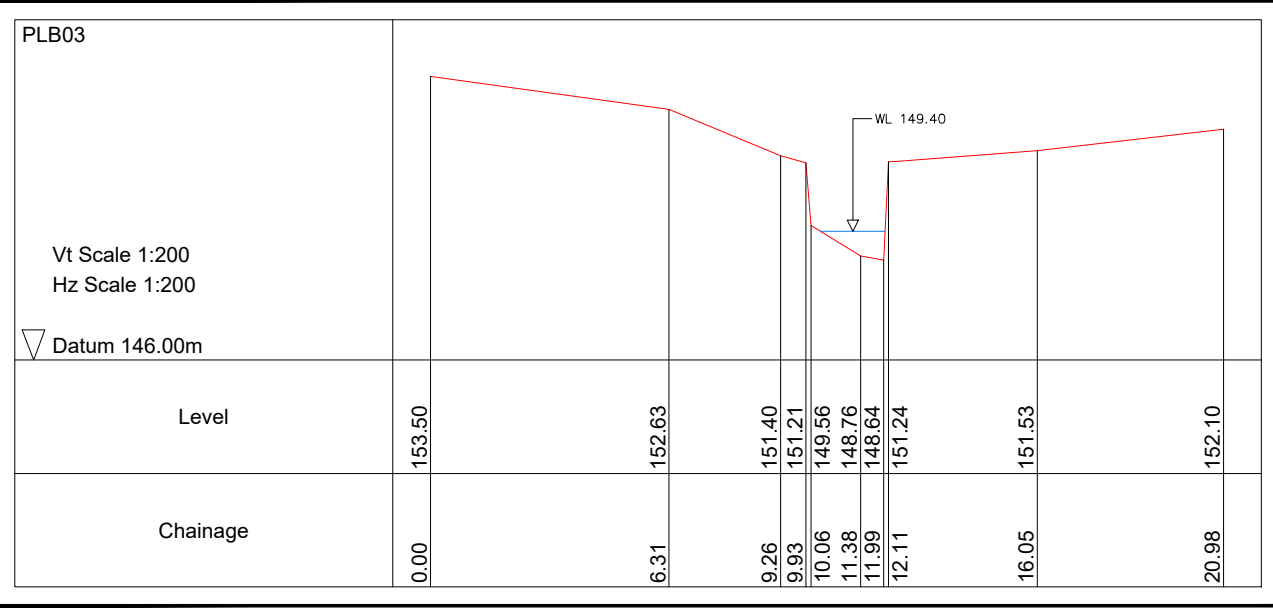
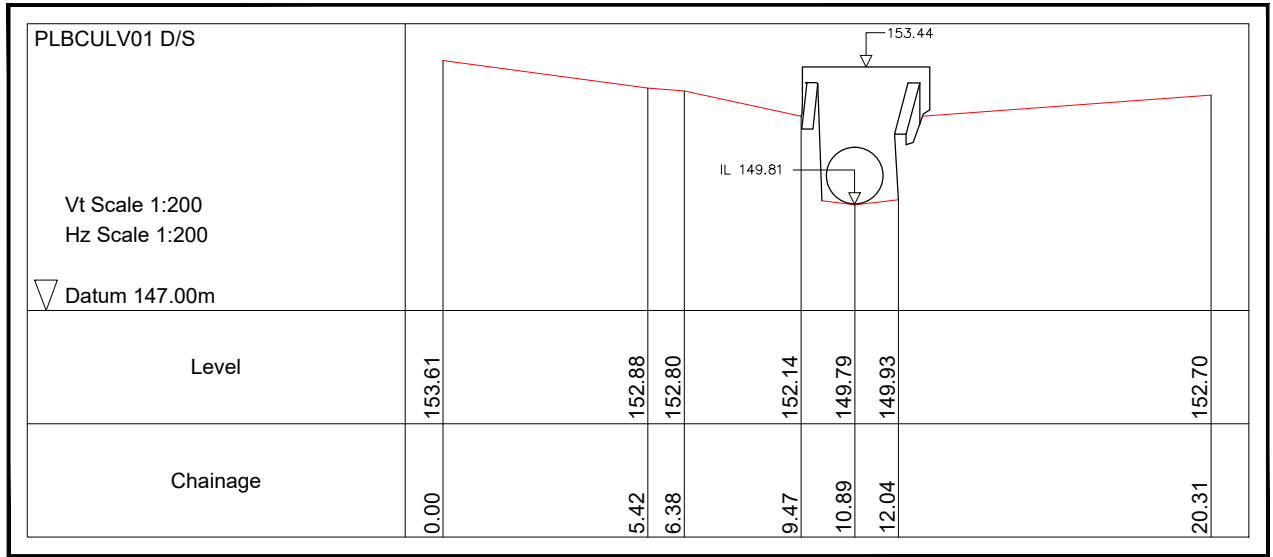
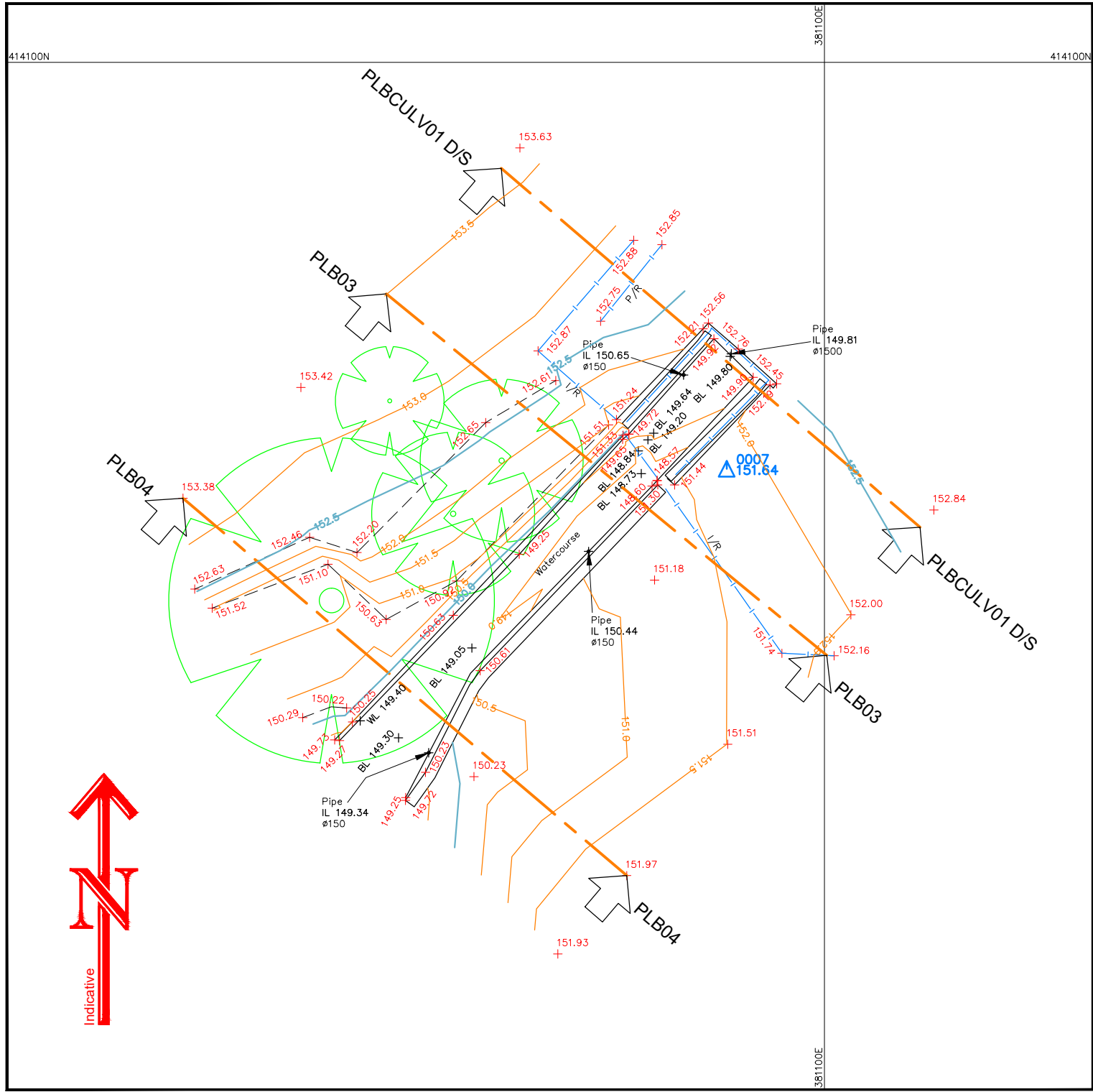
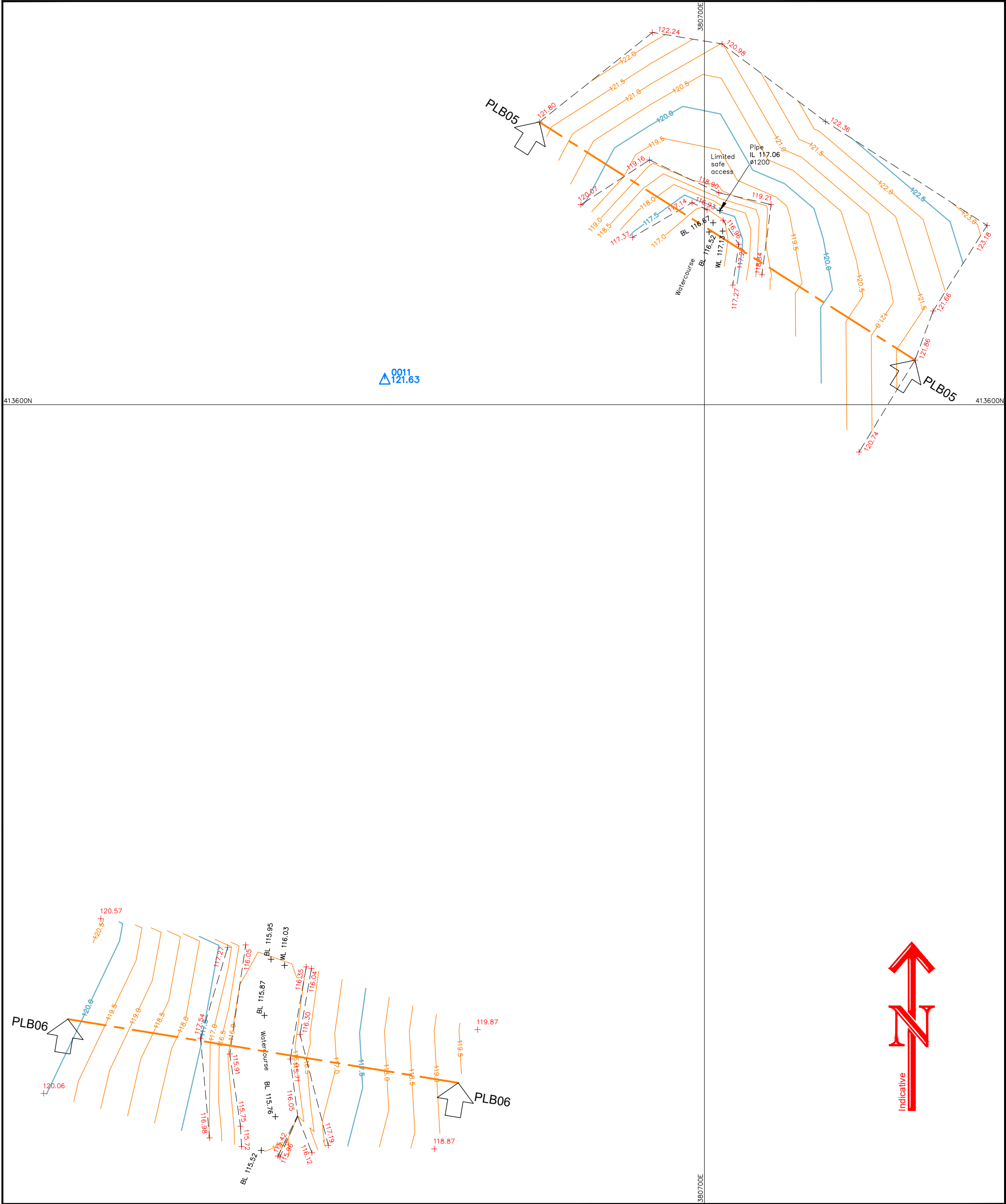
Client Barratt Homes Manchester

Topographical & Sectional Survey
Tetrosyl Ltd & Bevis Green
Walmersley Old Rd & BL9 6RE

Surveyed	AL AF	Drawn	ME dlp
Date	February 2018	Checked	AL
File name	18583_2D	Revision No.	Sheet size A1
Drawing No	SSL:18583:200:1:2		

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www.survsys.co.uk





Topographical Survey Legend

B	Bollard	h	Height	RWP	Rain Water Pipe	Level Prefix Descriptions	
BS	Bus Stop	Hyd	Hydrant	SP	Soil Pipe	AL	Arch Level
BT	Telecommunications	IC	Inspection Cover	SFS	Stone Faving Slabs	BL	Bed Level
CPS	Concrete Paving Slabs	Jb	Junction Box	ST	Stop Tap	CL	Cover Level
CTV	Cable Television Cover	KO	Koel Outlet	SLS	Stone Laying Slabs	EL	End Level
DP	Downpipe	LB	Letter Box	SV	Stop Valve	FL	Floor Level
DK	Drain	LP	Lamp Post	TCS	Telephone Call Box	IL	Invert Level
ELC	Electrical IC	Mx	Mile Marker	TL	Traffic Light	OL	Overhead Cable Level
EP	Elec. Pole	MP	Mile Post	TP	Telegraph Pole	PL	Parapet Level
FR	Earth Road	NP	Name Plate	TFS	Traffic Faving Slabs	RL	Roadbridge Level
FP	Flagpole	P	Post	VP	Vent Pipe	SL	Soft Level
FT	Floodlight	PM	Parking Meter	WM	Water Meter	Spr	Arch Springing Level
G	Gully	RE	Rodding Eye	WO	Water Outlet	TF	Top Of Fence Level
GV	Gate Post	RS	Road Sign	UTL	Unable To Lift	Thr	Threshold Level
GV	Gate Valve	RTW	Retaining Wall			WL	Water Level

Fence Annotation
B/W Barbed Wire
C/B Closed Boarded
C/A Corrugated Iron
C/L Chain Link
C/P Concrete Panel
H/R Hand Rail
I/R Iron Railings
O/B Open Boarded
P/C Post & Chain
P/R Post & Rail
P/W Post & Wire
W/M Wire Mesh

Gate
Site
Hedge
Limit of Foliage
& Tree Canopy

Spot Level
Survey Control Station (coordinated point)
Tree
MT Multi Trunk

Levels defining edge of carriageway are observed at channel (bottom of kerb), Unless otherwise stated.

Survey Station Coordinates

Stn	Easting	Northing	Level
0001	381213.74	414233.78	156.56
0002	381186.27	414195.48	155.21
0003	381172.64	414173.37	154.83
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mail@survsys.co.uk
www.survsys.co.uk

Levels are related to OSGB15
Value n/a
Tied to OS datum by GPS, see note.

Coordinates are related to National Grid (GPS)

Scale
0 2 4 6 8 10 12 14 16 18
metres
1:200

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Client **Barratt Homes Manchester**

Topographical & Sectional Survey
Tetrosyl Ltd & Bevis Green
Walmersley Old Rd BL9 6RE

Surveyed **AL AF** Drawn **ME dlp**
Date **February 2018** Checked **AL**
File name **18583_2D** Revision No. Sheet size **A1**
Drawing No **SSL:18583:200:2:2**

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WARNING TO HOUSE-PURCHASERS
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Buyers are warned that this is a working drawing and is not intended to be treated as descriptive material describing, in relation to any particular property or development, any of the specified matters prescribed by any Order made under the above Act. The contents of this drawing may be subject to change at any time and alterations and variations may occur during the progress of the work without revision of the drawing. Consequently **NO CONTRACT SHALL BE CONSIDERED** until the construction may differ materially from those shown here. The contents of this drawing constitute a contract, part of any contract or warranty.

Lighting Column:

Proposed galvanneal tubular steel lighting column of 6.0 metre nominal height with a planted base as supplied by Mullalite to Bury Metropolitan Council Street Lighting Specification.

Manufacturer Reference: 0106MDPT

Luminaire:
Proposed post top mounted Philips DigiStreet luminaire

Manufacturer Ref: BGP760 20LED DW50

Lumen Output: 6.000lm

Colour Temperature: Cool White

Luminaire Tilt Inclination: 0°

Control Gear: DALI enabled driver

Control Type: Philips CityTouch node 35lux (1:0.5 ratio)

Dimming Profile: SEE SCHEDULE

Supply: DNO

Primary Isolator: DNO Approved Double Pole

Secondary Isolator: N/A

Internal Wiring: Wiring to luminaire shall be 1.5mm² PVC insulated flexible cable

Door Orientation: Side Road - Facing road

Lighting Column:

Proposed galvanneal tubular steel lighting column of 6.0 metre nominal height with a planted base as supplied by Mullalite to Bury Metropolitan Council Street Lighting Specification.

Manufacturer Reference: 0106MDPT

Luminaire:
Proposed post top mounted Philips DigiStreet luminaire

Manufacturer Ref: BGP760 20LED DX10

Lumen Output: 6.000lm

Colour Temperature: Cool White

Luminaire Tilt Inclination: 0°

Control Gear: DALI enabled driver

Control Type: Philips CityTouch node 35lux (1:0.5 ratio)

Dimming Profile: 80%

Supply: DNO

Primary Isolator: DNO Approved Double Pole

Secondary Isolator: N/A

Internal Wiring: Wiring to luminaire shall be 1.5mm² PVC insulated flexible cable

Door Orientation: Side Road - Facing road

Lighting Column:

Existing lighting column to be replaced.

Manufacturer Reference: N/A

Luminaire:
N/A

Manufacturer Ref: N/A

Lumen Output: N/A

Colour Temperature: N/A

Luminaire Tilt Inclination: N/A

Control Gear: N/A

Control Type: N/A

Dimming Profile: N/A

Supply: DNO

Primary Isolator: DNO Approved Double Pole

Secondary Isolator: N/A

Internal Wiring: Wiring to luminaire shall be 1.5mm² PVC insulated flexible cable

Door Orientation: Side Road - Facing road

ENGINEERING LAYOUT NOTES

1. All adoptable drainage works have been designed and are to be constructed in accordance with 'Sewers for Adoption (8th Edition) and United Utilities' Guidelines for Sewers for Adoption (8th Edition). Where specification conflicts, the latter shall prevail.

2. All pipe work shall be in Extra Strength Clayware to BS 255 and BS 65 (SW pipe only).

3. All proposed concrete pipework shall be to Class 120 in accordance with BS611 Part 1, BS EN 1916 and bear the BS 1204 mark.

4. All adoptable drainage to be installed in Class 5 granular surround unless otherwise stated.

5. All concrete manholes and soakaways pipes, concrete cover slabs and Cms to be manufactured to BS EN 1917 and BS 5913 Part 1.

6. Rising Mains to be Black Polyethylene Pipes complying to BS EN 12242-2. Polyethylene fittings, including fusion joints, and electrician fittings shall comply with BS EN 12242-2.

7. All levels relate to Ordnance Datum. Contractor to ensure that this drawing is read in conjunction with the site specific Topographical Survey provided by Barratt Manchester and the benchmark information provided.

8. This drawing is to be read in accordance with all other relevant drawings.

9. The contractor shall be responsible for ensuring that any existing level indicators on the drawings are correct before commencing work.

10. All proposed connections to the sewer shall be 150mm unless stated otherwise.

11. All private house drainage shall be 150mm and all drop-out connections shall be 150mm of a minimum gradient of 1:80 unless otherwise stated and laid in accordance with Part H of the Building Regulations.

12. Runoff from private surfaces shall not discharge onto the highway. Gullies or channels shall be provided as appropriate to prevent this.

13. From crossings shall be provided at the least frequent points of all junctions.

14. Pipes shall be protected from concentrated loading by construction traffic during the construction period when installation cover to the pipe may make them vulnerable to damage.

15. Initial DMR tests of the road formation level are to be carried out to determine the depth of pavement construction required. This is to be approved by the adopting authority prior to construction of the road pavement.

16. Contractor to ensure that plot drainage be within the curbside of the plot they serve where possible and inspection covers kept while working where possible.

17. Contractor to provide limited access to the sewer with sufficient access prior to commencement of sewer works on their inspection telephone number: Tel 0845 602 0406.

18. Contractor to obtain all necessary highway opening notices from the relevant Local Authority, obtain approval to work on United Utilities Sewerage system, obtain approval to install drainage from the Environment Agency for any works affecting a watercourse.

ENGINEERING KEY

Adoptable S.W. Sewer & Manhole Adoptable F.W. Sewer & Manhole

S. Water Drop-Out and Invert Foul Drop-Out and Invert

Adoptable Road Gully Yard Gully

Street Name Plate Drainage Channel

MH Ref.	Max. Output	Details
S38	139/s	150mm dia orifice plate, 0.6 discharge coefficient
S48	82/s	150mm dia orifice plate, 0.6 discharge coefficient
S54	94/s	175mm dia orifice plate, 0.6 discharge coefficient
S58	93/s	200mm dia orifice plate, 0.6 discharge coefficient

L Amended to suit latest planning layout rev 11.06.20 DK
L polystyrene crates outside plot 144 amended to fit within POS.
K Street Lighting design added. 14.01.20 DK
J Polystyrene crate volume increased for SW 14.12.19 DK
H Flow control changed to orifice plates. SW Pipe 2.000 upped from 150w to 225w. 20.11.19 DK
G Planning layout rev G inserted, plots 255,266 lifted to suit Cole Easdon's river modelling requirements. Drainage CLs & Ls amended at the end of Road77/Road77H to tie in with lifted road levels. 18.10.19 DK
F Planning layout rev F inserted, plots 10-21 amended slightly to suit. 10.09.19 DK
E Fully amended road & sewer design added to suit latest planning layout rev E. 09.08.19 DK
D FFLs amended to suit cut & fill rev A. 02.07.19 DK
C FFLs amended to latest planning layout PL02 rev B. 04.01.19 DK
B FFLs amended to latest planning layout rev J. Drainage design removed. 10.01.18 DK
A Full road & sewer design added. 06.11.17 DK

REV DESCRIPTION	DATE	DRAWN
R Planning layout rev P used with Cole Easdon's latest proposed watercourse alignment.	12.02.21	DK
Q Culvert levels and gradients amended to tie into modelling. SW10.001 upped to 1500w, 12.000,12.001 & 13.000 downgraded to 800w. Polystyrene crates near outfall amended & orifices amended to suit new design. Gullies amended to all ramp locations. CLs amended throughout to suit latest road design.	13.01.20	DK
P Plot drainage changed to show correct RWP and soil stock positions for the Ellerton Housetype and white lining added.	12.10.20	LB
N Finished Floor levels updated to suit the latest External levels plans.	28.07.20	LB
M Cover levels updated to suit the latest planning layout rev L.	16.06.20	LB

Scale Bar

0m 10m 20m 30m 40m 50m 60m 70m 80m 90m 100m

BARRATT HOMES MANCHESTER

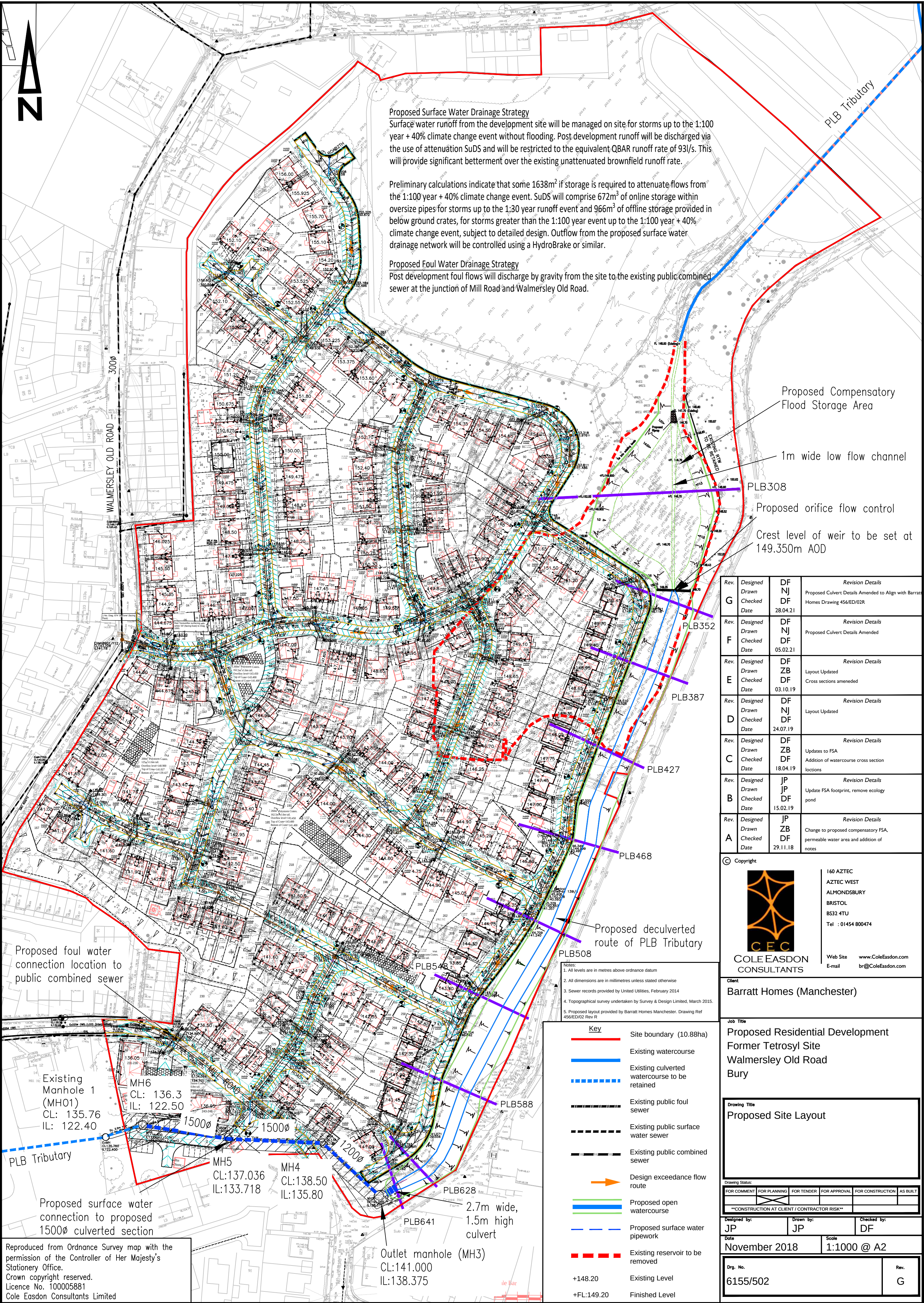
Barratt Homes Manchester
(A division of BDW Trading Ltd)
4 Bolder Road
City Park
Manchester
M16 9AD
Tel: 0161 872 0161
Fax: 0161 855 2828

Job: Tetrosyl Bury

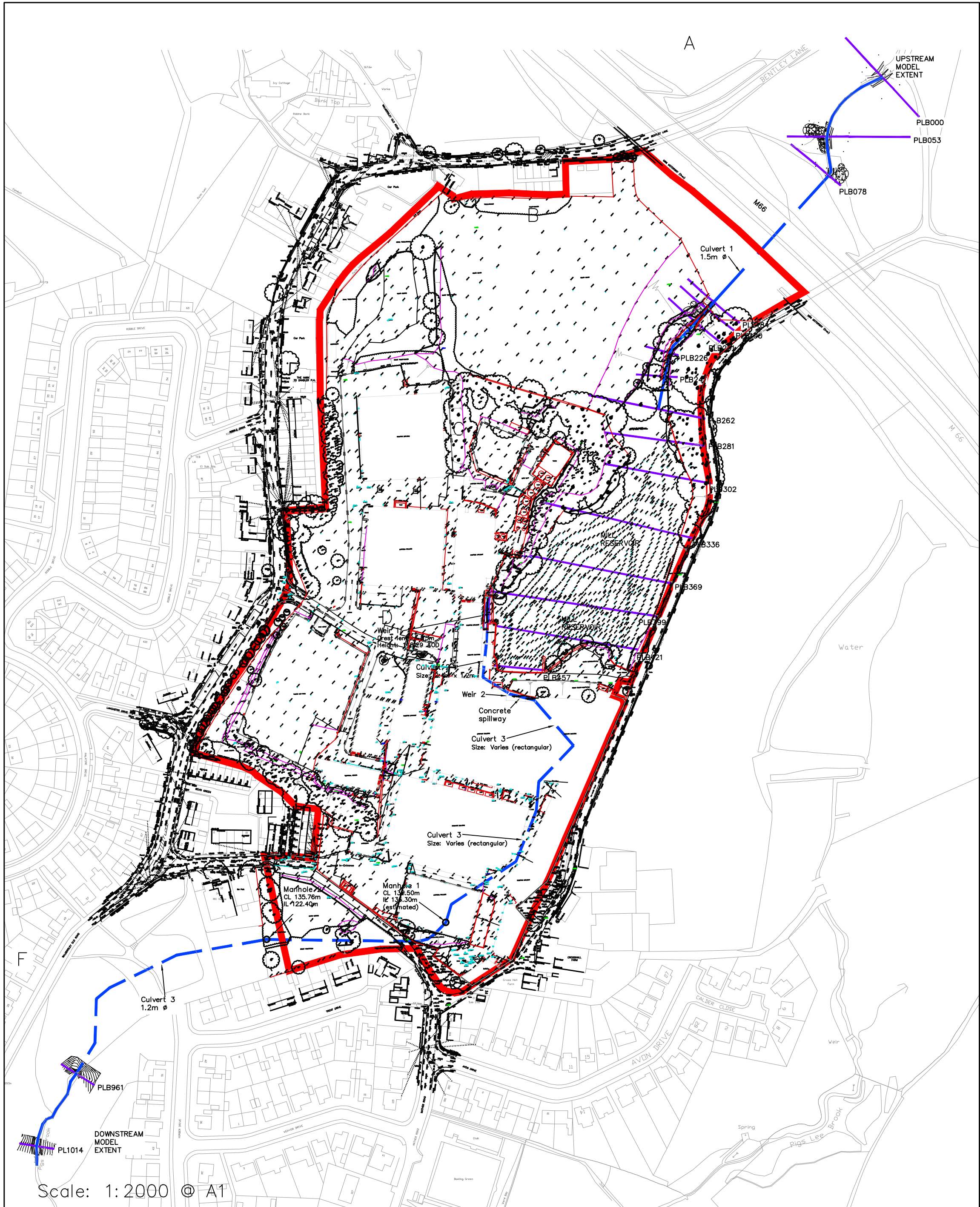
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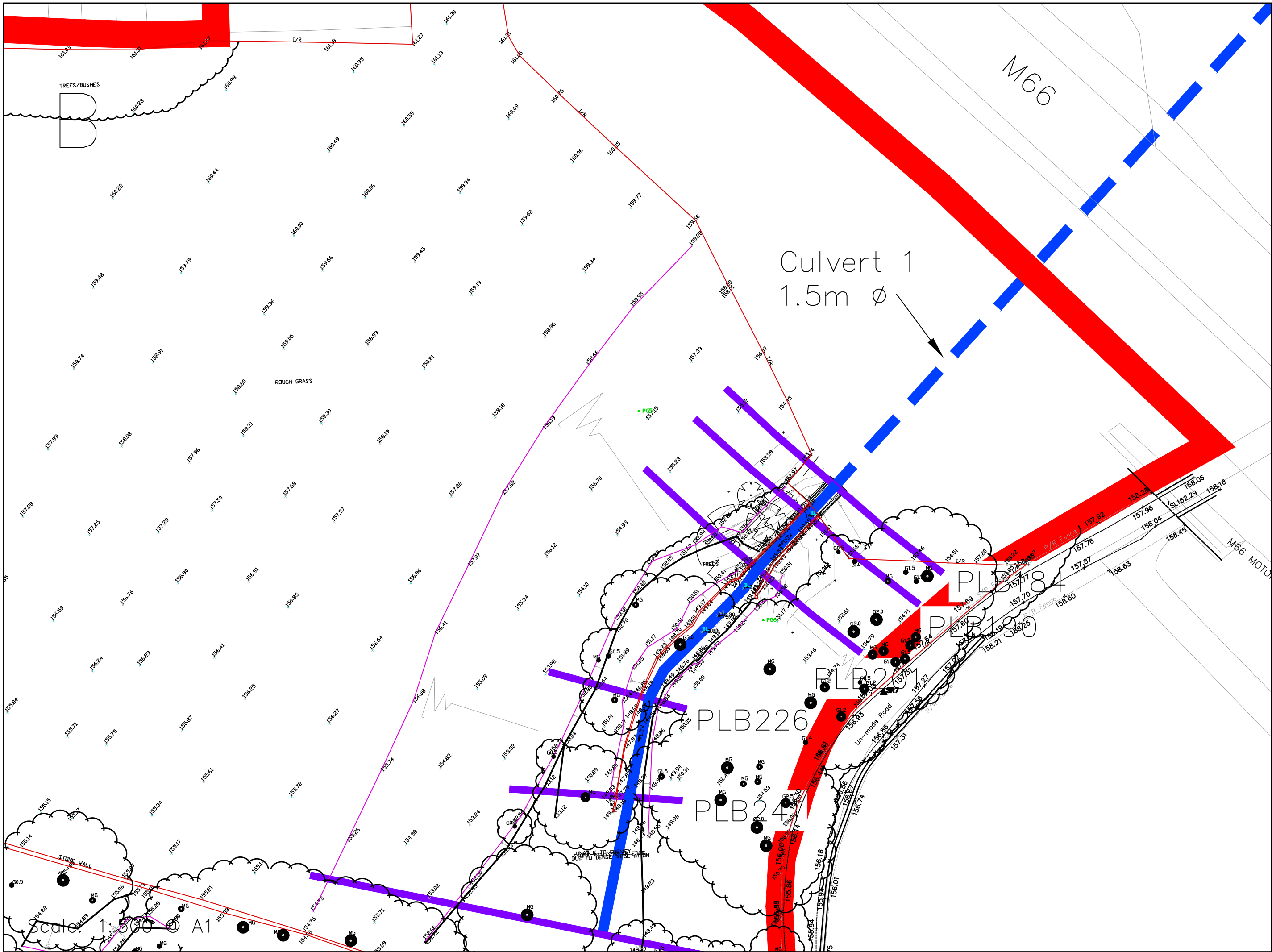
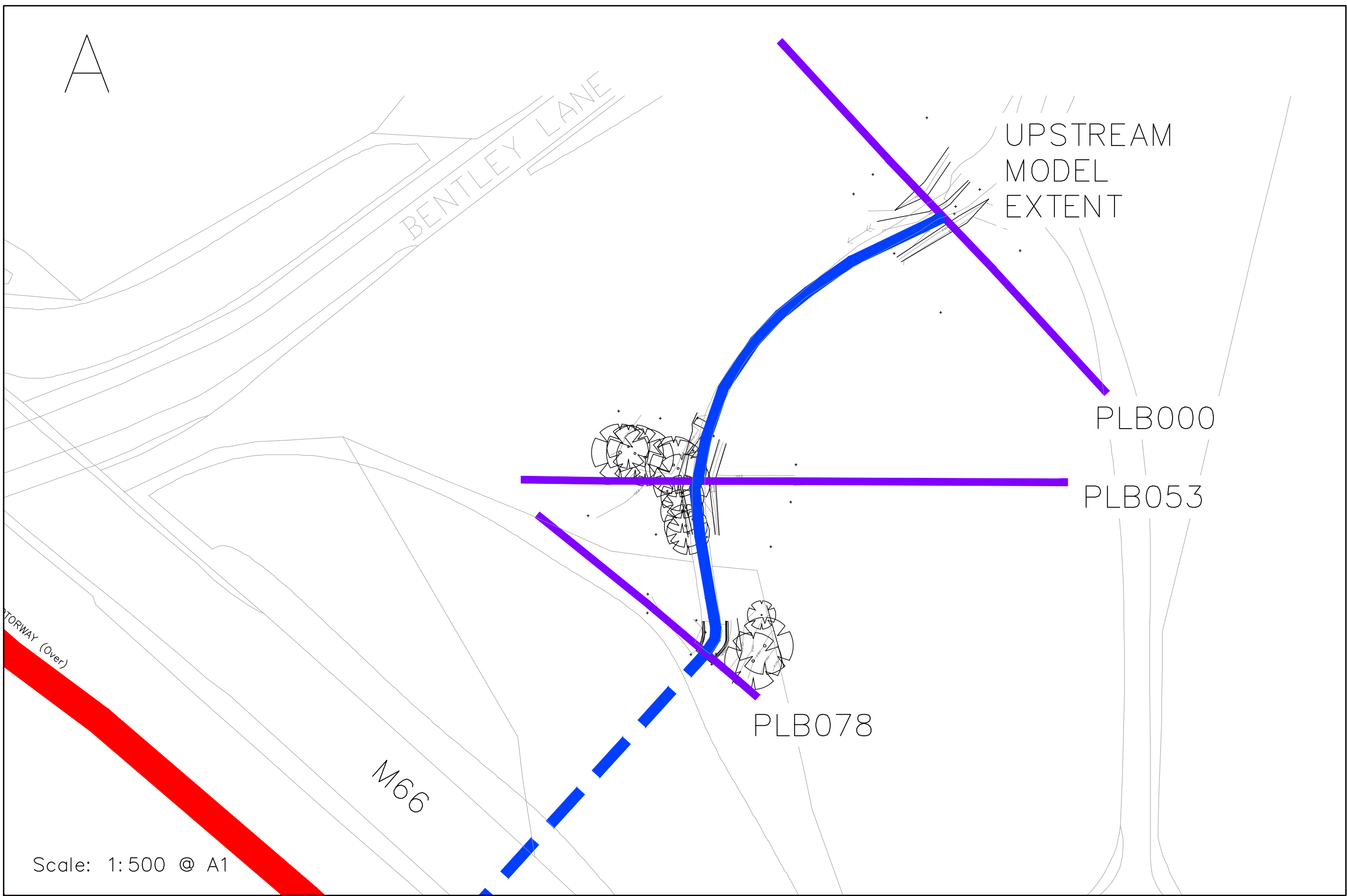
Appendix 5



N



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- Key
- Site boundary
 - Cross sections
 - Centreline of culvert of tributary of Pigs Lee Brook
 - Centreline of tributary of Pigs Lee Brook

PLB078 Model cross-section numbers

- Notes
- All levels in meters above ordnance datum.
 - Topographical survey provided by Survey & Design Ltd, March 2015.
 - Cross-section data provided by Survey and Design Ltd, March 2018.
 - LIDAR Contours not shown

B	ZB	11.10.2019	Proposed model X-section locations removed
A	ZB	23.08.2019	Weir 1 text amended
No.	By	Date	Revision Details
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		160 AZTEC AZTEC WEST ALMONDSBURY BRISTOL BS32 4TU Tel : 01454 800474	
COLE EASDON CONSULTANTS		Web Site www.ColeEasdon.com E-mail br@ColeEasdon.com	

Client
Barratt Homes Manchester

Job Title
Hydraulic Modelling Study
Former Tetrosyl Site
Walmersley Old Road
Bury
Greater Manchester

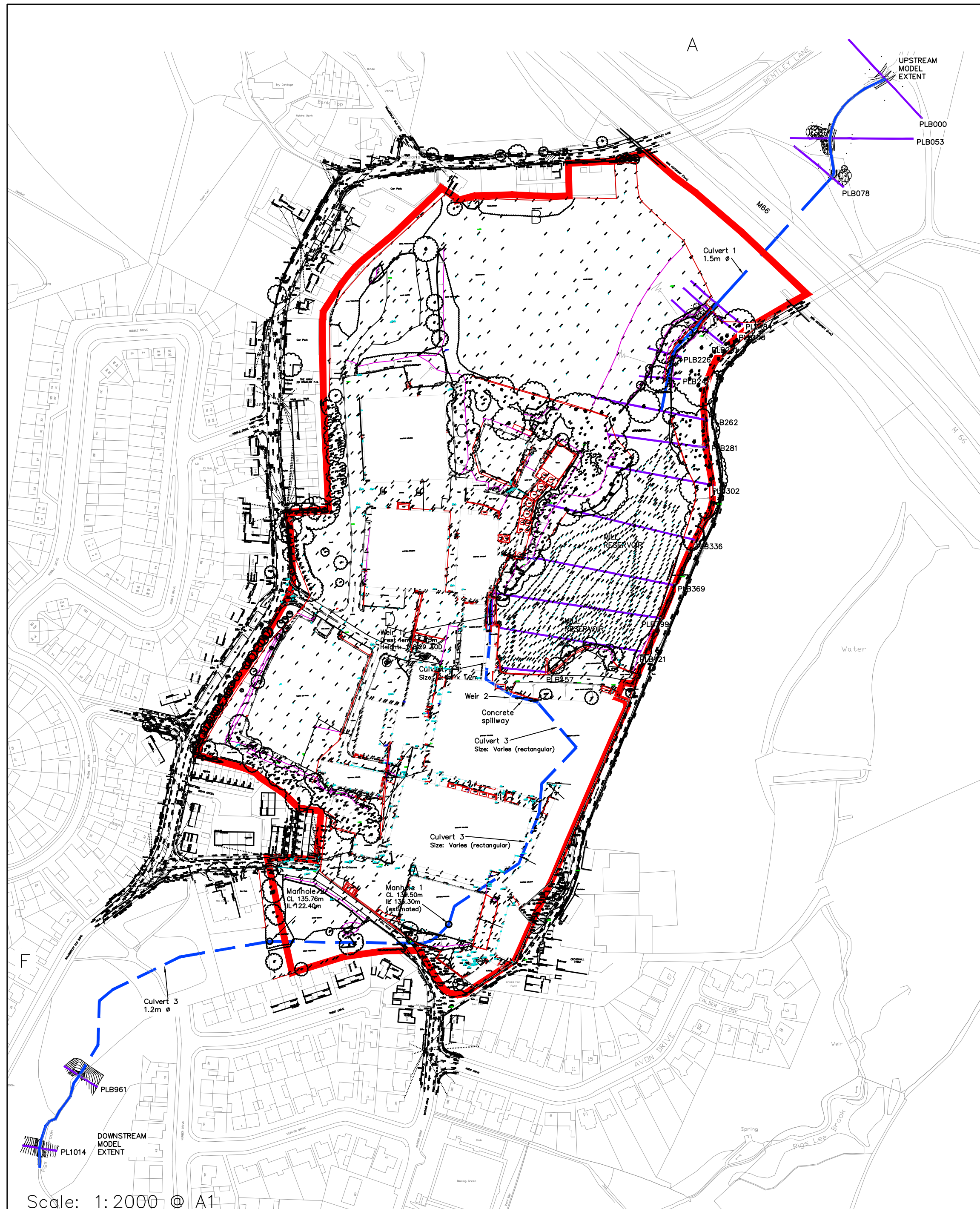
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Existing Hydraulic Model Extent

Sheet 1 of 3

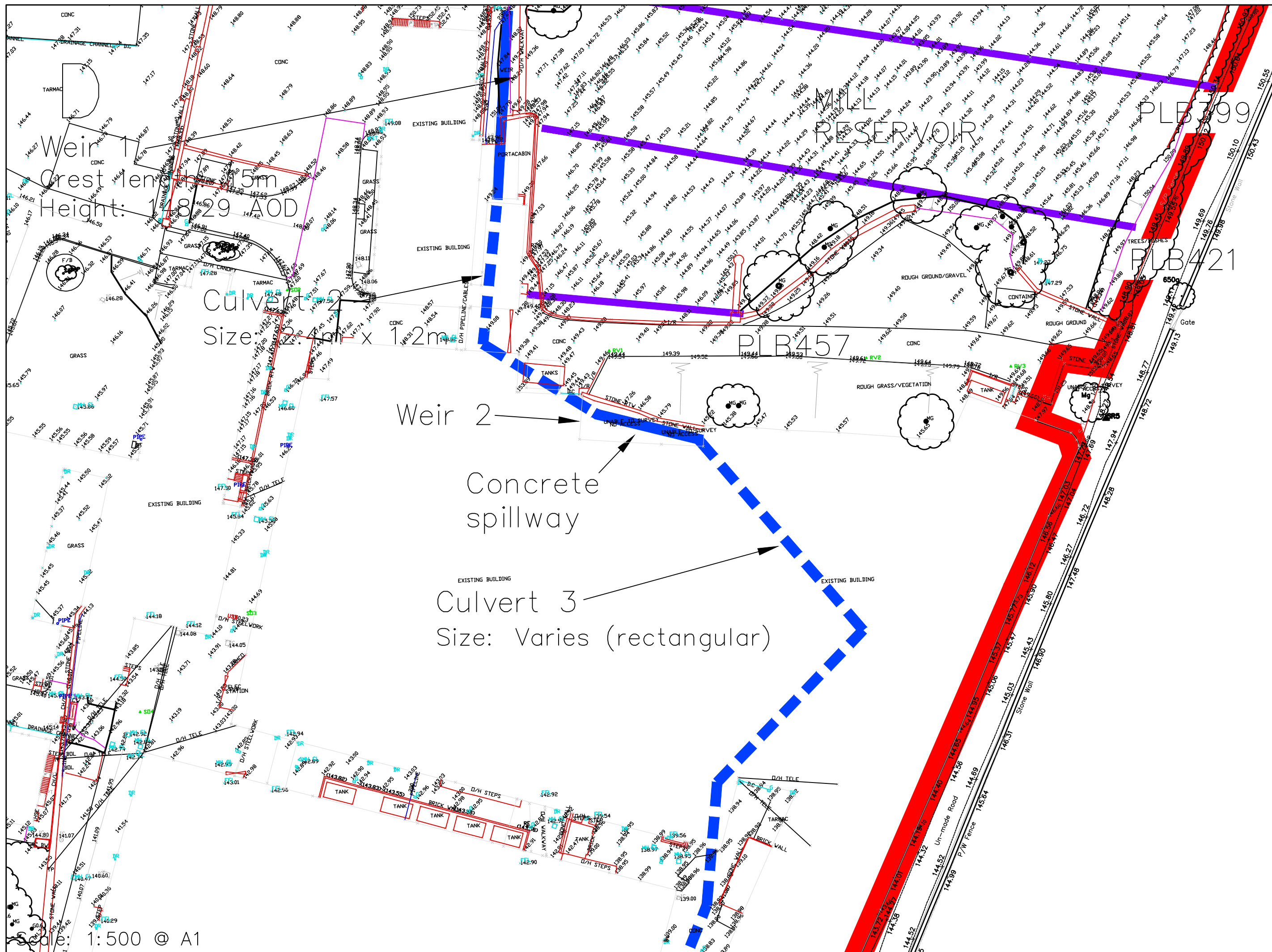
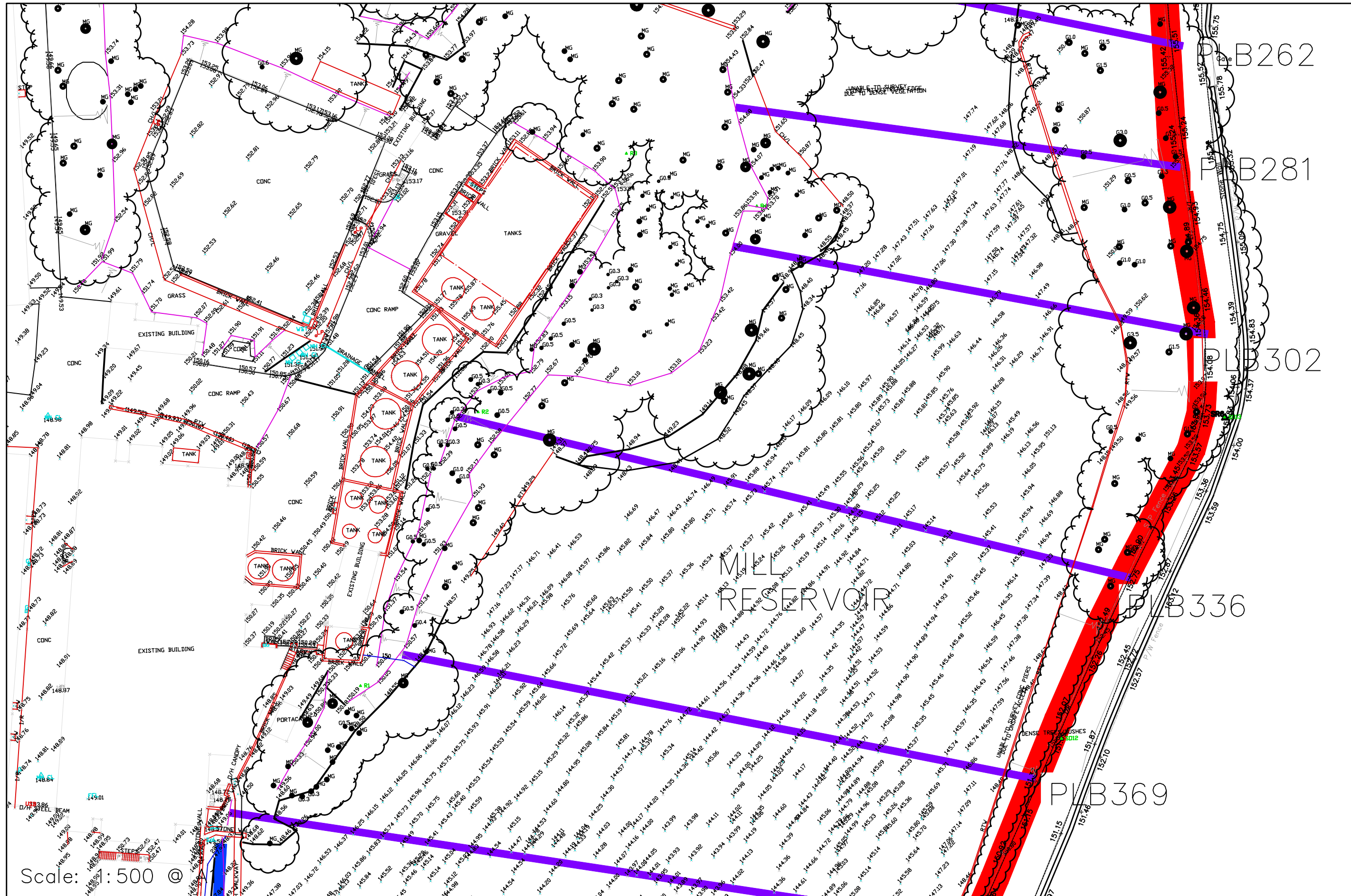
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				AS BUILT
CONSTRUCTION AT CLIENT / CONTRACTOR RISK				

Designed by JP	Drawn by JP	Checked by DF
Date December 2018	Scale As shown @ A1	

Dwg. No. 6155-503-01	Rev. B
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Key

- Site boundary
- Cross sections
- Centreline of culvert of tributary of Pigs Lee Brook
- Centreline of tributary of Pigs Lee Brook

PLB078 Model cross-section numbers

Notes

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- Cross-section data provided by Survey and Design Ltd, March 2018.
- LiDAR Contours not shown

B	ZB	11.10.2019	Proposed model X-section locations removed
A	ZB	23.08.2019	Weir 1 text amended

No.	By	Date	Revision Details
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E-mail : br@ColeEasdon.com

Client
Barratt Homes Manchester

Job Title
Hydraulic Modelling Study
Former Tetrosyl Site
Walmersley Old Road
Bury
Greater Manchester

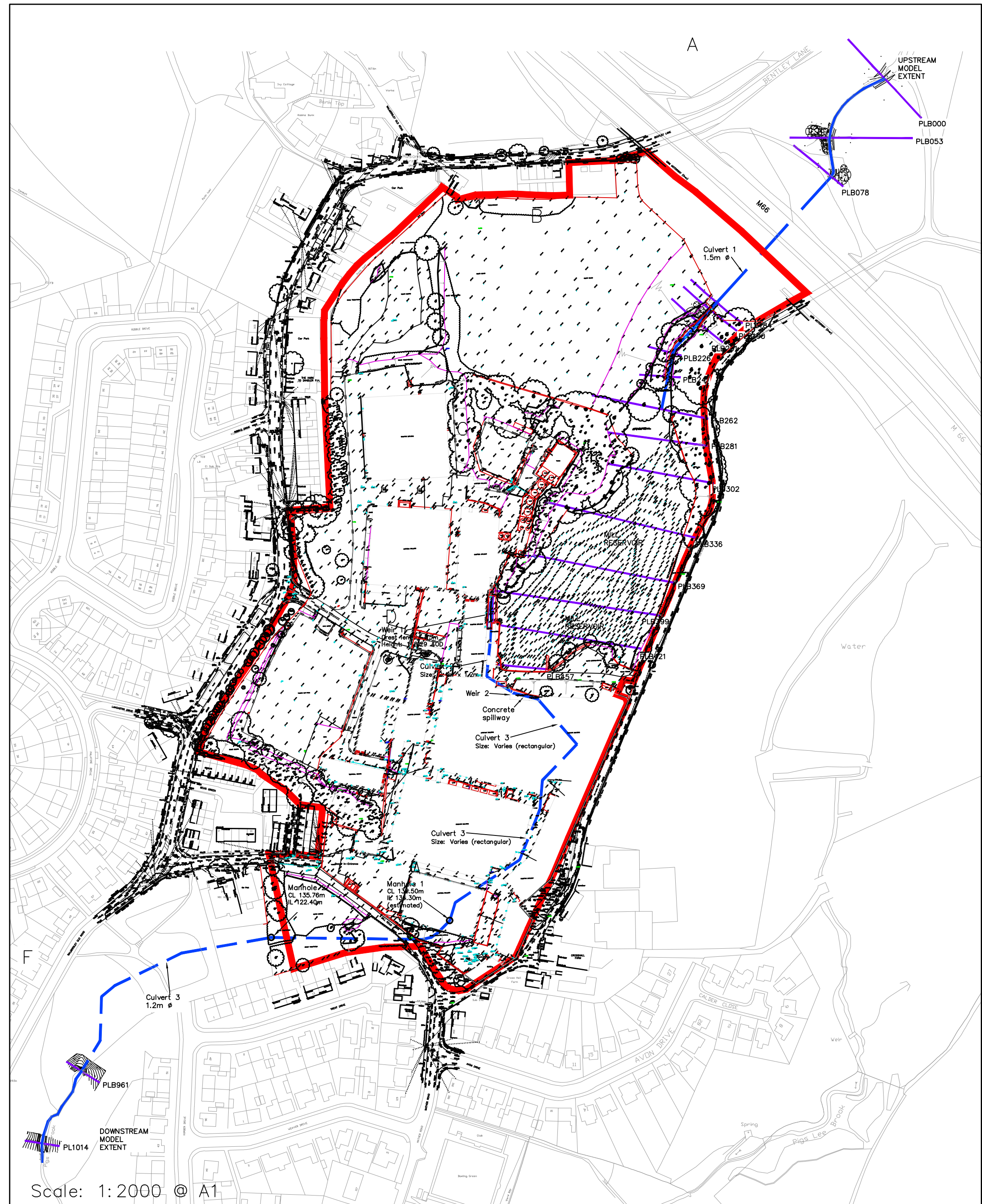
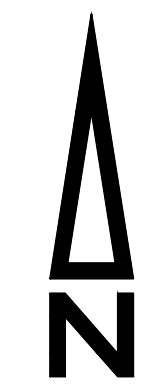
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Existing Hydraulic Model Extent

Sheet 2 of 3

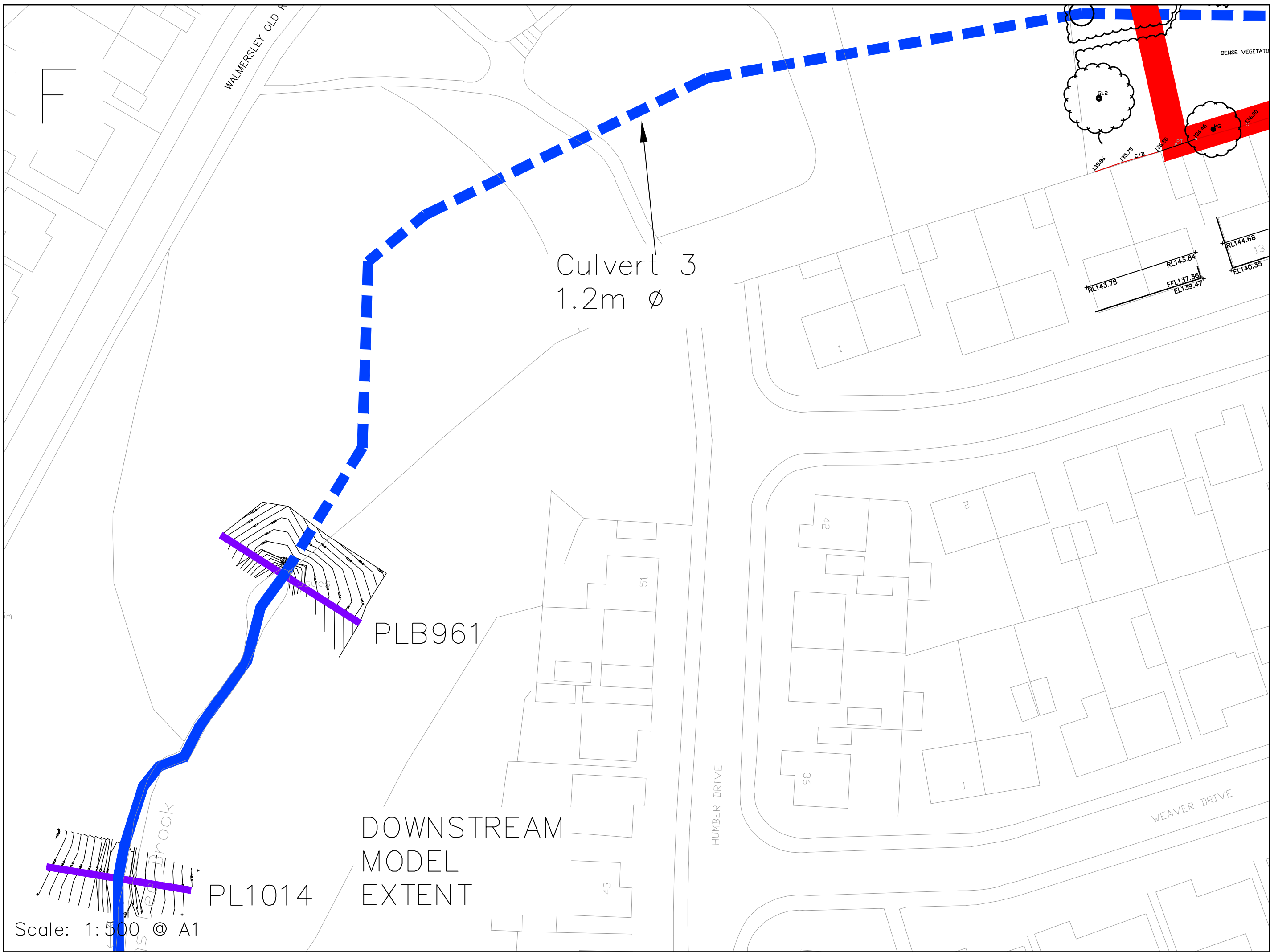
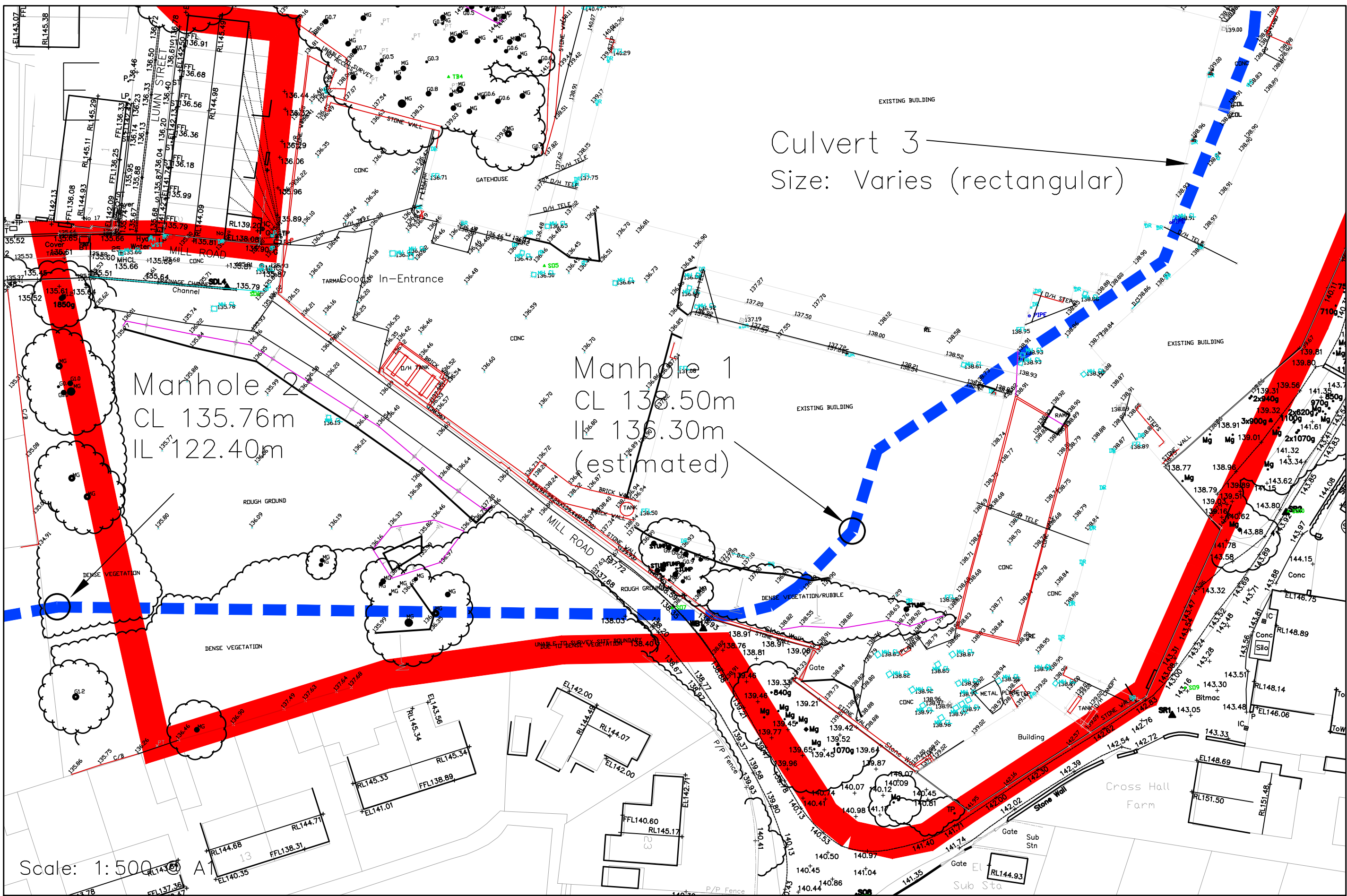
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Designed by	Drawn by	Checked by
JP	JP	DF
Date	Scale	
December 2018	As shown @ A1	

Dwg. No.	Rev.
6155-503-02	B



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No.	By	Date	Revision Details
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		160 AZTEC AZTEC WEST ALMONDSBURY BRISTOL BS32 4TU Tel : 01454 800474	
COLE EASDON CONSULTANTS		Web Site : www.ColeEasdon.com E-mail : br@ColeEasdon.com	

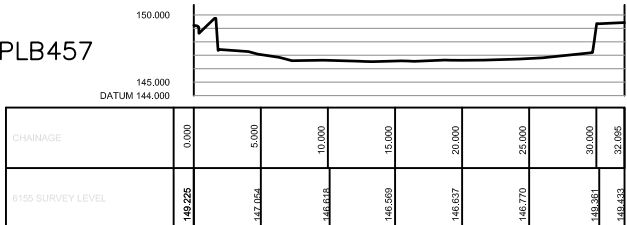
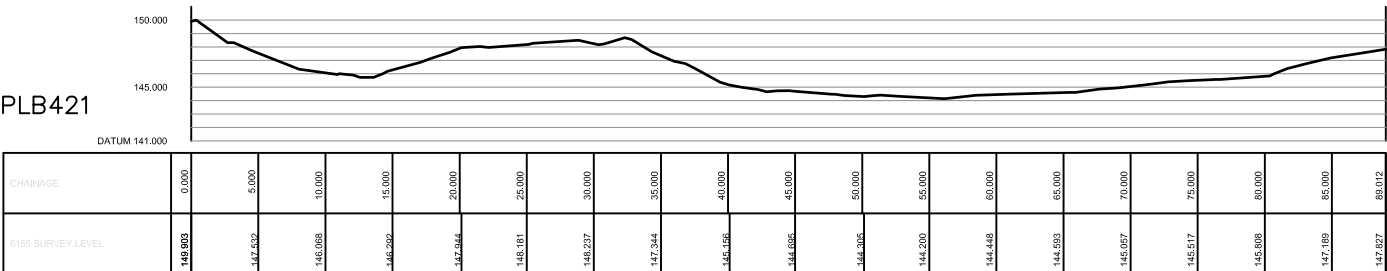
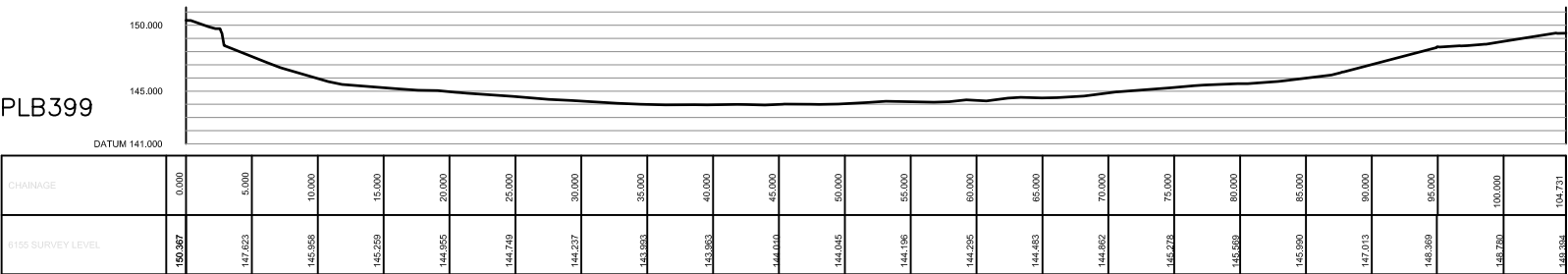
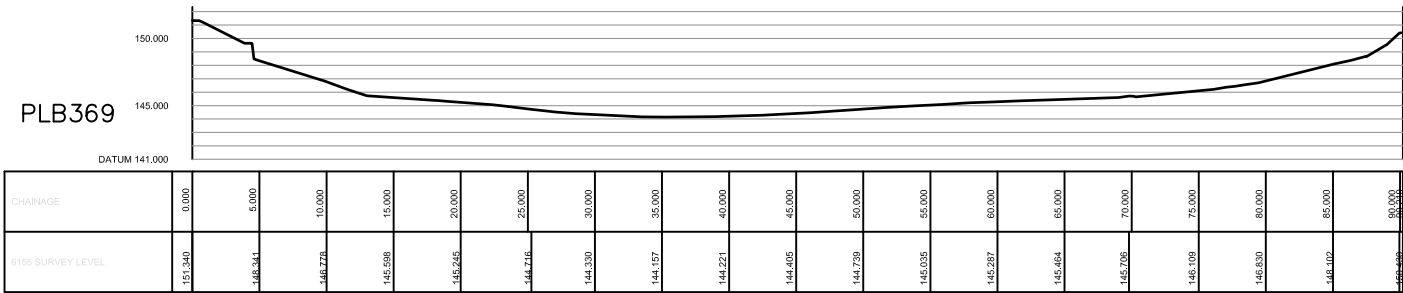
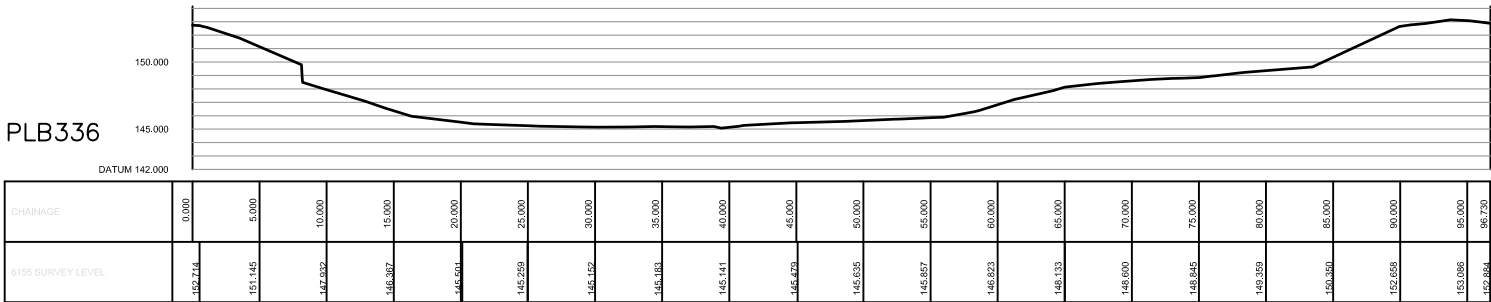
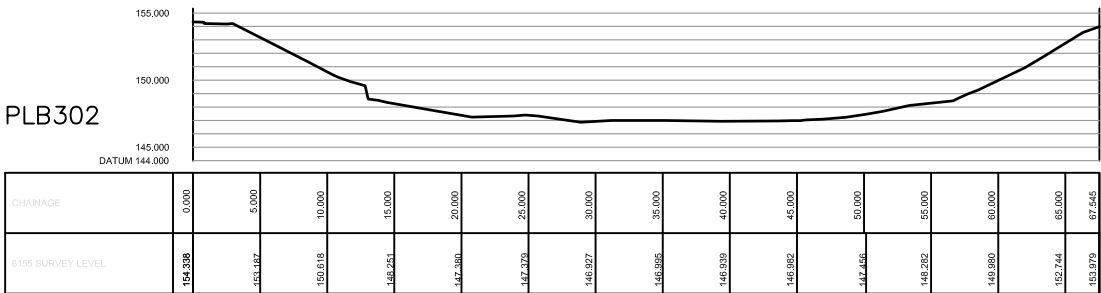
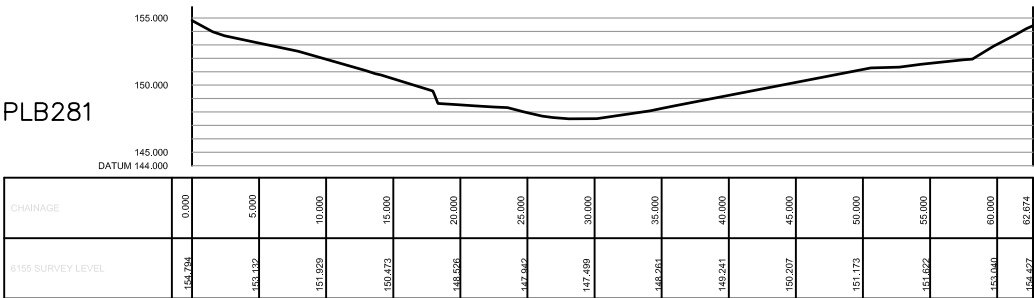
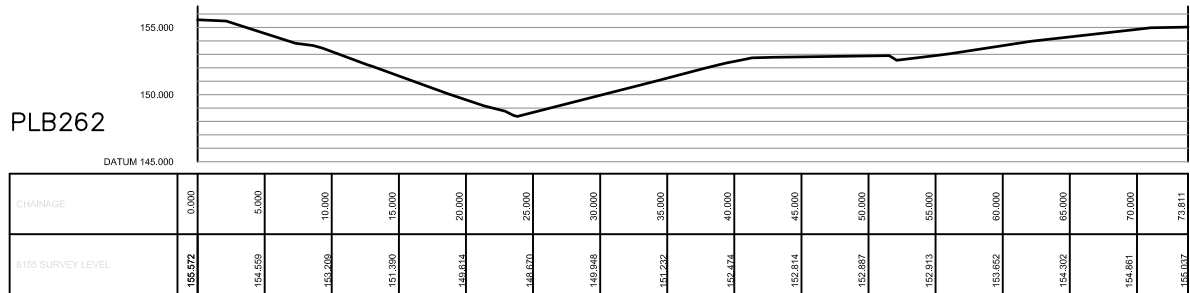
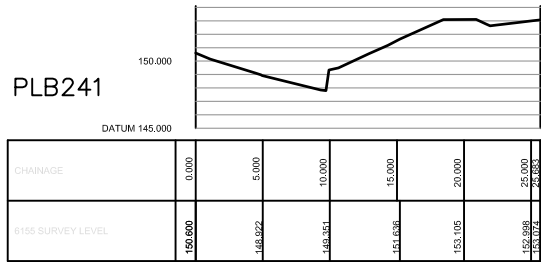
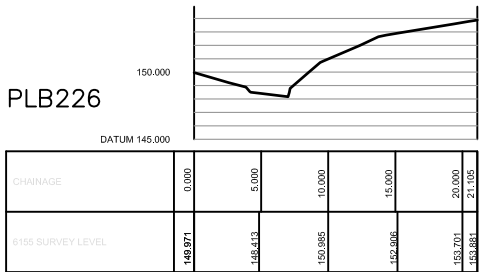
Client
Barratt Homes Manchester

Job Title
Hydraulic Modelling Study
Former Tetrosyl Site
Walmersley Old Road
Bury
Greater Manchester

Drawing Title
Existing Hydraulic Model Extent

Sheet 3 of 3

Drawing Status:				
FOR COMMENT	FOR PLANNING	FOR TENDER	FOR APPROVAL	FOR CONSTRUCTION
				AS BUILT
CONSTRUCTION AT CLIENT / CONTRACTOR RISK				
Designed by JP		Drawn by JP		Checked by DF
Date December 2018		Scale As shown @ A1		
Dwg. No. 6155-503-01				Rev. B



Notes

- All levels in meters above ordnance datum.
- All sections are viewed downstream.

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Client
Barratt Homes (Manchester)

Job Title
Proposed Development
Tetrosyl Site
Walmersley Old Road
Bury

Drawing Title
Hydraulic Modelling Study
Mill Reservoir - Existing Cross-sections

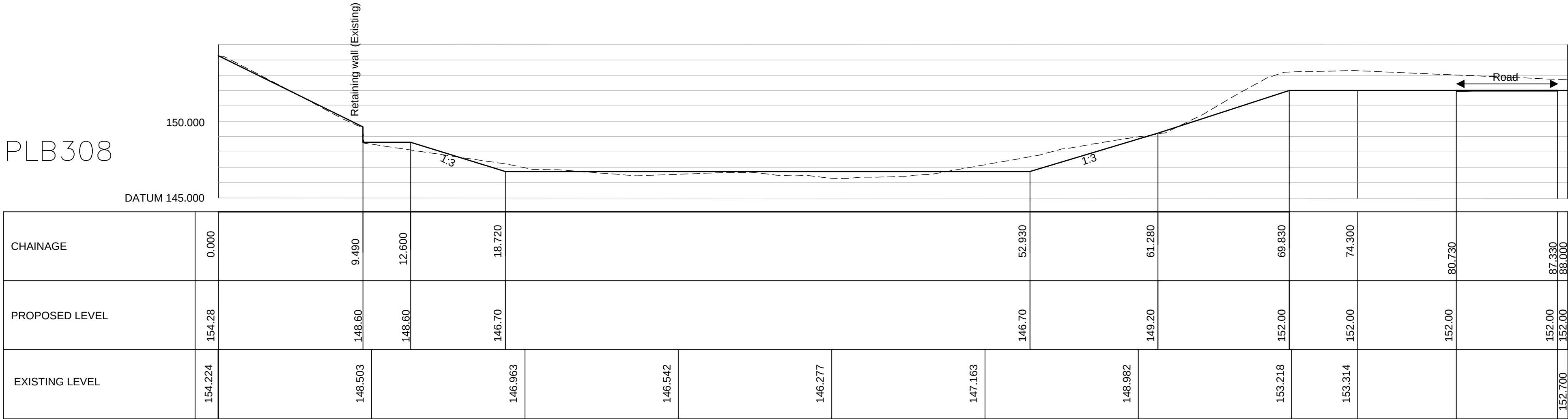
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				AS BUILT
CONSTRUCTION AT CLIENT / CONTRACTOR RISK				

Designed by: JP	Drawn by: JP	Checked by: DF
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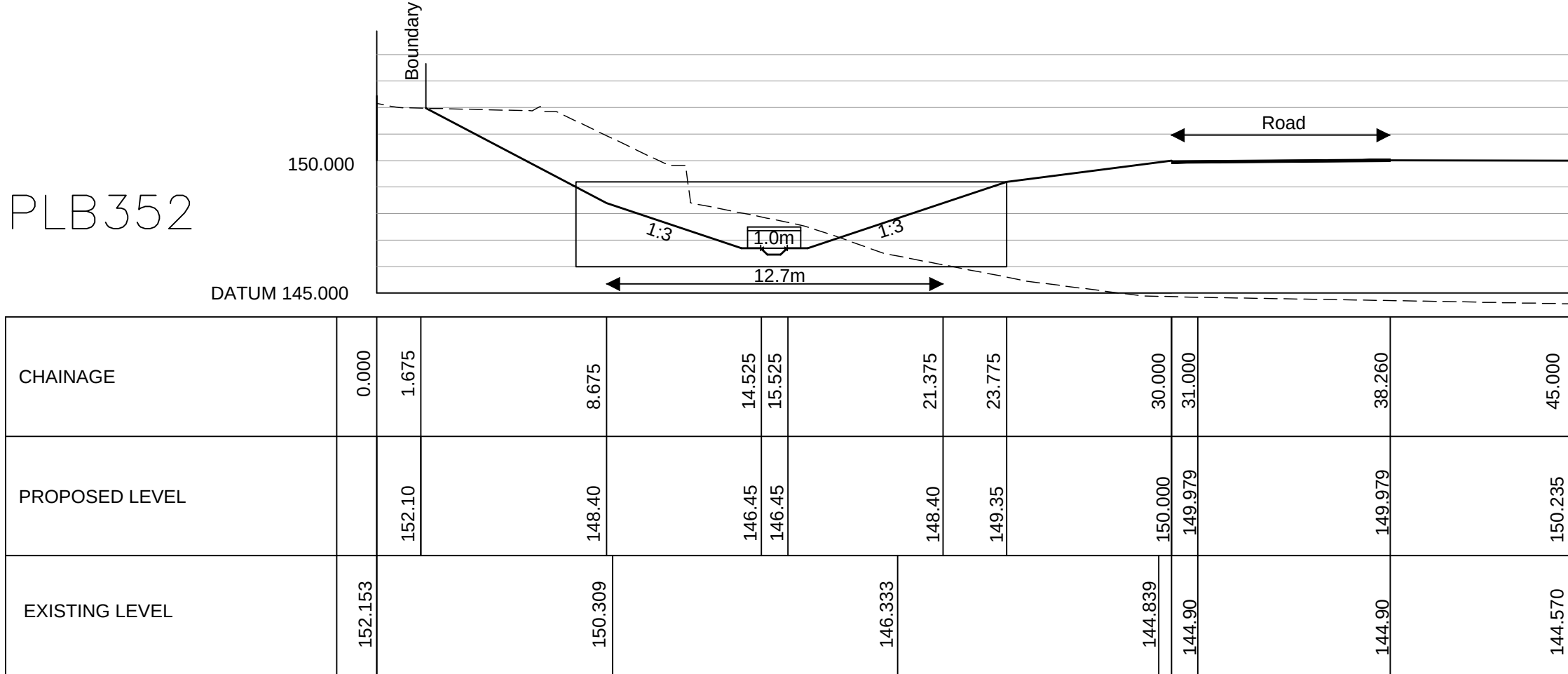
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Dwg. No. 6155/505	Rev.
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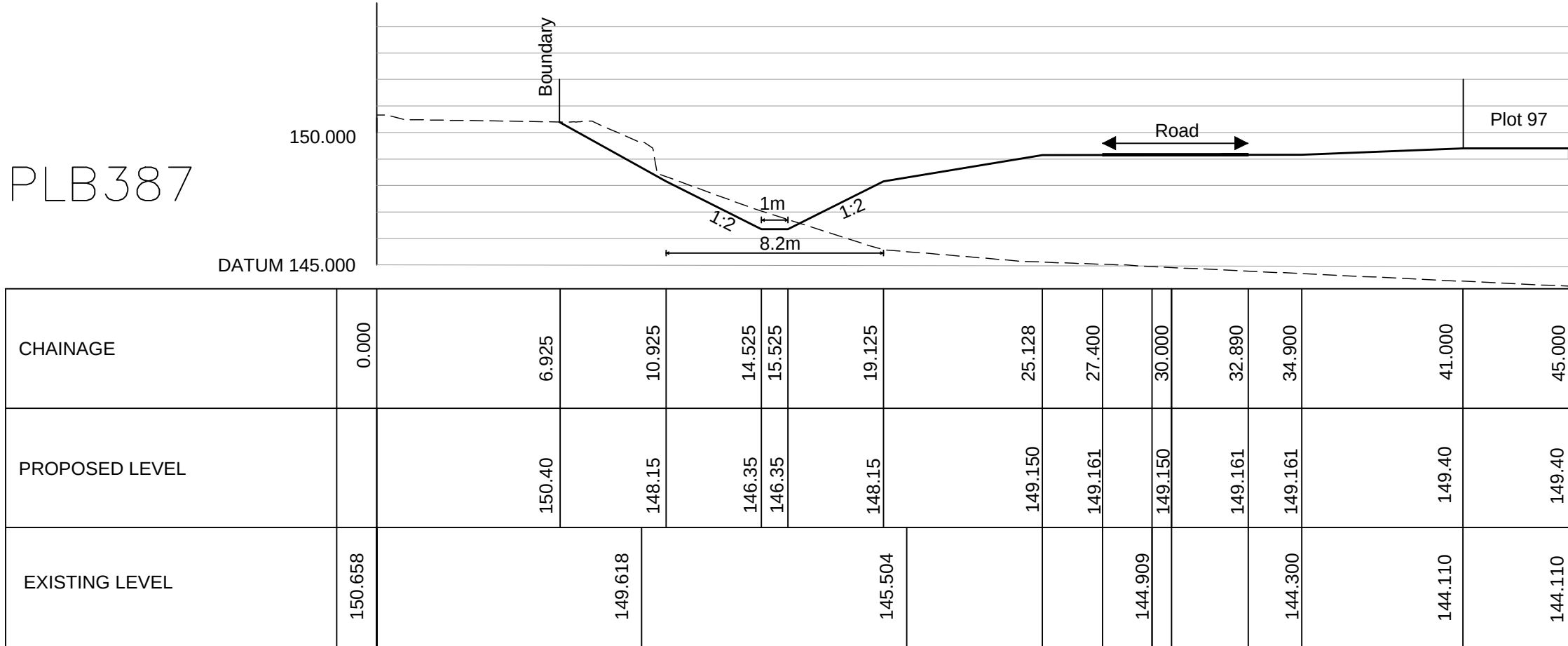
PLB308



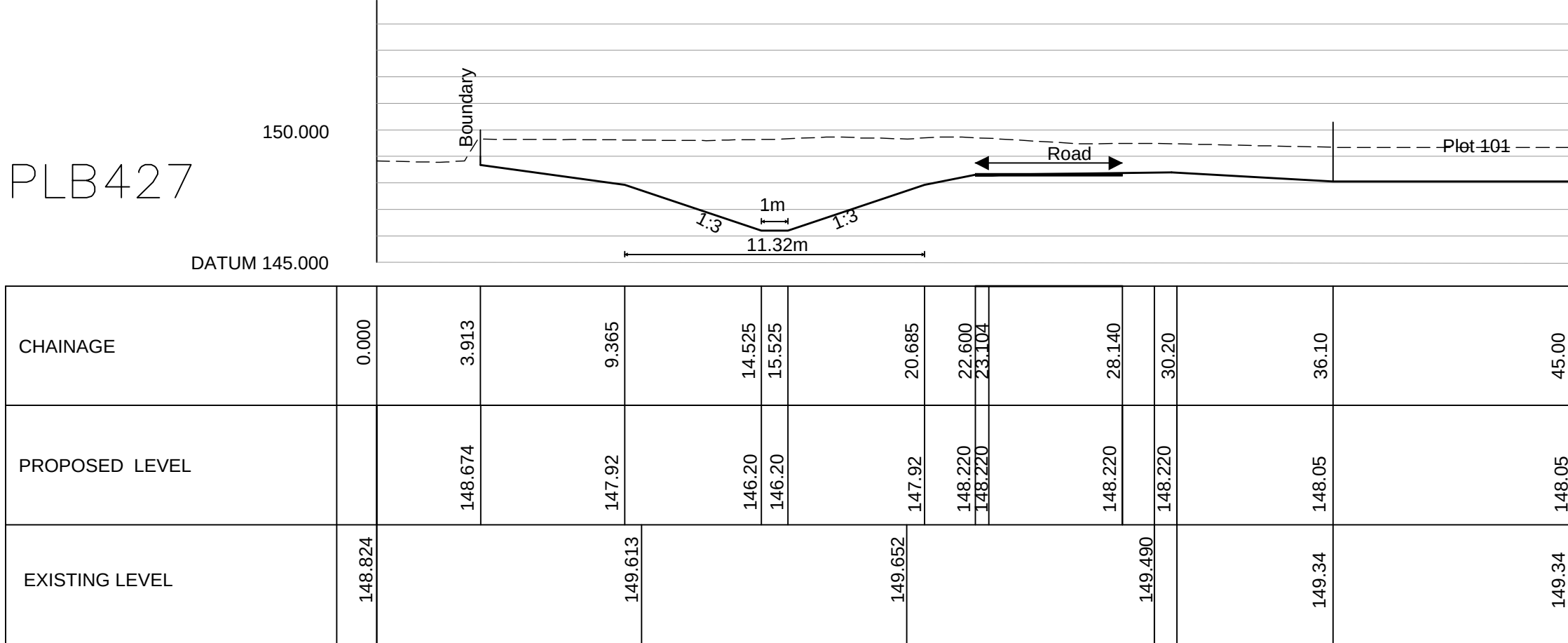
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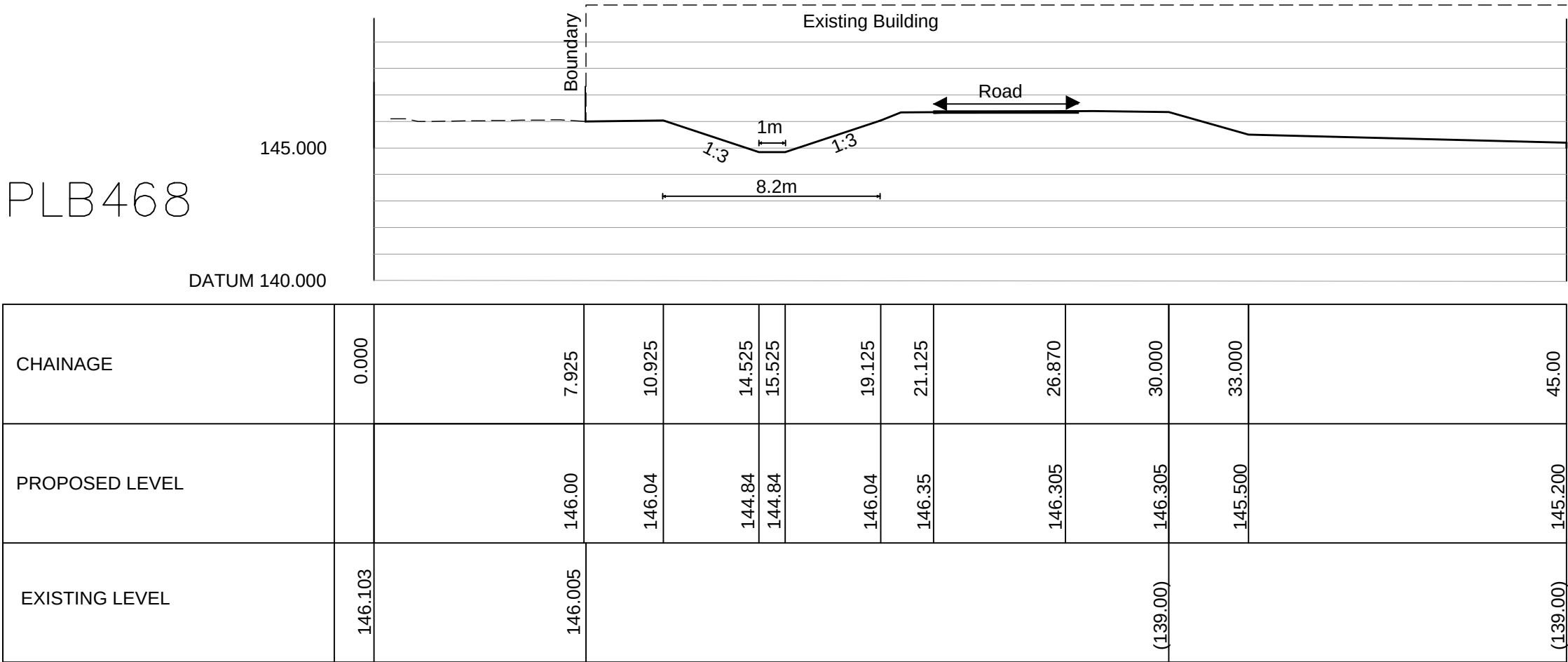
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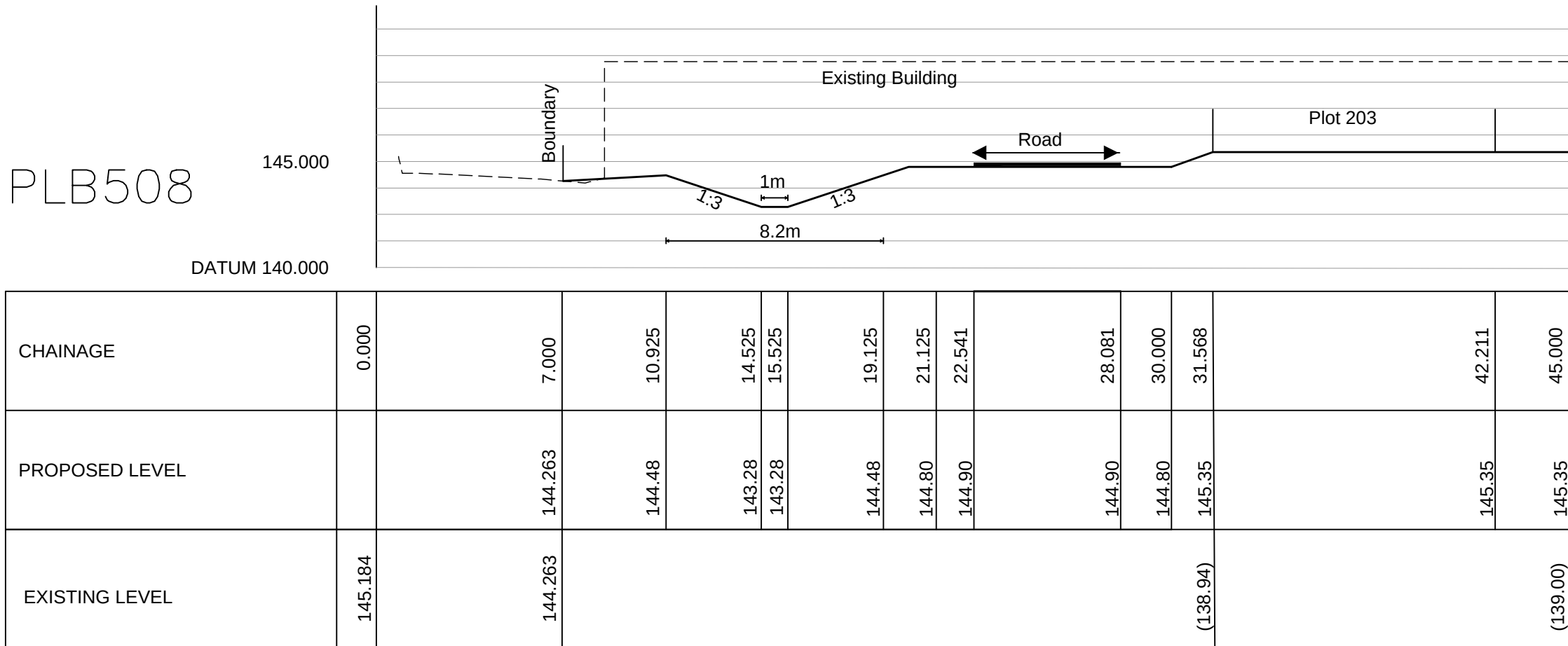
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PLB468



PLB508



Notes

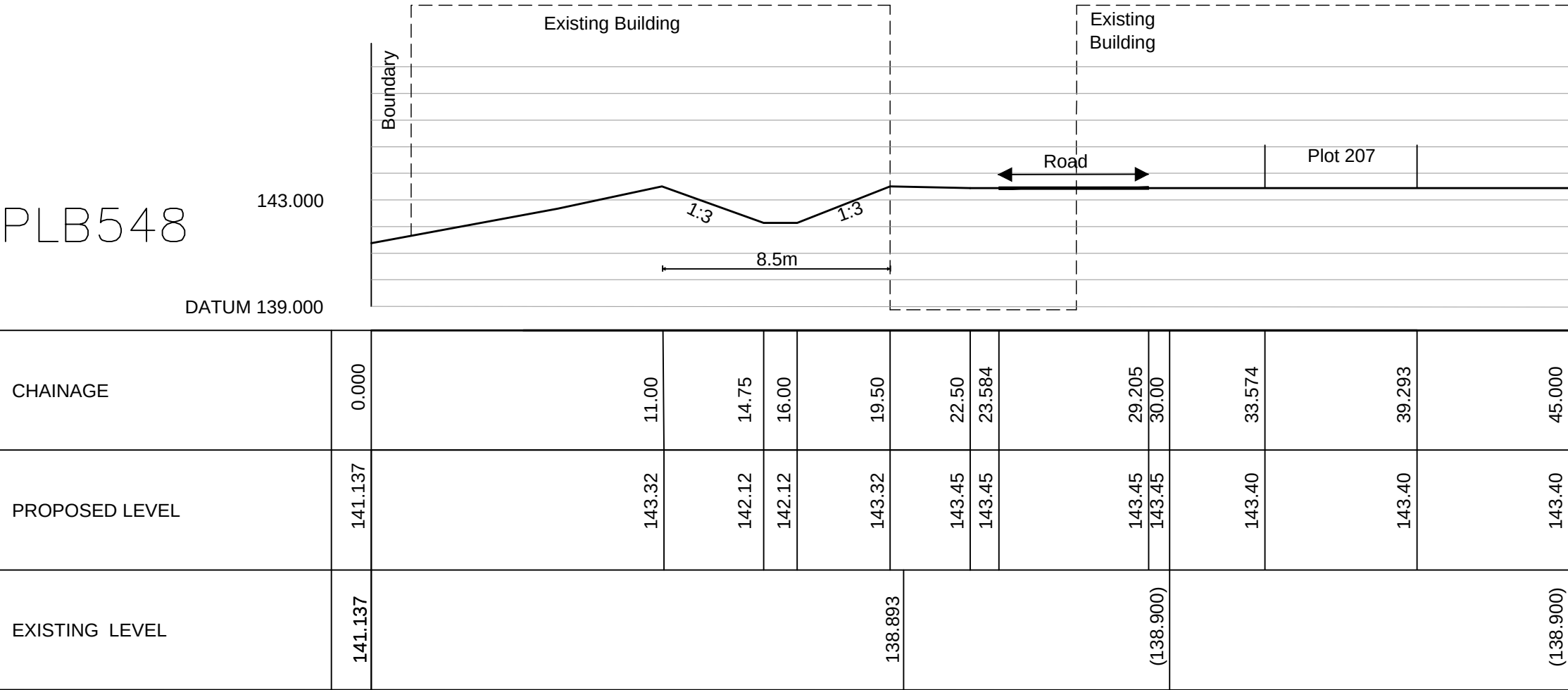
1. All levels in meters above ordnance datum.
2. All sections are viewed downstream.
3. Refer to Drawing 6155/502 for section locations

Key

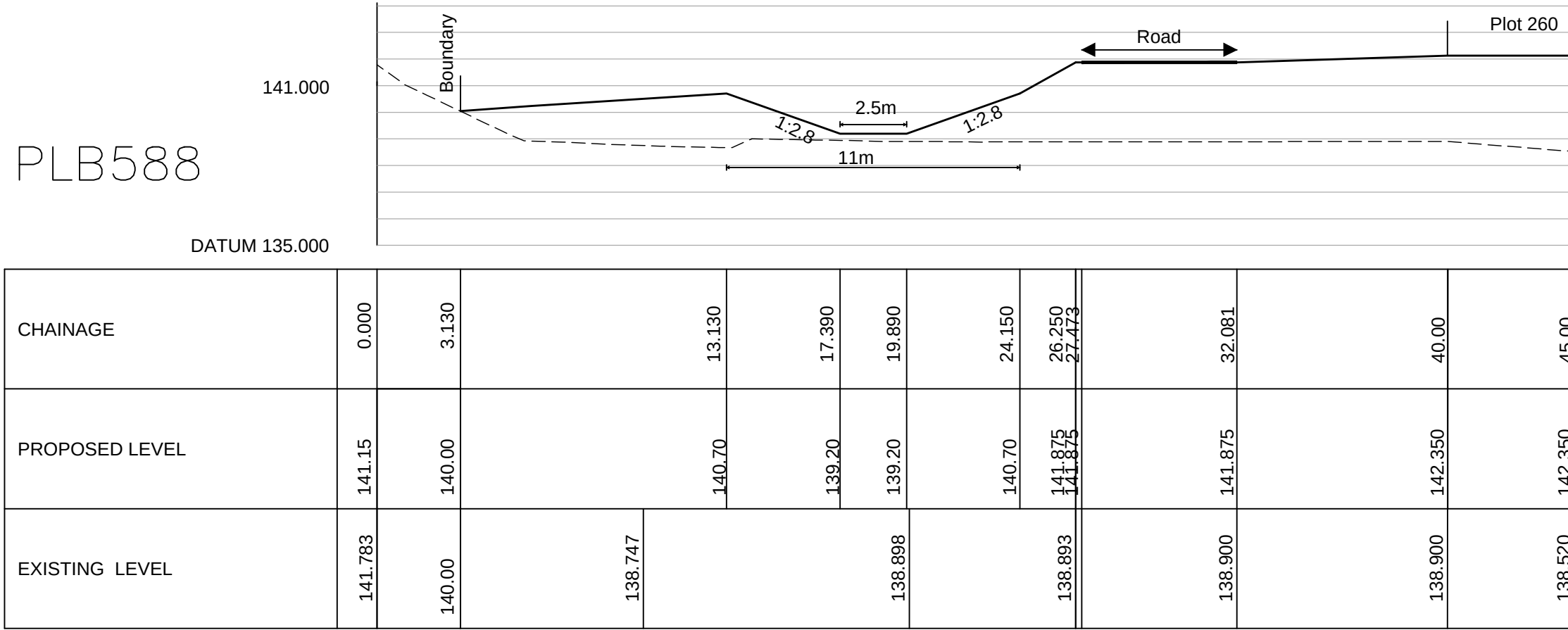
- Proposed ground profile
- - - Existing ground profile

A	SS	30.09.19	Existing ground profile added																		
No.	By	Date	Revision Details																		
 COLE EASDON CONSULTANTS YORK HOUSE EDISON PARK DORCAN WAY SWINDON WILTSHIRE SN3 3RB Tel : 01793 619965 Web Site : www.ColeEasdon.com E-mail : cec@ColeEasdon.com																					
Client Barratt Homes (Manchester)																					
Job Title Proposed Development Tetrosyl Site Walmersley Old Road Bury																					
Drawing Title Deculverted Watercourse Proposed Cross-sections Page 1 of 2																					
Drawing Status: <table><tr><td>FOR COMMENT</td><td>FOR PLANNING</td><td>FOR TENDER</td><td>FOR APPROVAL</td><td>FOR CONSTRUCTION</td><td>AS BUILT</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td colspan="6">**CONSTRUCTION AT CLIENT / CONTRACTOR RISK**</td></tr></table>				FOR COMMENT	FOR PLANNING	FOR TENDER	FOR APPROVAL	FOR CONSTRUCTION	AS BUILT							**CONSTRUCTION AT CLIENT / CONTRACTOR RISK**					
FOR COMMENT	FOR PLANNING	FOR TENDER	FOR APPROVAL	FOR CONSTRUCTION	AS BUILT																
CONSTRUCTION AT CLIENT / CONTRACTOR RISK																					
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Date April 2019		Scale 1:200@A1																			
Dwg. No. 6155/507/01					Rev. A																

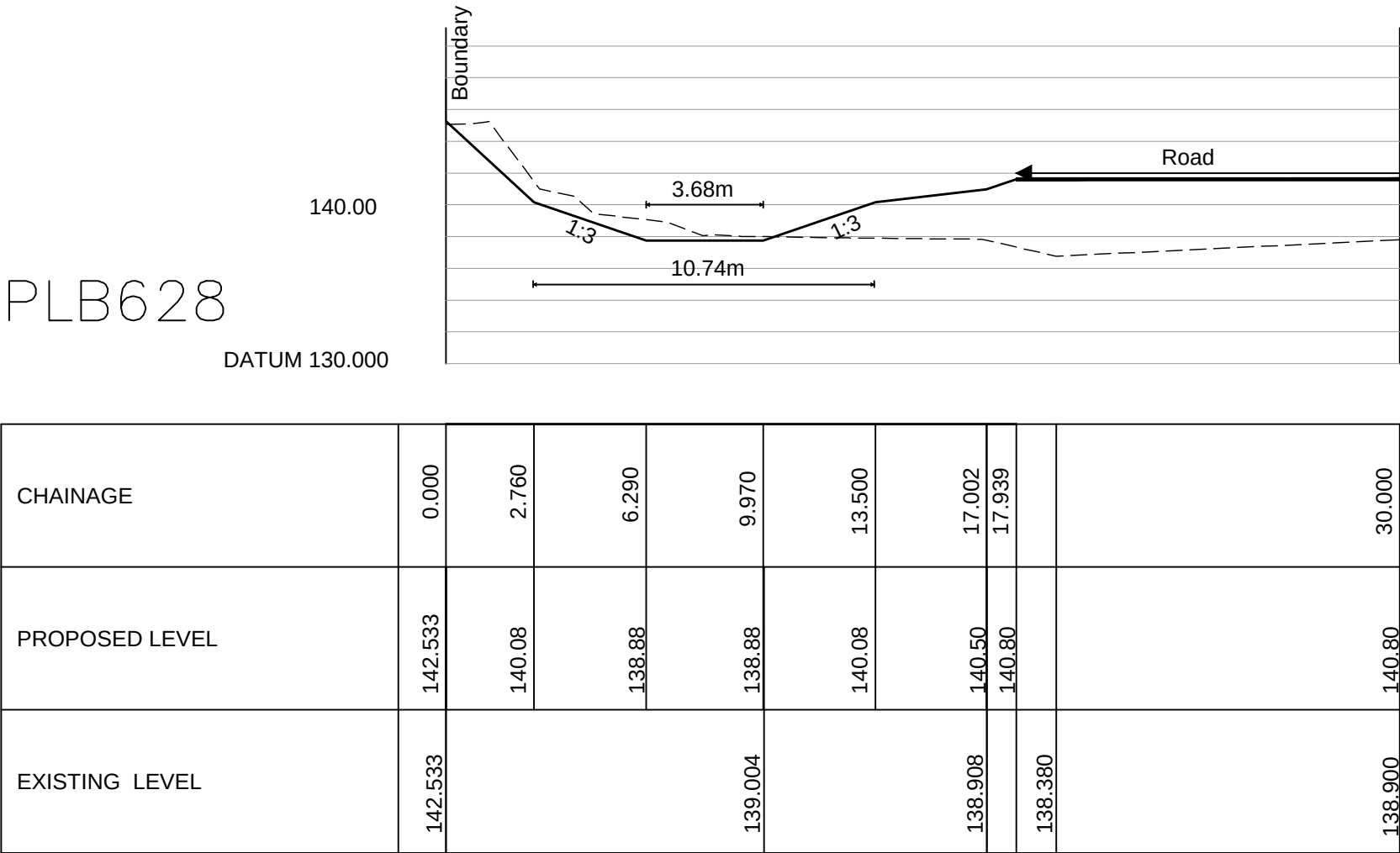
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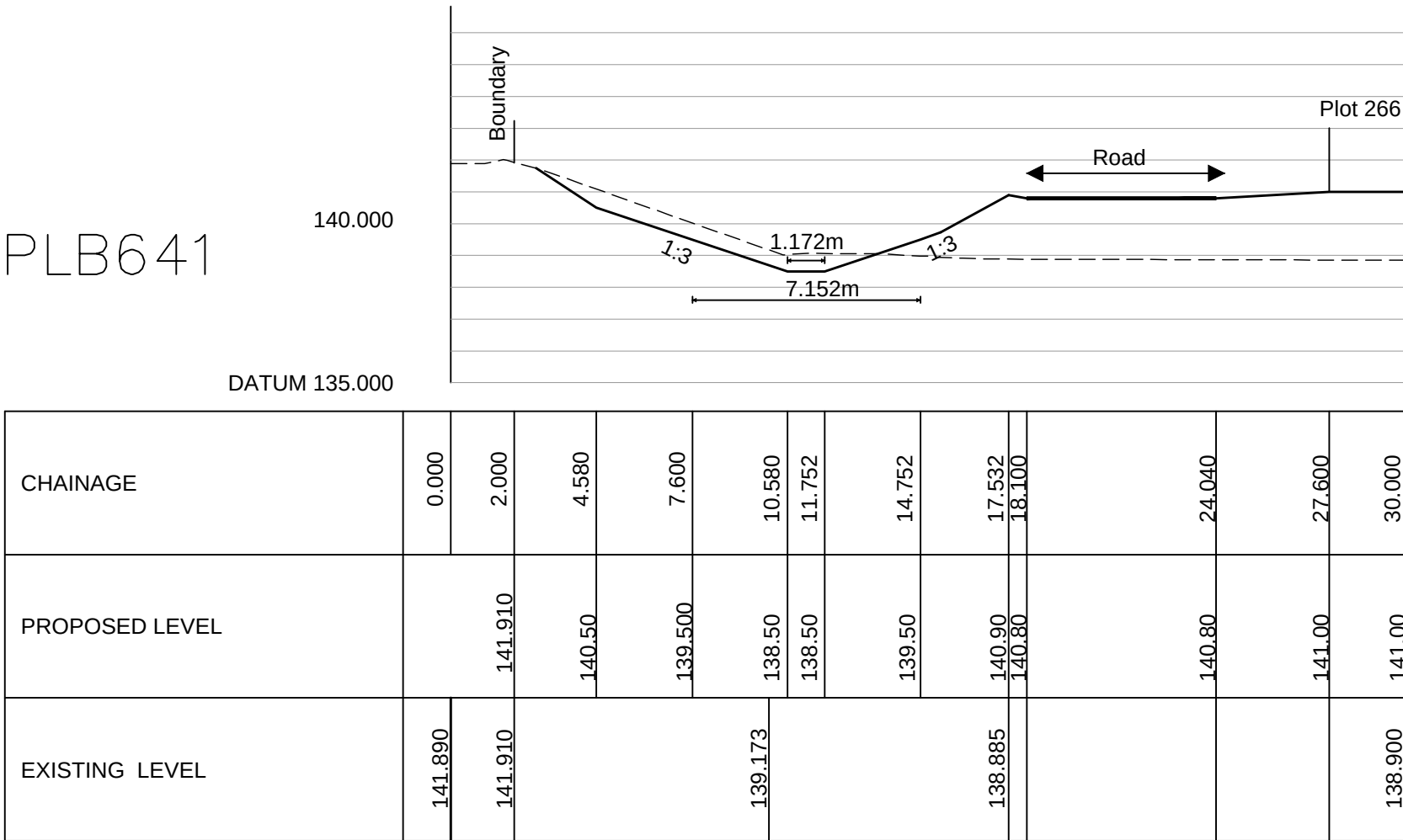
PLB588



PLB628



PLB641



Notes

1. All levels in meters above ordnance datum.
2. All sections are viewed downstream.
3. Refer to Drawing 6155/502 for section locations

Key

———— Proposed ground profile
----- Existing ground profile

A	SS	30.09.19	Existing ground profile added
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Client
Barratt Homes (Manchester)

Job Title
**Proposed Development
Tetrosyl Site
Walmersley Old Road
Bury**

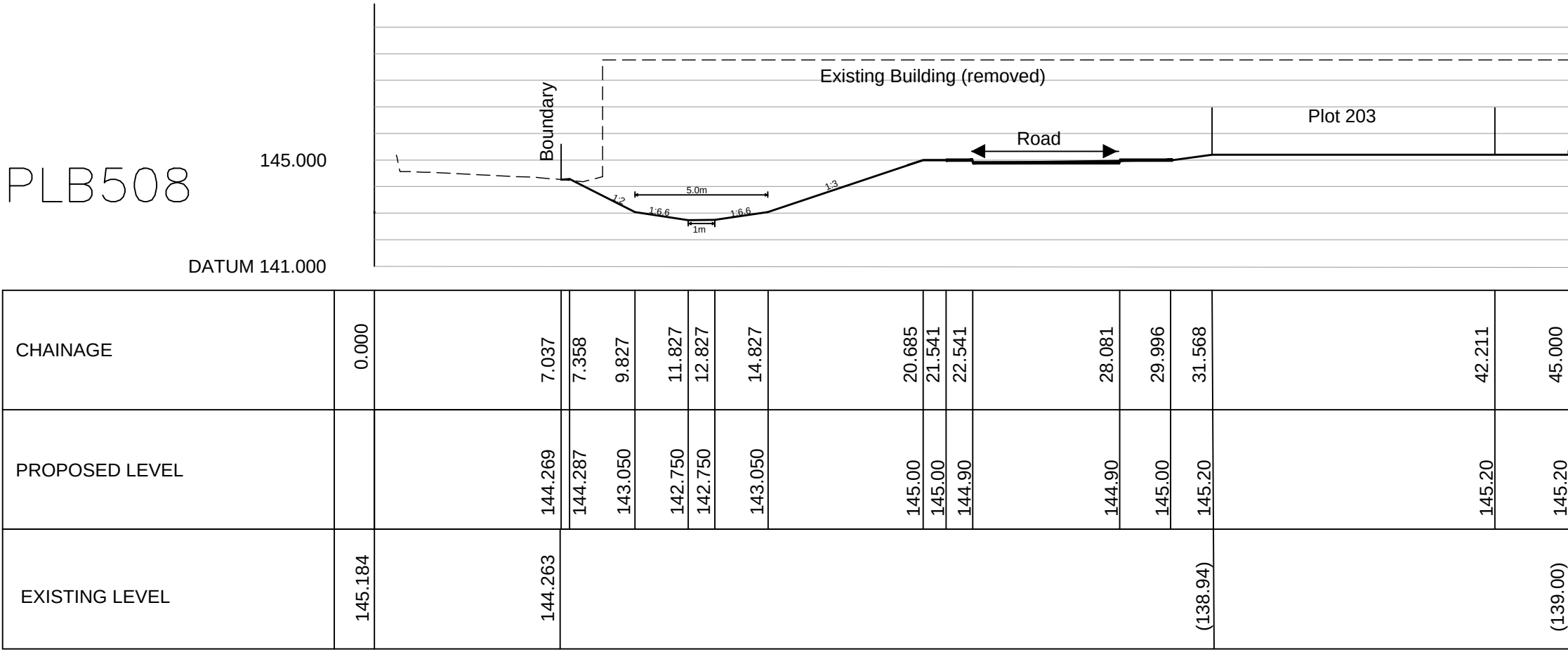
Drawing Title
**Deculverted Watercourse
Proposed Cross-sections
Page 2 of 2**

Drawing Status:					
FOR COMMENT	FOR PLANNING	FOR TENDER	FOR APPROVAL	FOR CONSTRUCTION	AS BUILT
CONSTRUCTION AT CLIENT / CONTRACTOR RISK					

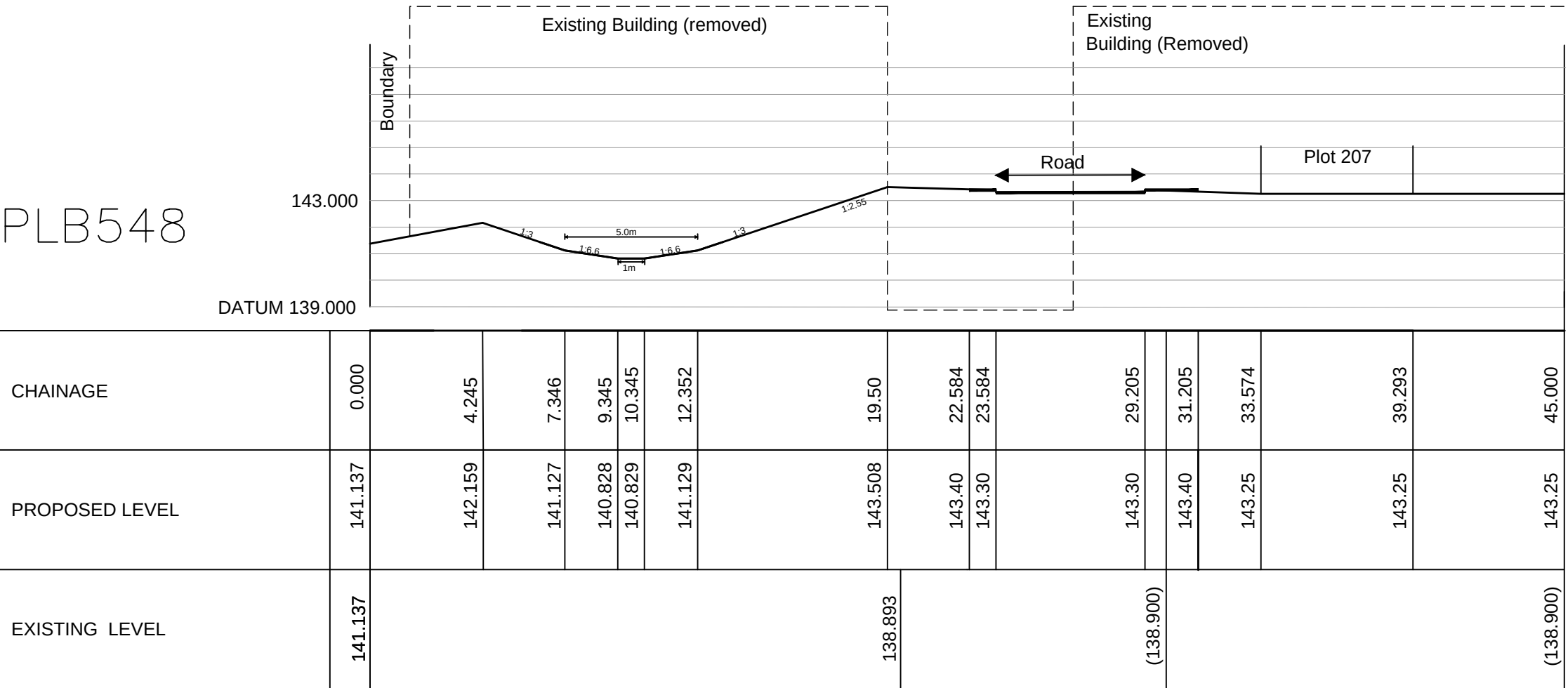
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Date: April 2019	Scale: 1:200@A1	

Orig. No. 6155/507/02	Rev. A
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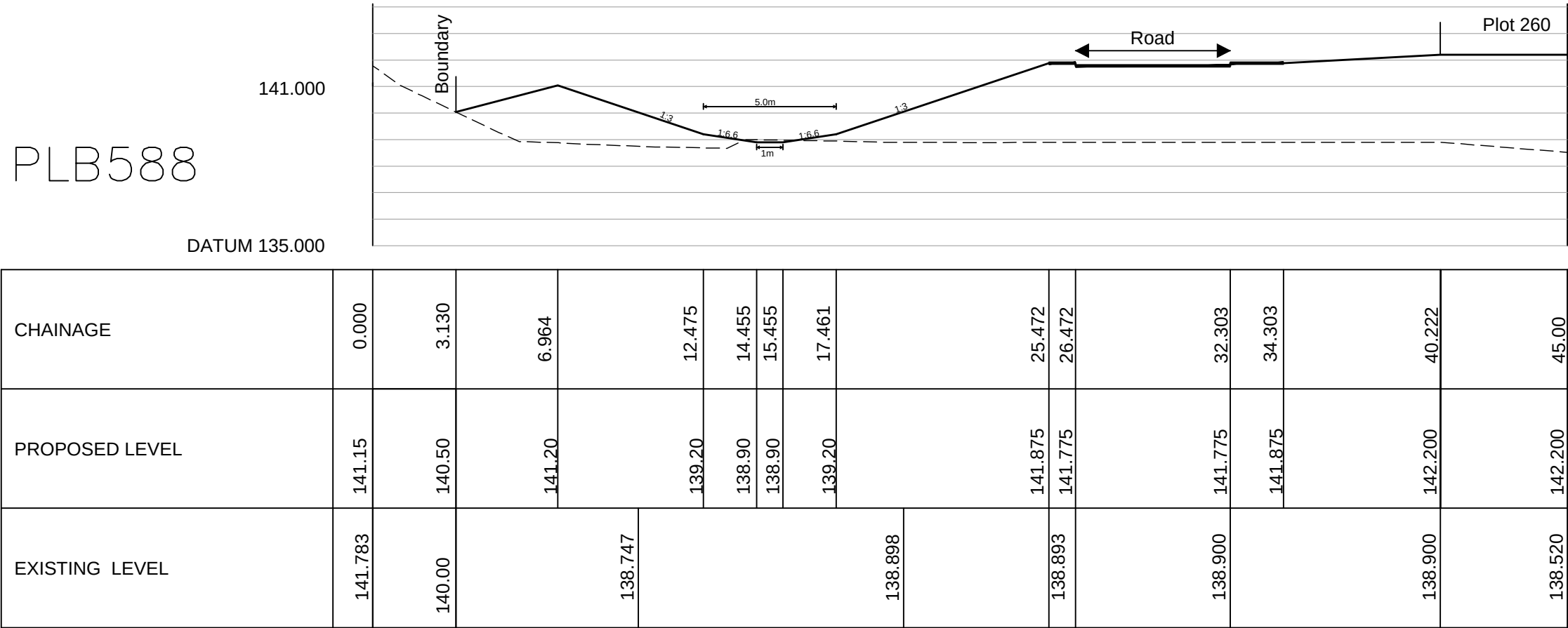
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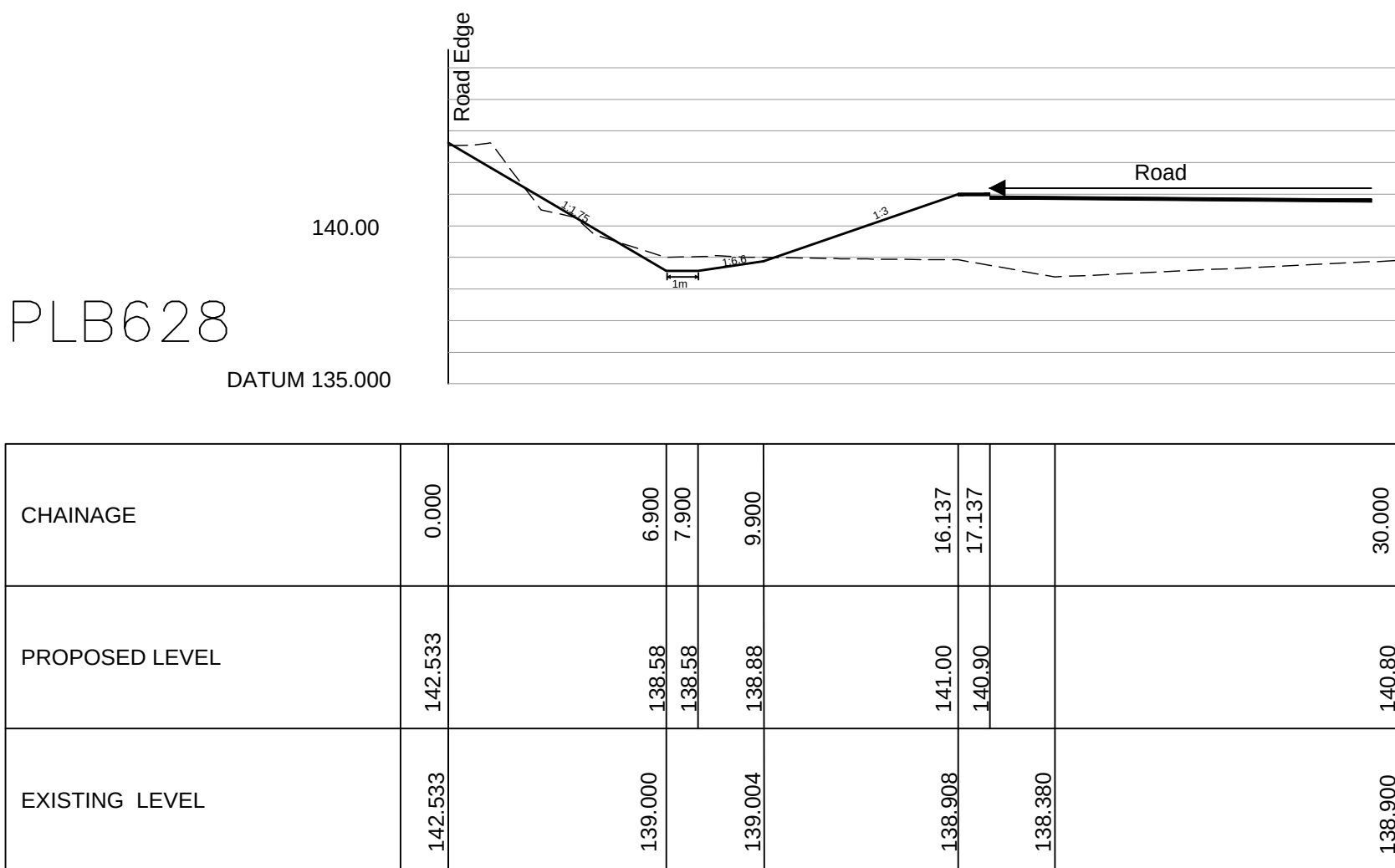
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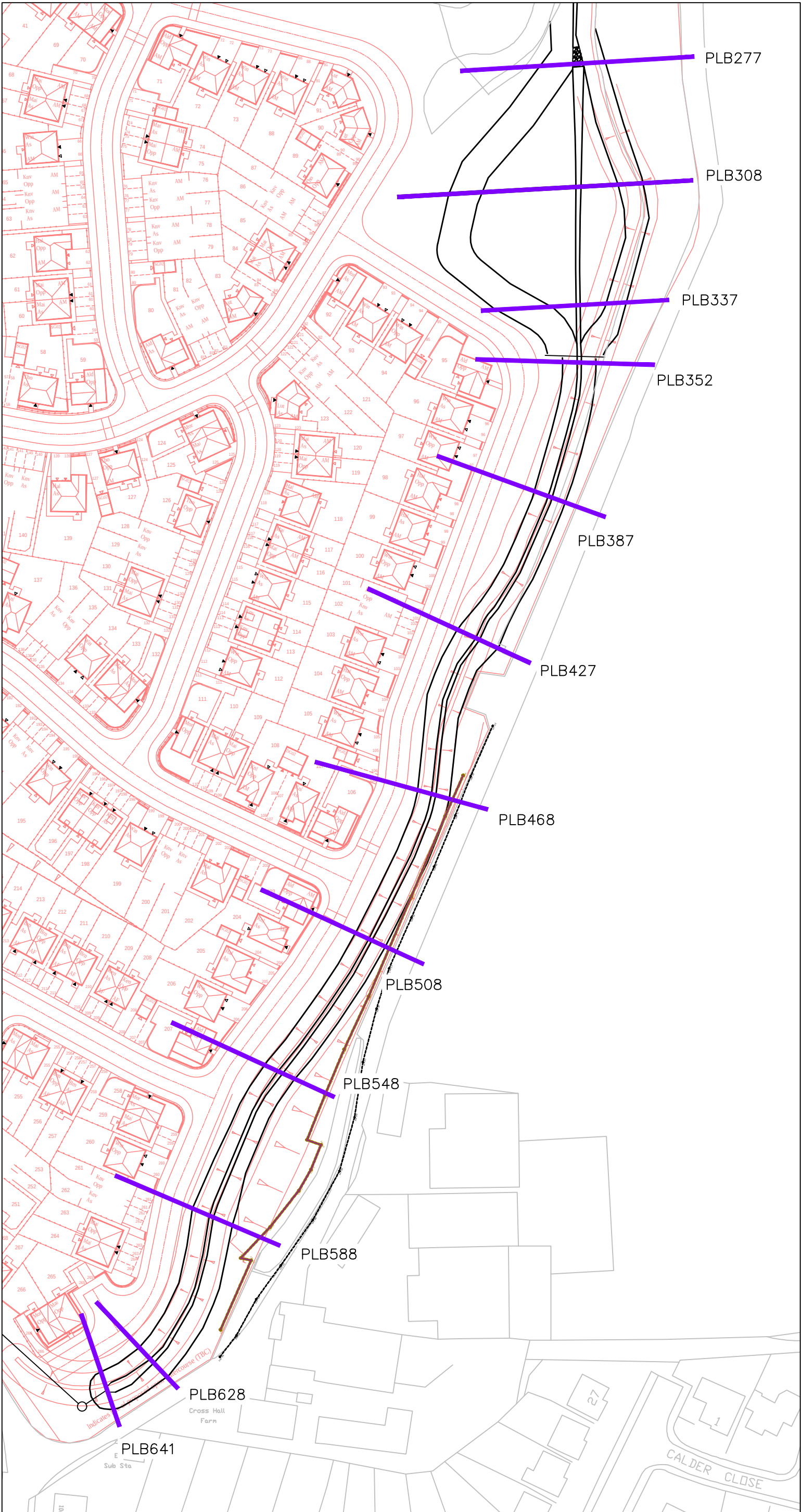
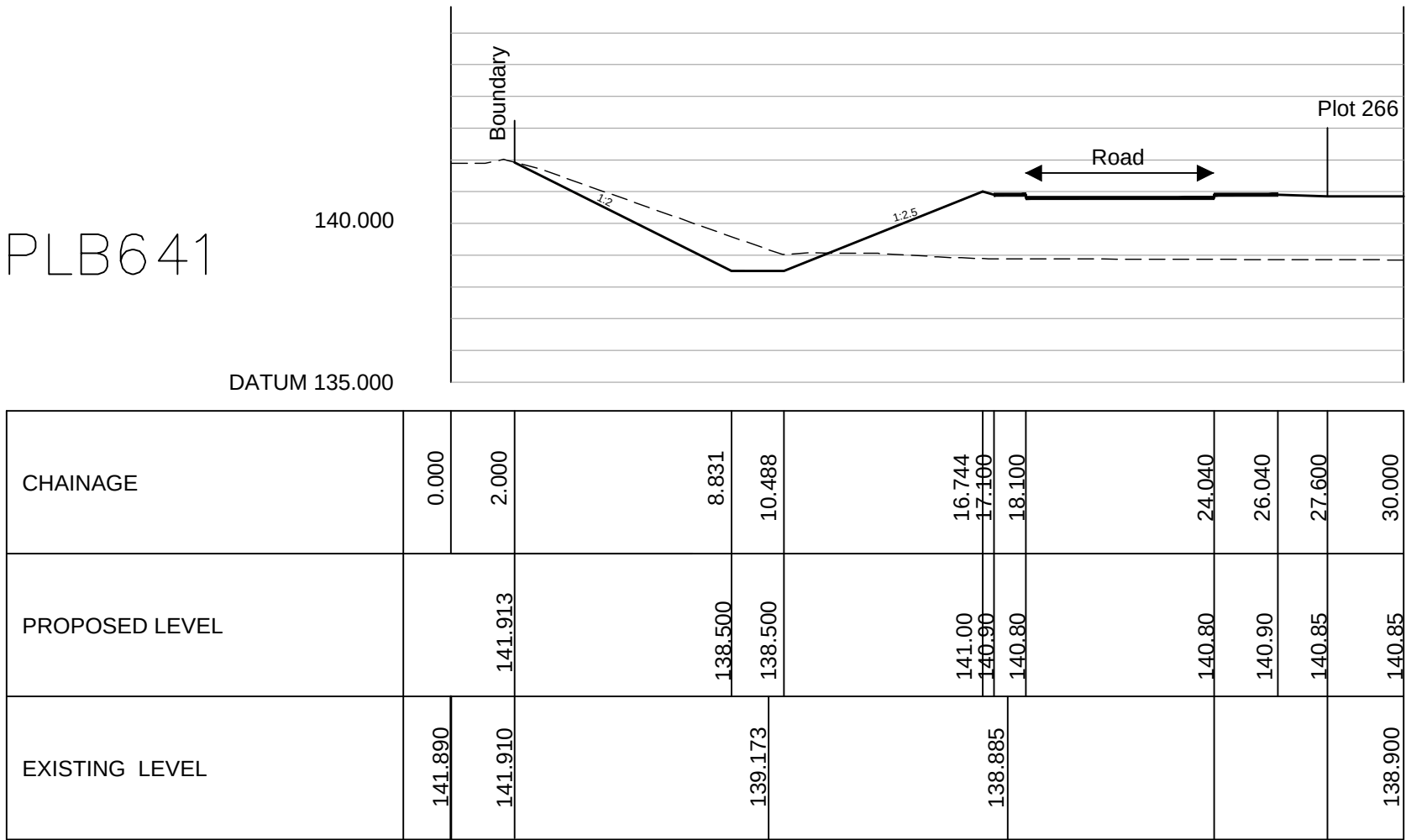
PLB588



PLB628



PLB641



Notes

1. All levels in meters above ordnance datum.
2. All sections are viewed downstream.
3. Refer to Drawing 6155/502 for section locations

Key

- Proposed ground profile
- - - Existing ground profile

Refer to sheet 2 for cross section location insert

D	NJ	02.02.21	PLB628 updated
C	NJ	15.12.20	Retaining Wall Added
B	NJ	09.10.20	Low flow channels added
A	SS	30.09.19	Existing ground profile added
No.	By	Date	Revision Details

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Client
Barratt Homes (Manchester)

Job Title
Proposed Development
Tetrosyl Site
Walmersley Old Road
Bury

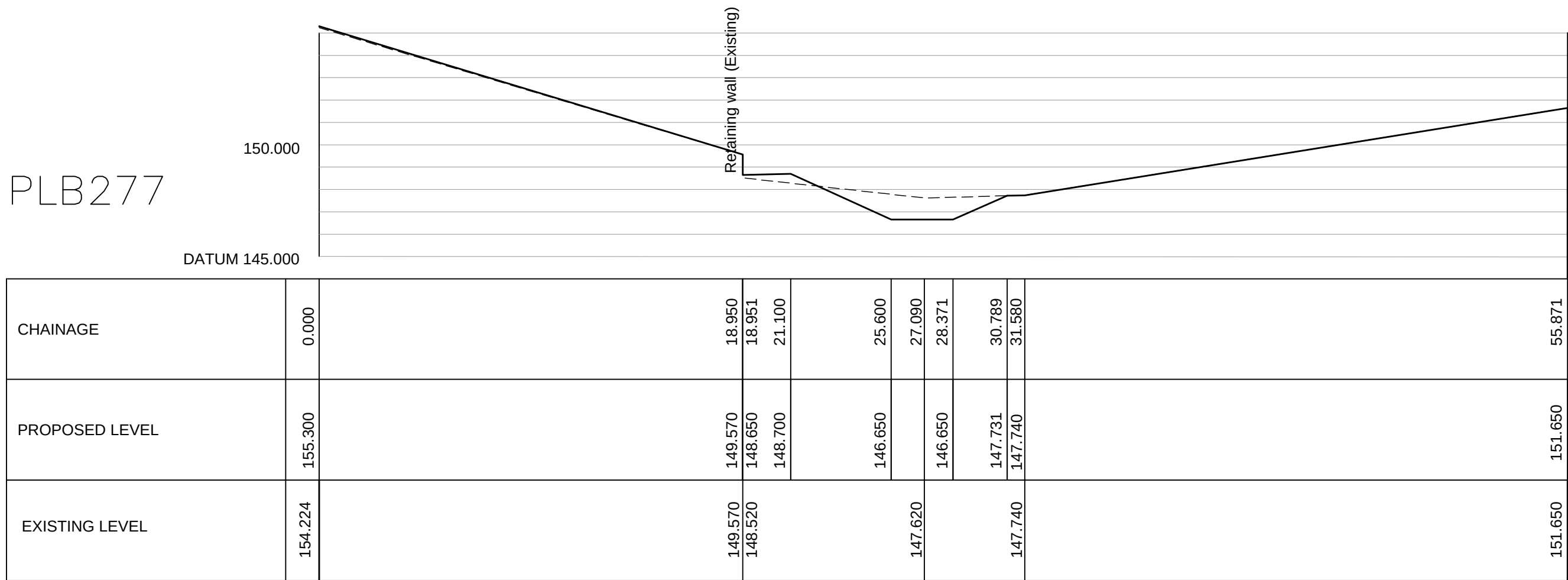
Drawing Title
Deculverted Watercourse
Proposed Cross-sections
Page 2 of 2

Drawing Status:
FOR COMMENT FOR PLANNING FOR TENDER FOR APPROVAL FOR CONSTRUCTION AS BUILT

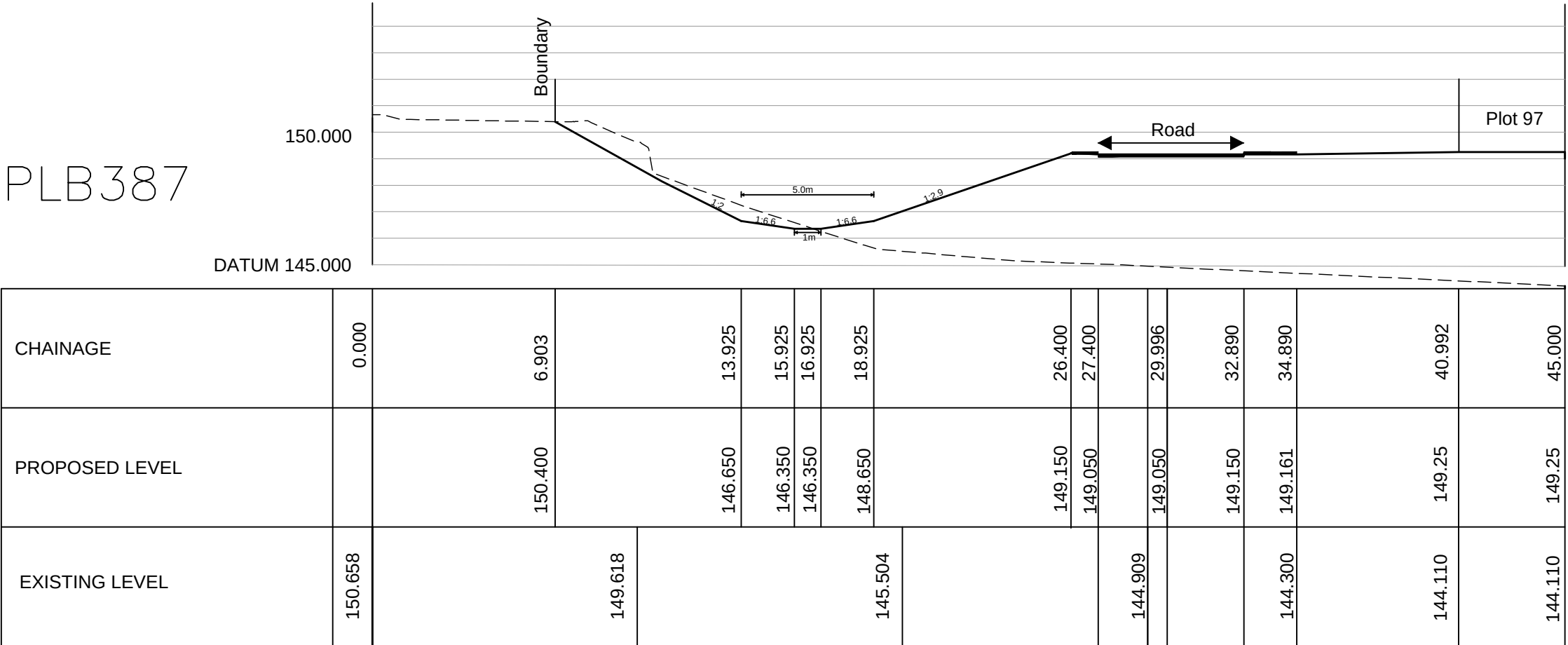
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Drawn by: ZB Scale: 1:200@A1
Checked by: DF

Dwg. No.: 6155/507/02
Rev.: D

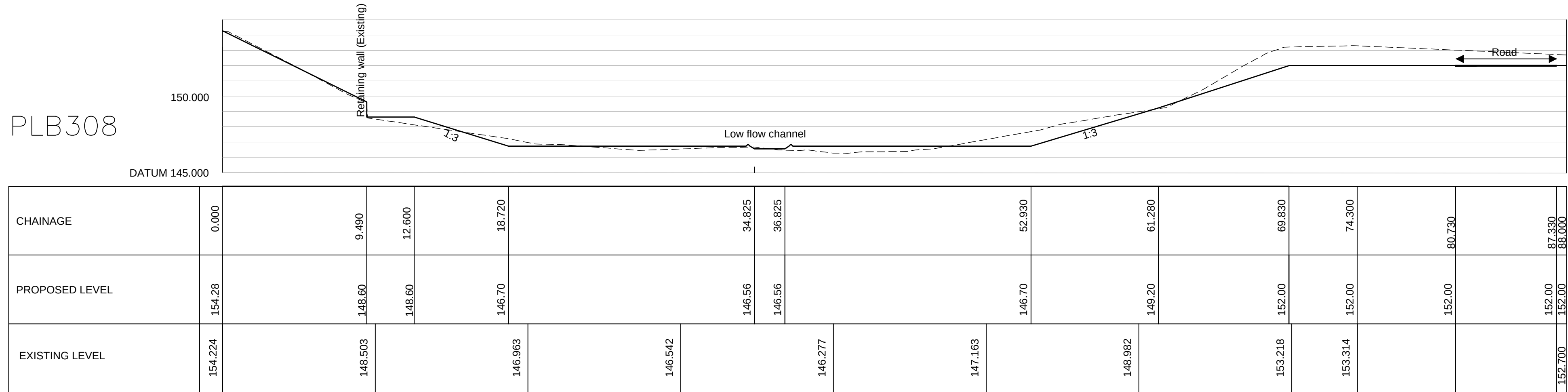
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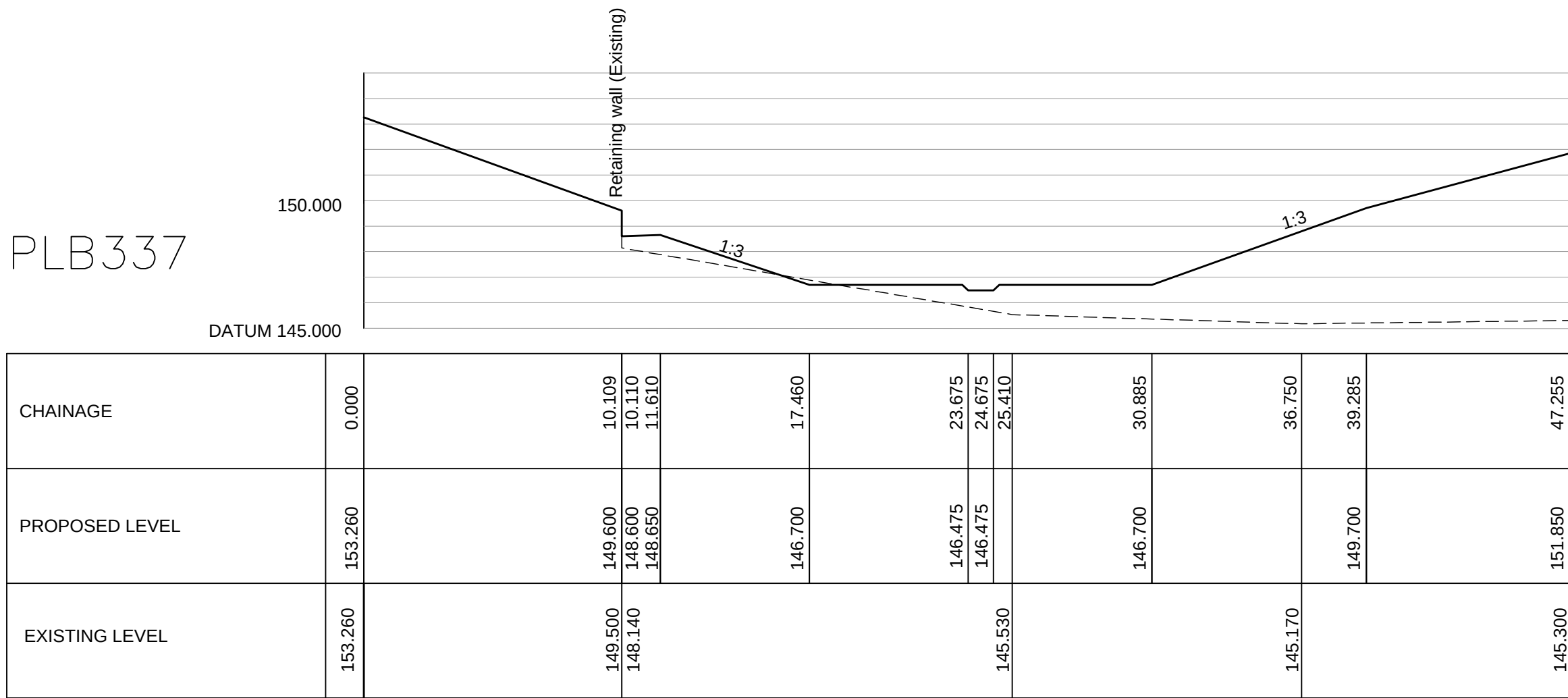
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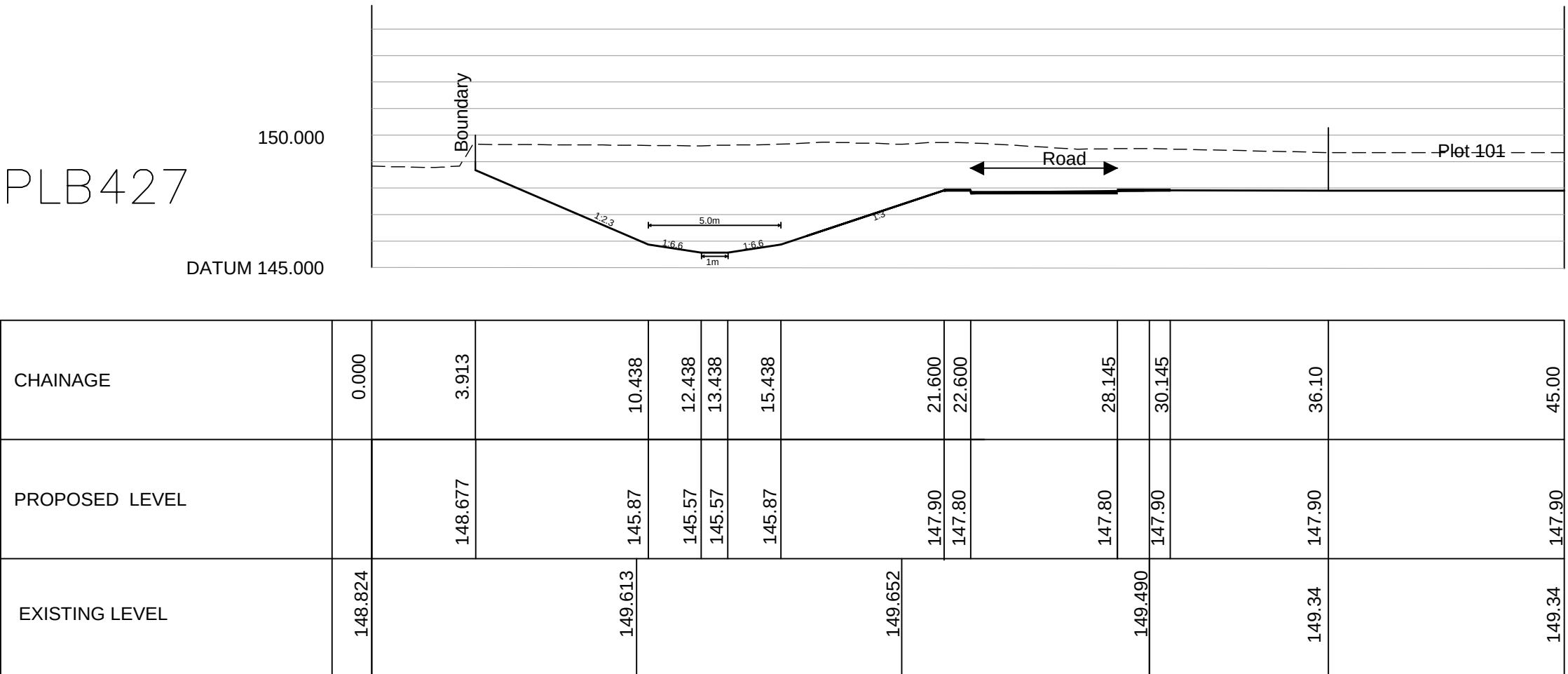
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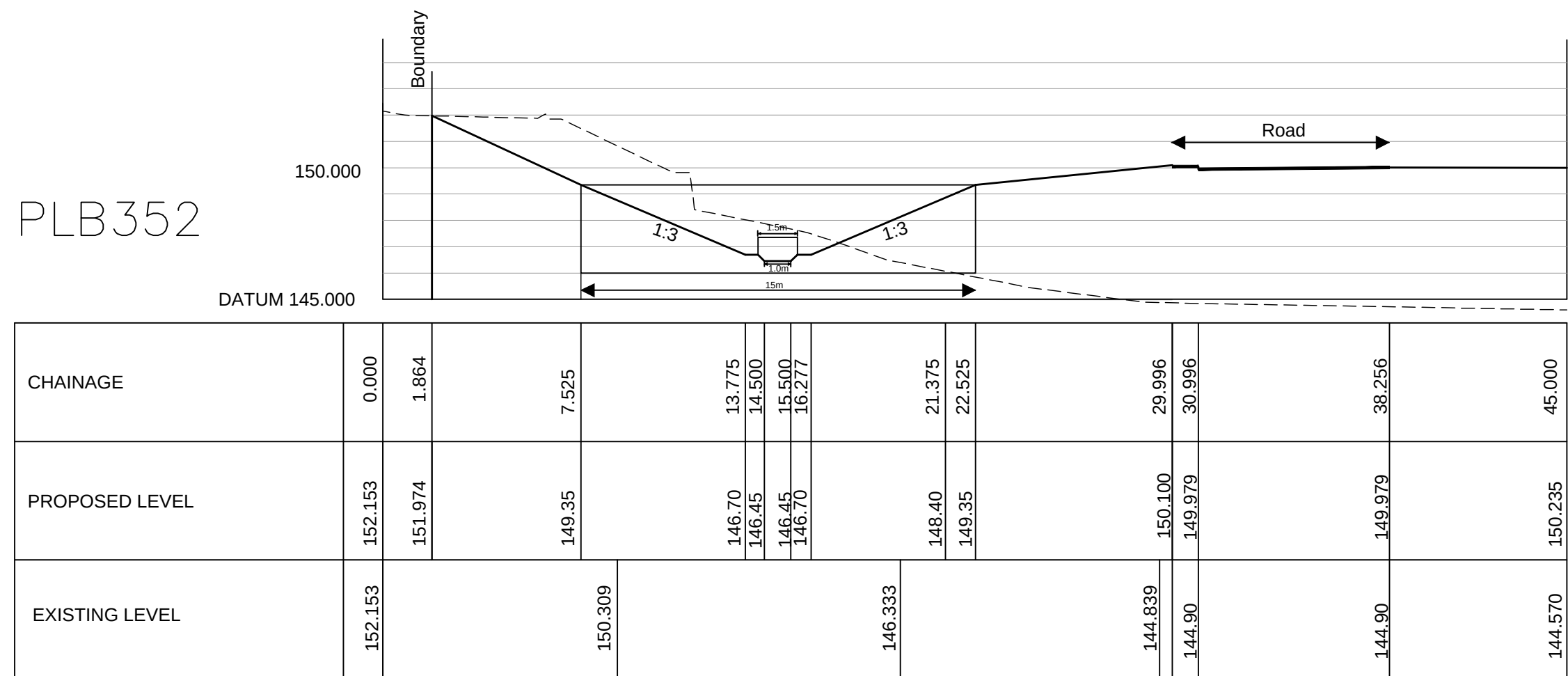
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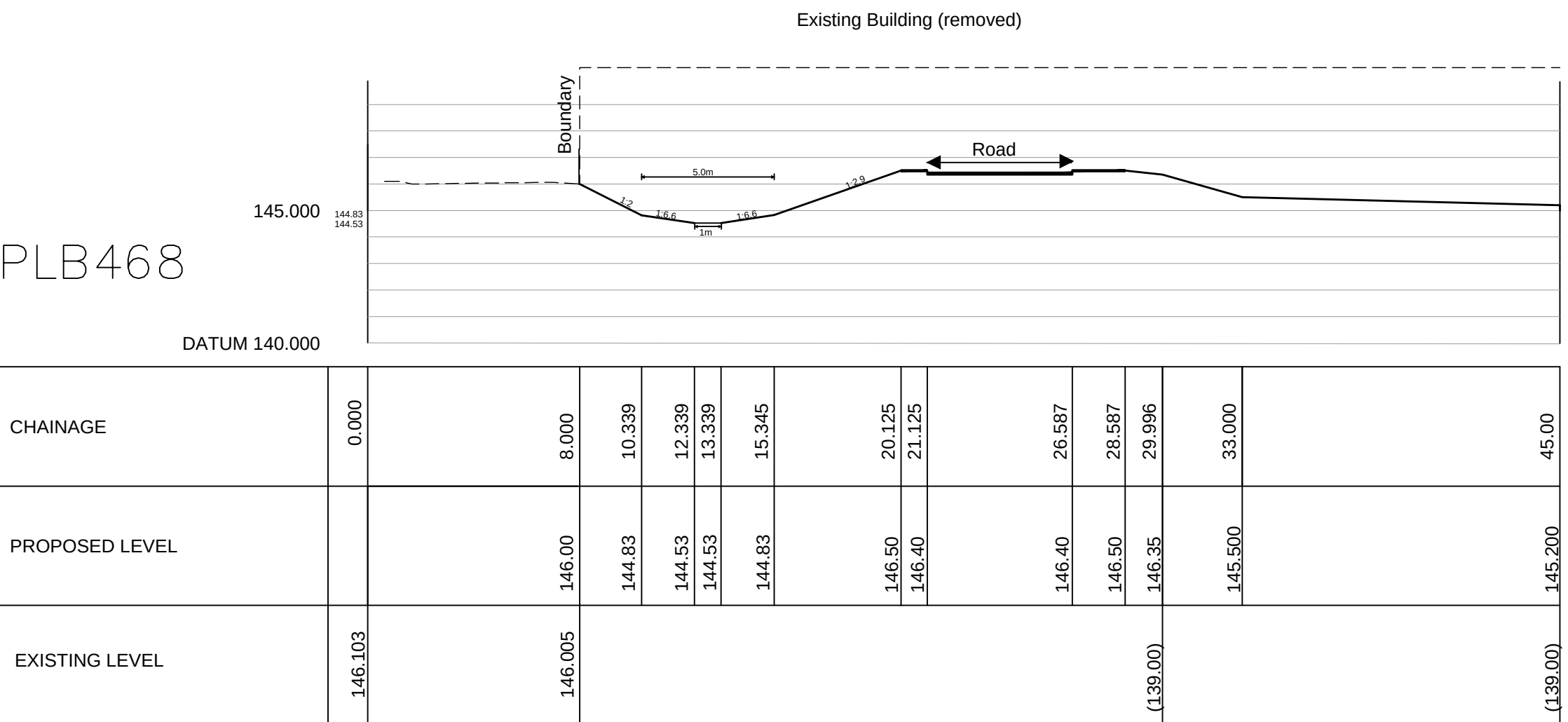
PLB427



PLB352



PLB468



Notes

1. All levels in meters above ordnance datum.
2. All sections are viewed downstream.
3. Refer to Drawing 6155/502 for section locations

Key

———— Proposed ground profile
----- Existing ground profile

Refer to sheet 2 for cross section location insert

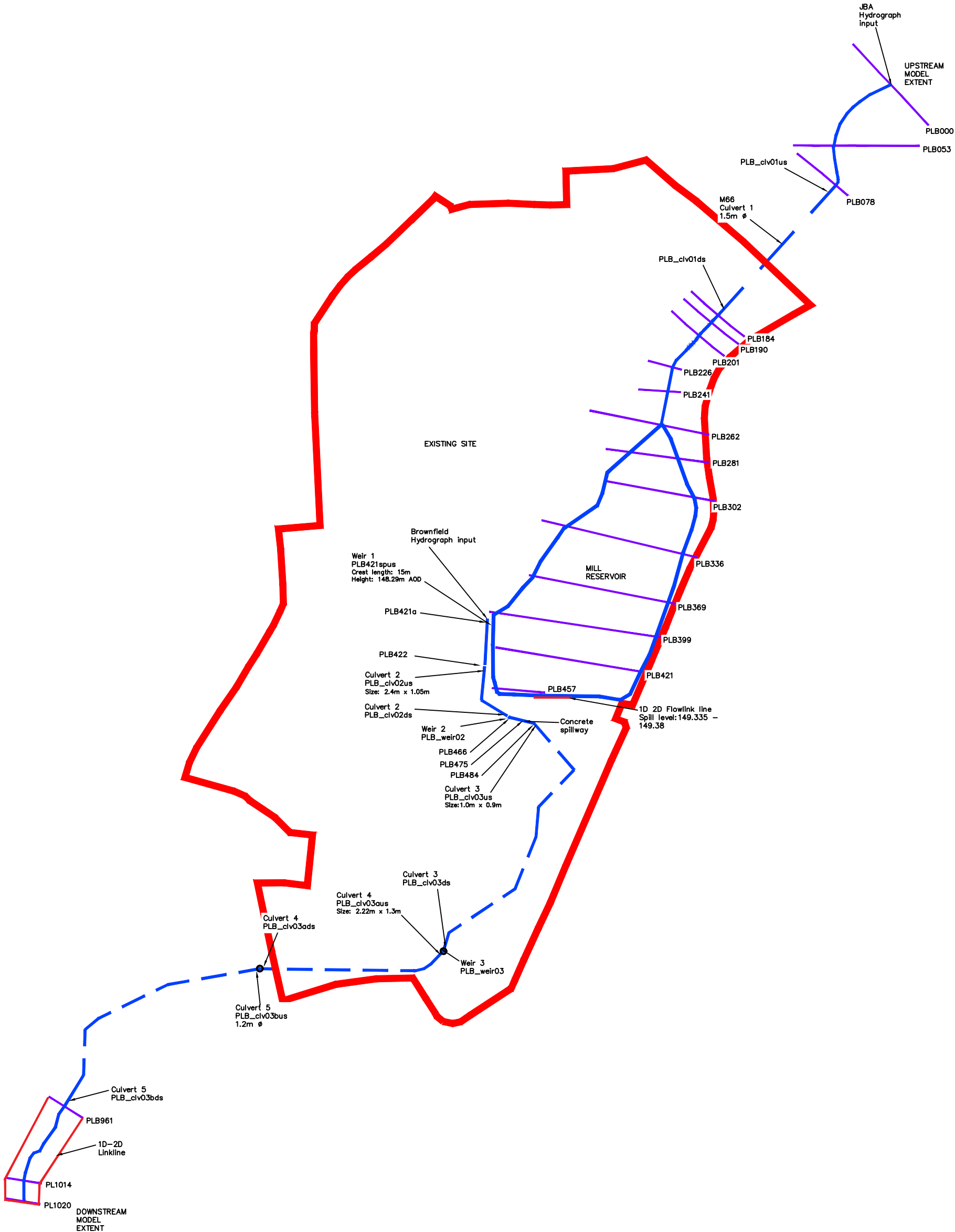
D	NJ	02.02.21	PLB628 updated
C	NJ	15.12.20	Retaining Wall Added
B	NJ	09.10.20	Low flow channels added
A	SS	30.09.19	Existing ground profile added

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Client
Barratt Homes (Manchester)
Job Title
Proposed Development Tetrosyl Site Walmersley Old Road Bury

Drawing Title
Deculverted Watercourse Proposed Cross-sections Page 1 of 2
Drawing Status
FOR COMMENT FOR TENDER FOR APPROVAL FOR CONSTRUCTION AS BUILT ***CONSTRUCTION AT CLIENT / CONTRACTOR RISK***

Designed by	Drawn by	Checked by
DF	ZB	DF
Date	Scale	
April 2019	1:200@A1	
Dwg. No.	Rev.	
6155/507/01	D	



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Job Title:

Proposed Development
Former Tetrosyl Site
Walmersley Old Road
Bury

Drawing Title:

Schematic Plan:
Existing Model

Drawing Status:

CONSTRUCTION AT CLIENT AND/OR CONTRACTOR RISK	FOR COMMENT	
	FOR PLANNING	X
	FOR TENDER	
	FOR APPROVAL	
	FOR CONSTRUCTION	
	AS BUILT	

No.	By	Date	Revision Details
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Client:

Barratt Homes (Manchester)

Drawn By

NJ

Date Drawn

August 2019

Scale

NTS @ A3

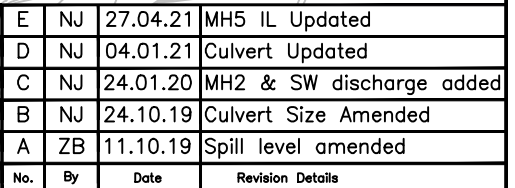
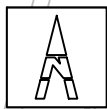
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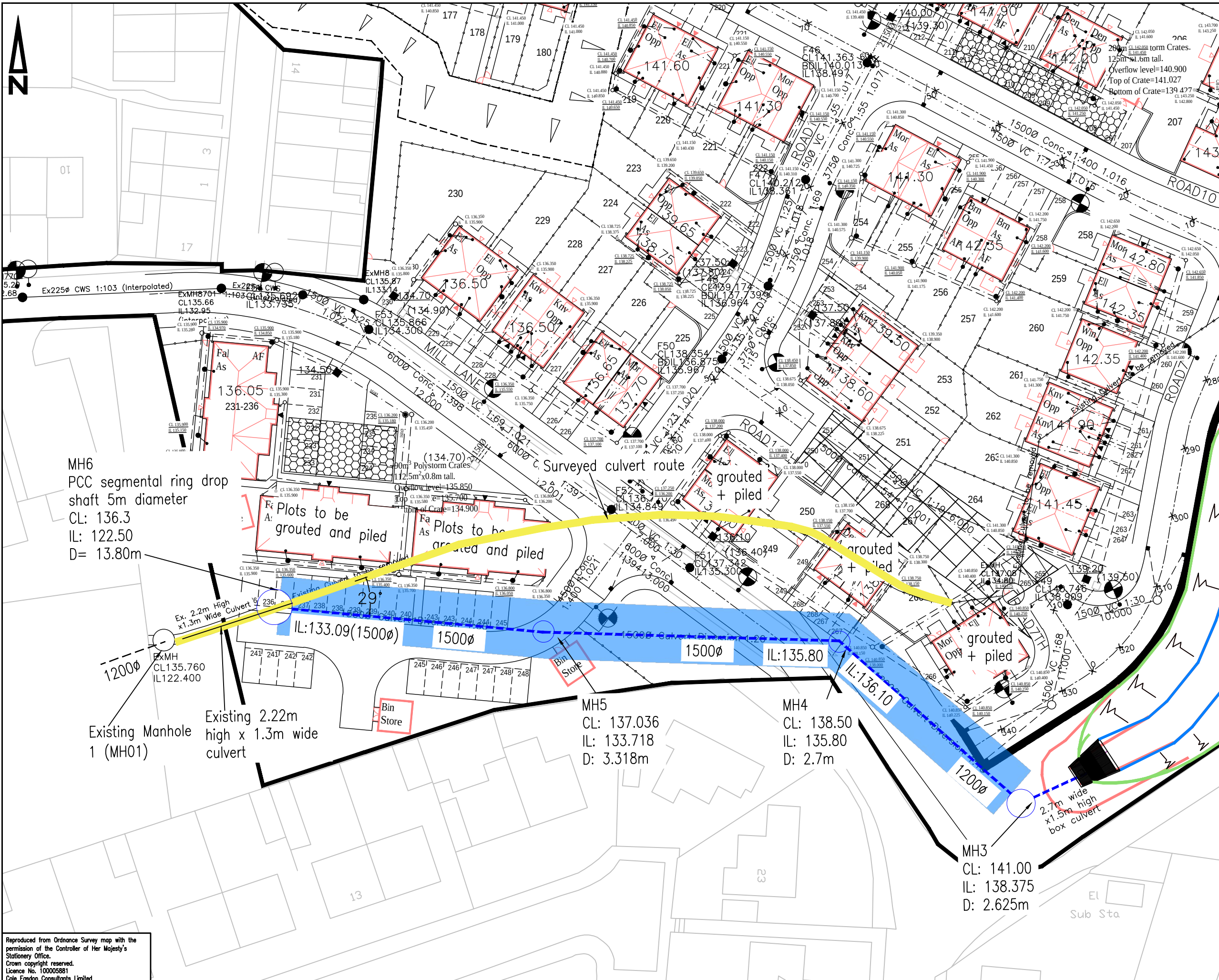
DF

Drawing No.

6155/511

Revision





Key

- Existing surveyed culvert
- Proposed culvert and 4m easement from centre line of culvert
- Proposed Manhole
- Existing Culvert
- Existing Level

Notes:

- All levels are in metres above ordnance datum
- All dimensions are in millimetres unless stated otherwise
- Sewer records provided by United Utilities, February 2014
- Topographical survey undertaken by Survey & Design Limited, March 2015.
- Proposed layout provided by Barratt Homes Manchester. Drawing Ref 456/ED/02 Rev K

B	NJ	27.04.21	MH5 IL updated
A	NJ	04.01.21	Drawing Key Updated

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Client
Barratt Homes (Manchester)

Job Title
**Proposed Residential Development
Former Tetrosyl Site
Walmersley Old Road
Bury**

Drawing Title
Proposed Culvert Diversion Route

Drawing Status				
FOR COMMENT	FOR PLANNING	FOR TENDER	FOR APPROVAL	FOR CONSTRUCTION
AS BUILT				
CONSTRUCTION AT CLIENT / CONTRACTOR RISK				
Designed by	NJ	Drawn by	NJ	Checked by
Date	September 2020	Scale	1:500 @ A3	DF
Orig. No.	6155/514	Rev.	B	

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Appendix 6

Cole Easdon Consultants		Page 1
160 Aztec Aztec West Bristol BS32 4TU	6155 Existing Brownfield Runoff 100 year + 40%	
Date 21/08/2019 12:55 File 6155 - EXISTING	Designed by njackson Checked by DF	
XP Solutions	Network 2016.1	

STORM SEWER DESIGN by the Modified Rational Method

Design Criteria for Storm

Pipe Sizes STANDARD Manhole Sizes STANDARD

FSR Rainfall Model - England and Wales

Return Period (years)	100	Add Flow / Climate Change (%)	0
M5-60 (mm)	19.000	Minimum Backdrop Height (m)	0.200
Ratio R	0.262	Maximum Backdrop Height (m)	1.500
Maximum Rainfall (mm/hr)	50	Min Design Depth for Optimisation (m)	1.200
Maximum Time of Concentration (mins)	30	Min Vel for Auto Design only (m/s)	1.00
Foul Sewage (l/s/ha)	0.000	Min Slope for Optimisation (1:X)	500
Volumetric Runoff Coeff.	0.750		

Designed with Level Soffits

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
1.000	140.000	7.000	20.0	4.510	1.00	0.0	0.600	o	825	Pipe/Conduit	
1.001	140.000	7.000	20.0	0.000	0.00	0.0	0.600	o	900	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
1.000	50.00	1.35	148.600	4.510	0.0	0.0	0.0	6.66	3557.8	610.7
1.001	50.00	1.68	142.000	4.510	0.0	0.0	0.0	7.02	4467.9	610.7

Cole Easdon Consultants		Page 2
160 Aztec Aztec West Bristol BS32 4TU	6155 Existing Brownfield Runoff 100 year + 40%	
Date 21/08/2019 12:55 File 6155 - EXISTING	Designed by njackson Checked by DF	
XP Solutions		Network 2016.1

Manhole Schedules for Storm

MH Name	MH CL (m)	MH Depth (m)	MH Connection	MH Diam., L*W (mm)	PN	Pipe Out Invert Level (m)	Diameter (mm)	PN	Pipes In Invert Level (m)	Diameter (mm)	Backdrop (mm)
S1	149.600	1.000	Junction	0	1.000	148.600	825				
S2	143.000	1.400	Open Manhole	1800	1.001	142.000	900	1.000	141.600	825	
	140.000	5.000	Open Manhole	0		OUTFALL		1.001	135.000	900	

Free Flowing Outfall Details for Storm

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D, L (mm)	W (mm)
---------------------	--------------	--------------	--------------	------------------	-----------	--------

1.001		140.000	135.000	0.000	0	0
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Cole Easdon Consultants		Page 3
160 Aztec Aztec West Bristol BS32 4TU	6155 Existing Brownfield Runoff 100 year + 40%	
Date 21/08/2019 12:55 File 6155 - EXISTING	Designed by njackson Checked by DF	
XP Solutions	Network 2016.1	

Summary of Critical Results by Maximum Level (Rank 1) for Storm

Simulation Criteria

Areal Reduction Factor 1.000 Additional Flow - % of Total Flow 0.000
 Hot Start (mins) 0 MADD Factor * 10m³/ha Storage 2.000
 Hot Start Level (mm) 0 Inlet Coefficient 0.800
 Manhole Headloss Coeff (Global) 0.500 Flow per Person per Day (l/per/day) 0.000
 Foul Sewage per hectare (l/s) 0.000

Number of Input Hydrographs 0 Number of Storage Structures 0
 Number of Online Controls 0 Number of Time/Area Diagrams 0
 Number of Offline Controls 0 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR Ratio R 0.267
 Return Period (years) 100 Cv (Summer) 0.750
 Region England and Wales Cv (Winter) 0.840
 M5-60 (mm) 19.000

Margin for Flood Risk Warning (mm) 300.0 DVD Status OFF
 Analysis Timestep Fine Inertia Status OFF
 DTS Status ON

Profile(s) Summer and Winter
 Duration(s) (mins) 600
 Sensitivity flows(s) (%) 0, +40

PN	US/MH Name	Storm	Climate Change	First (X) Surge	First (Y) Flood	First (Z) Overflow	Overflow Act.	Water Level (m)	Surcharged Depth (m)
1.000	S1	600 Summer	+40%					148.784	-0.641
1.001	S2	600 Summer	+40%					142.188	-0.712

PN	US/MH Name	Flooded Volume (m ³)	Flow / Cap. (l/s)	Pipe Flow (l/s)	Level Exceeded
1.000	S1	0.000	0.11	405.6	OK*
1.001	S2	0.000	0.10	405.6	OK

Cole Easdon Consultants				Page 1	
160 Aztec Aztec West Bristol BS32 4TU		6155 Tetrosyl Storage Calcs 100yr + 40% CC			
Date 29/01/2020 16:38 File 6155 STORAGE CALCS.SRCX		Designed by njackson Checked by			
XP Solutions		Source Control 2016.1			
<u>Summary of Results for 100 year Return Period (+40%)</u>					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	10.285	0.485	90.4	666.8	O K
30 min Summer	10.463	0.663	92.4	911.1	O K
60 min Summer	10.644	0.844	92.5	1160.7	O K
120 min Summer	10.786	0.986	92.5	1356.4	O K
180 min Summer	10.834	1.034	92.5	1421.2	O K
240 min Summer	10.851	1.051	92.5	1444.5	O K
360 min Summer	10.850	1.050	92.5	1444.1	O K
480 min Summer	10.823	1.023	92.5	1407.2	O K
600 min Summer	10.785	0.985	92.5	1353.9	O K
720 min Summer	10.740	0.940	92.5	1292.7	O K
960 min Summer	10.647	0.847	92.5	1164.8	O K
1440 min Summer	10.476	0.676	92.4	928.9	O K
2160 min Summer	10.291	0.491	90.5	675.5	O K
2880 min Summer	10.185	0.385	87.2	528.8	O K
4320 min Summer	10.108	0.308	71.6	423.6	O K
5760 min Summer	10.070	0.270	59.5	371.0	O K
7200 min Summer	10.045	0.245	51.3	336.5	O K
8640 min Summer	10.026	0.226	45.3	311.1	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
15 min Summer	112.612	0.0	698.0	18	
30 min Summer	79.525	0.0	993.1	32	
60 min Summer	53.779	0.0	1361.1	62	
120 min Summer	34.971	0.0	1772.9	110	
180 min Summer	26.617	0.0	2025.3	144	
240 min Summer	21.806	0.0	2213.1	176	
360 min Summer	16.454	0.0	2506.0	246	
480 min Summer	13.443	0.0	2730.5	314	
600 min Summer	11.478	0.0	2914.7	382	
720 min Summer	10.080	0.0	3071.7	448	
960 min Summer	8.199	0.0	3331.5	578	
1440 min Summer	6.111	0.0	3722.0	824	
2160 min Summer	4.538	0.0	4158.5	1172	
2880 min Summer	3.670	0.0	4483.0	1500	
4320 min Summer	2.727	0.0	4991.1	2208	
5760 min Summer	2.211	0.0	5408.4	2936	
7200 min Summer	1.881	0.0	5749.2	3672	
8640 min Summer	1.649	0.0	6048.2	4408	
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Cole Easdon Consultants				Page 2	
160 Aztec Aztec West Bristol BS32 4TU		6155 Tetrosyl Storage Calcs 100yr + 40% CC			
Date 29/01/2020 16:38		Designed by njackson			
File 6155 STORAGE CALCS.SRCX		Checked by			
XP Solutions		Source Control 2016.1			
<u>Summary of Results for 100 year Return Period (+40%)</u>					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
10080 min Summer	10.012	0.212	40.7	291.8	O K
15 min Winter	10.346	0.546	91.5	750.8	O K
30 min Winter	10.549	0.749	92.5	1029.7	O K
60 min Winter	10.760	0.960	92.5	1319.9	O K
120 min Winter	10.934	1.134	92.5	1559.4	O K
180 min Winter	10.981	1.181	92.5	1623.2	O K
240 min Winter	10.991	1.191	92.5	1638.2	O K
360 min Winter	10.975	1.175	92.5	1615.5	O K
480 min Winter	10.923	1.123	92.5	1543.4	O K
600 min Winter	10.854	1.054	92.5	1449.1	O K
720 min Winter	10.778	0.978	92.5	1345.4	O K
960 min Winter	10.626	0.826	92.5	1136.1	O K
1440 min Winter	10.372	0.572	91.8	786.3	O K
2160 min Winter	10.165	0.365	86.3	501.3	O K
2880 min Winter	10.109	0.309	71.8	424.2	O K
4320 min Winter	10.053	0.253	54.1	348.1	O K
5760 min Winter	10.023	0.223	44.1	306.1	O K
7200 min Winter	10.002	0.202	37.5	278.4	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
10080 min Summer	1.478	0.0	6314.6	5136	
15 min Winter	112.612	0.0	783.9	18	
30 min Winter	79.525	0.0	1114.5	32	
60 min Winter	53.779	0.0	1525.6	60	
120 min Winter	34.971	0.0	1986.8	116	
180 min Winter	26.617	0.0	2269.5	168	
240 min Winter	21.806	0.0	2479.8	190	
360 min Winter	16.454	0.0	2807.8	268	
480 min Winter	13.443	0.0	3059.4	344	
600 min Winter	11.478	0.0	3265.6	416	
720 min Winter	10.080	0.0	3441.6	486	
960 min Winter	8.199	0.0	3732.9	618	
1440 min Winter	6.111	0.0	4170.9	854	
2160 min Winter	4.538	0.0	4658.4	1148	
2880 min Winter	3.670	0.0	5022.1	1500	
4320 min Winter	2.727	0.0	5592.6	2208	
5760 min Winter	2.211	0.0	6058.0	2944	
7200 min Winter	1.881	0.0	6440.0	3672	
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160 Aztec Aztec West Bristol BS32 4TU			6155 Tetrosyl Storage Calcs 100yr + 40% CC			
Date 29/01/2020 16:38 File 6155 STORAGE CALCS.SRCX			Designed by njackson Checked by			
XP Solutions			Source Control 2016.1			

Micro Drainage

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160 Aztec Aztec West Bristol BS32 4TU	6155 Tetrosyl Storage Calcs 100yr + 40% CC	
Date 29/01/2020 16:38 File 6155 STORAGE CALCS.SRCX	Designed by njackson Checked by	
XP Solutions Source Control 2016.1		

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)	19.000	Shortest Storm (mins)	15
Ratio R	0.262	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 3.400

Time (mins)	Area
From: To:	(ha)
0 4	3.400

Time Area Diagram

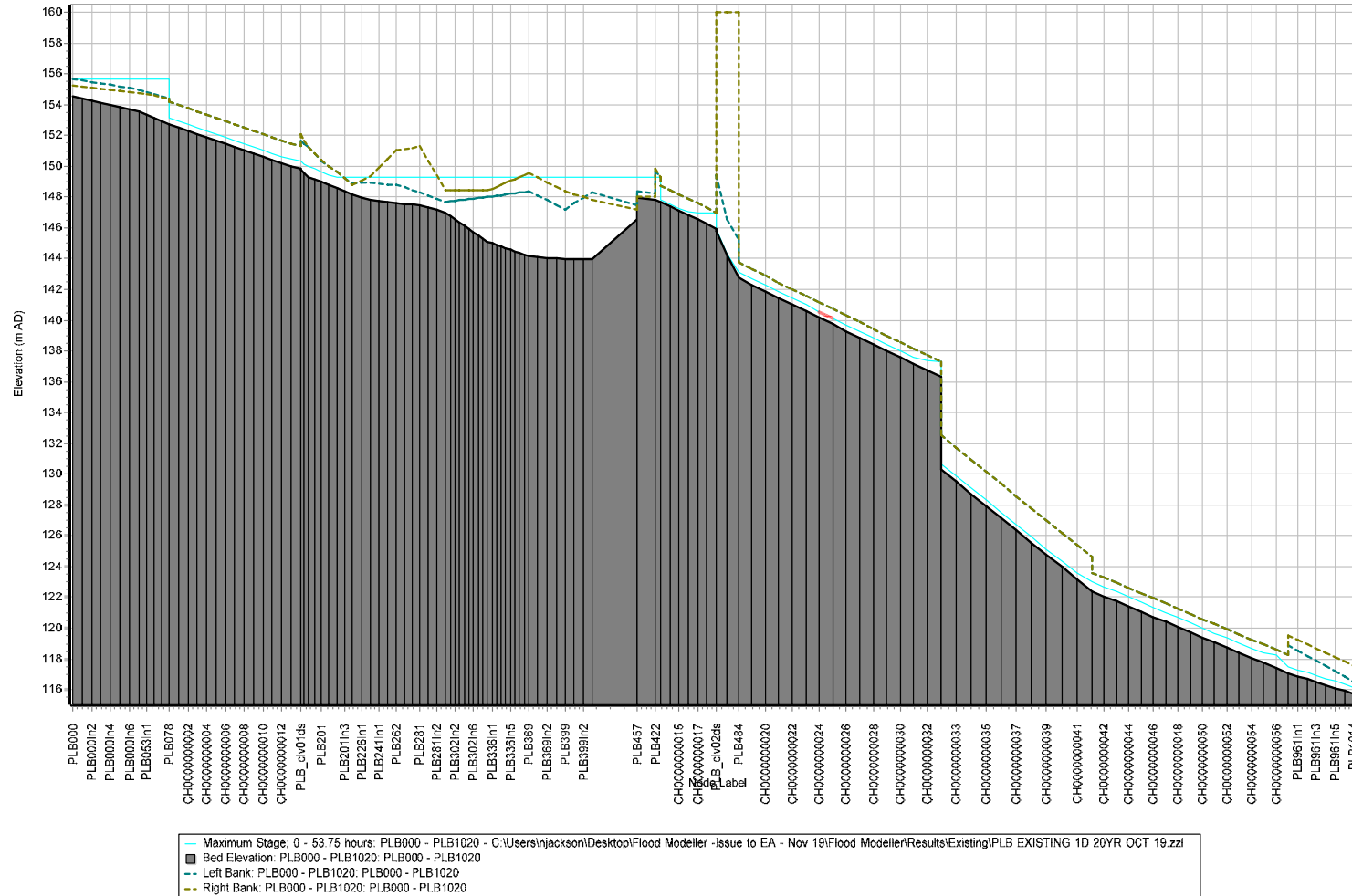
Total Area (ha) 0.000

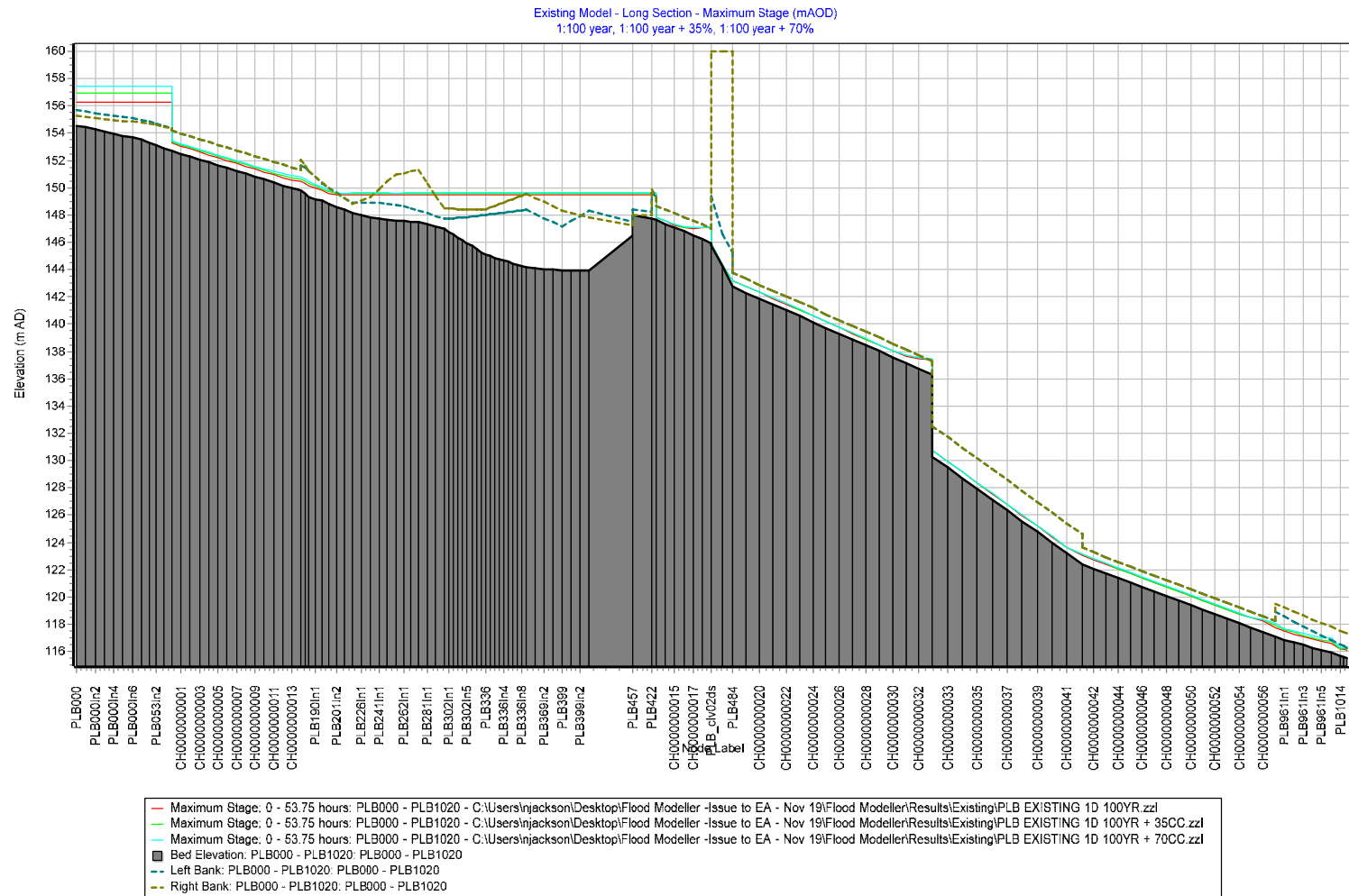
Time (mins)	Area
From: To:	(ha)
0 4	0.000

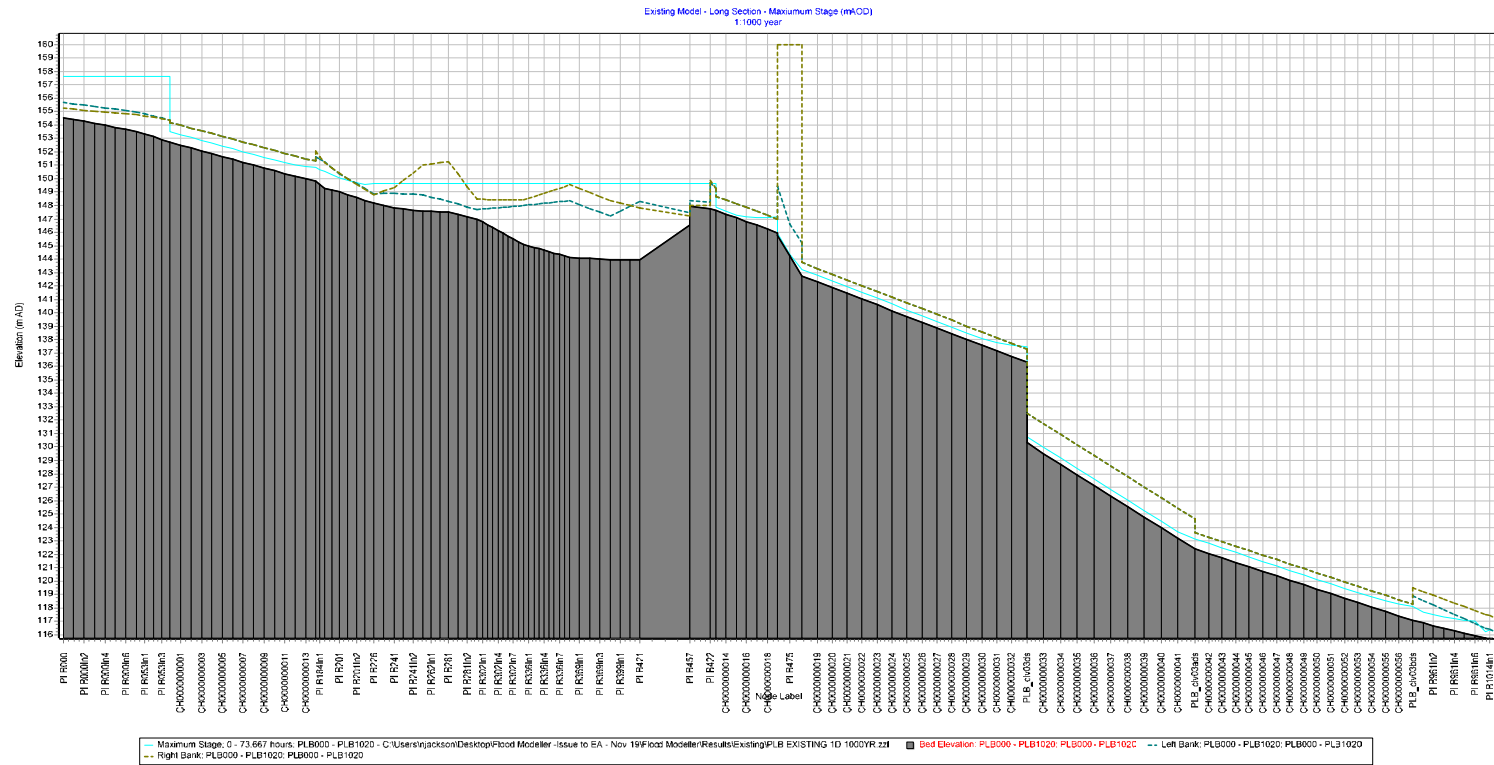
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<u>Model Details</u> Storage is Online Cover Level (m) 12.000 <u>Tank or Pond Structure</u> Invert Level (m) 9.800 <table><tr><th>Depth (m)</th><th>Area (m²)</th><th>Depth (m)</th><th>Area (m²)</th><th>Depth (m)</th><th>Area (m²)</th></tr><tr><td>0.000</td><td>1375.0</td><td>1.200</td><td>1375.0</td><td>1.210</td><td>0.0</td></tr></table> <u>Hydro-Brake Optimum® Outflow Control</u> <table><tr><td>Unit Reference</td><td>MD-SHE-0368-9300-2200-9300</td></tr><tr><td>Design Head (m)</td><td>2.200</td></tr><tr><td>Design Flow (l/s)</td><td>93.0</td></tr><tr><td>Flush-Flo™</td><td>Calculated</td></tr><tr><td>Objective</td><td>Minimise upstream storage</td></tr><tr><td>Application</td><td>Surface</td></tr><tr><td>Sump Available</td><td>Yes</td></tr><tr><td>Diameter (mm)</td><td>368</td></tr><tr><td>Invert Level (m)</td><td>9.800</td></tr><tr><td>Minimum Outlet Pipe Diameter (mm)</td><td>450</td></tr><tr><td>Suggested Manhole Diameter (mm)</td><td>Site Specific Design (Contact Hydro International)</td></tr></table> <table><tr><th>Control Points</th><th>Head (m)</th><th>Flow (l/s)</th><th>Control Points</th><th>Head (m)</th><th>Flow (l/s)</th></tr><tr><td>Design Point (Calculated)</td><td>2.200</td><td>93.0</td><td>Kick-Flo®</td><td>1.506</td><td>77.4</td></tr><tr><td>Flush-Flo™</td><td>0.698</td><td>92.5</td><td>Mean Flow over Head Range</td><td>-</td><td>79.1</td></tr></table> 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 <table><tr><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th></tr><tr><td>0.100</td><td>10.5</td><td>1.200</td><td>88.1</td><td>3.000</td><td>108.1</td><td>7.000</td><td>163.4</td></tr><tr><td>0.200</td><td>36.8</td><td>1.400</td><td>82.7</td><td>3.500</td><td>116.5</td><td>7.500</td><td>169.0</td></tr><tr><td>0.300</td><td>69.2</td><td>1.600</td><td>79.7</td><td>4.000</td><td>124.3</td><td>8.000</td><td>174.4</td></tr><tr><td>0.400</td><td>87.8</td><td>1.800</td><td>84.3</td><td>4.500</td><td>131.7</td><td>8.500</td><td>179.7</td></tr><tr><td>0.500</td><td>90.7</td><td>2.000</td><td>88.8</td><td>5.000</td><td>138.6</td><td>9.000</td><td>184.8</td></tr><tr><td>0.600</td><td>92.1</td><td>2.200</td><td>93.0</td><td>5.500</td><td>145.2</td><td>9.500</td><td>189.7</td></tr><tr><td>0.800</td><td>92.2</td><td>2.400</td><td>97.0</td><td>6.000</td><td>151.5</td><td></td><td></td></tr><tr><td>1.000</td><td>90.6</td><td>2.600</td><td>100.8</td><td>6.500</td><td>157.6</td><td></td><td></td></tr></table>			Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	0.000	1375.0	1.200	1375.0	1.210	0.0	Unit Reference	MD-SHE-0368-9300-2200-9300	Design Head (m)	2.200	Design Flow (l/s)	93.0	Flush-Flo™	Calculated	Objective	Minimise upstream storage	Application	Surface	Sump Available	Yes	Diameter (mm)	368	Invert Level (m)	9.800	Minimum Outlet Pipe Diameter (mm)	450	Suggested Manhole Diameter (mm)	Site Specific Design (Contact Hydro International)	Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)	Design Point (Calculated)	2.200	93.0	Kick-Flo®	1.506	77.4	Flush-Flo™	0.698	92.5	Mean Flow over Head Range	-	79.1	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	0.100	10.5	1.200	88.1	3.000	108.1	7.000	163.4	0.200	36.8	1.400	82.7	3.500	116.5	7.500	169.0	0.300	69.2	1.600	79.7	4.000	124.3	8.000	174.4	0.400	87.8	1.800	84.3	4.500	131.7	8.500	179.7	0.500	90.7	2.000	88.8	5.000	138.6	9.000	184.8	0.600	92.1	2.200	93.0	5.500	145.2	9.500	189.7	0.800	92.2	2.400	97.0	6.000	151.5			1.000	90.6	2.600	100.8	6.500	157.6		
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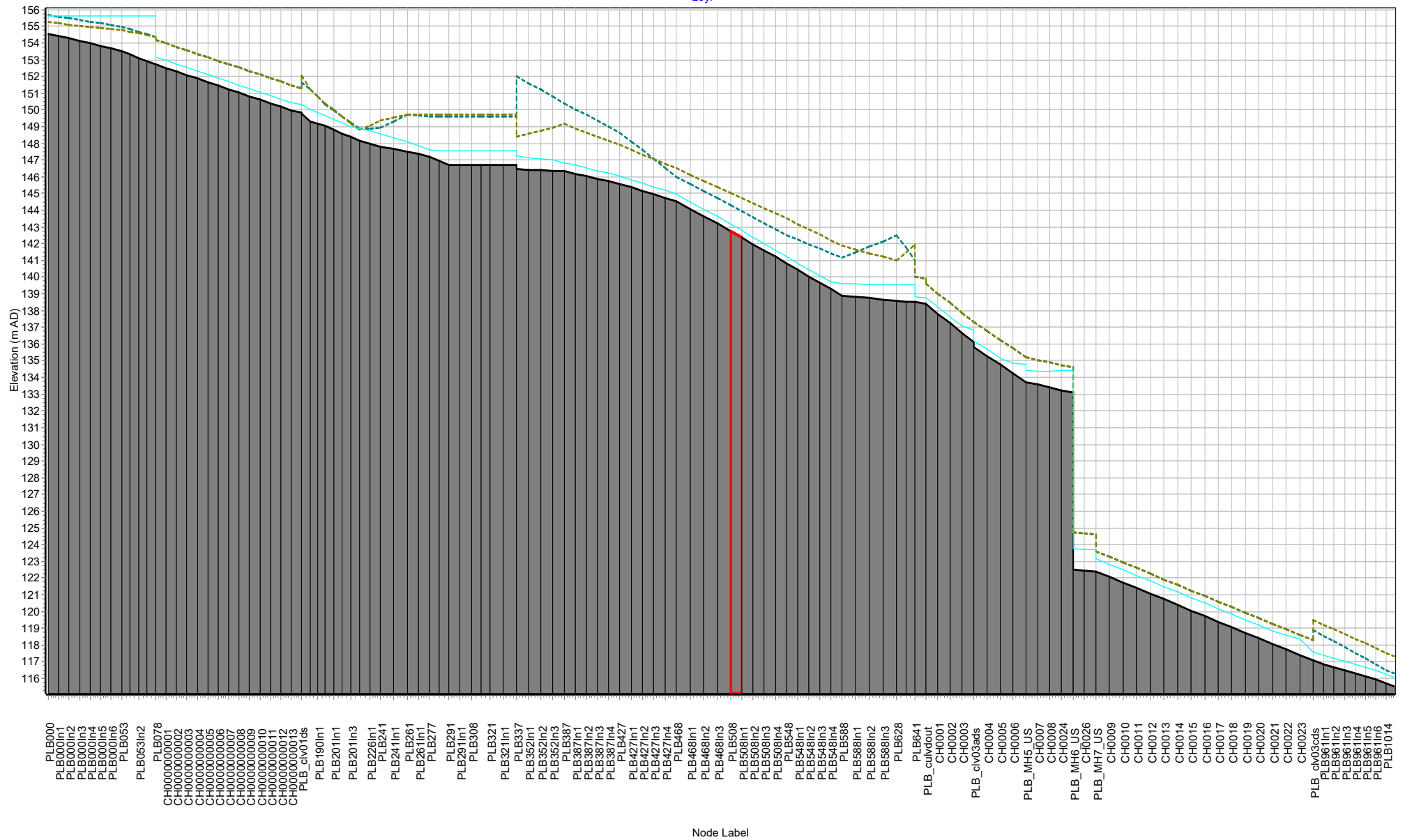
Existing Model - Long Section - Maximum Stage (mAOD)
1.20 year





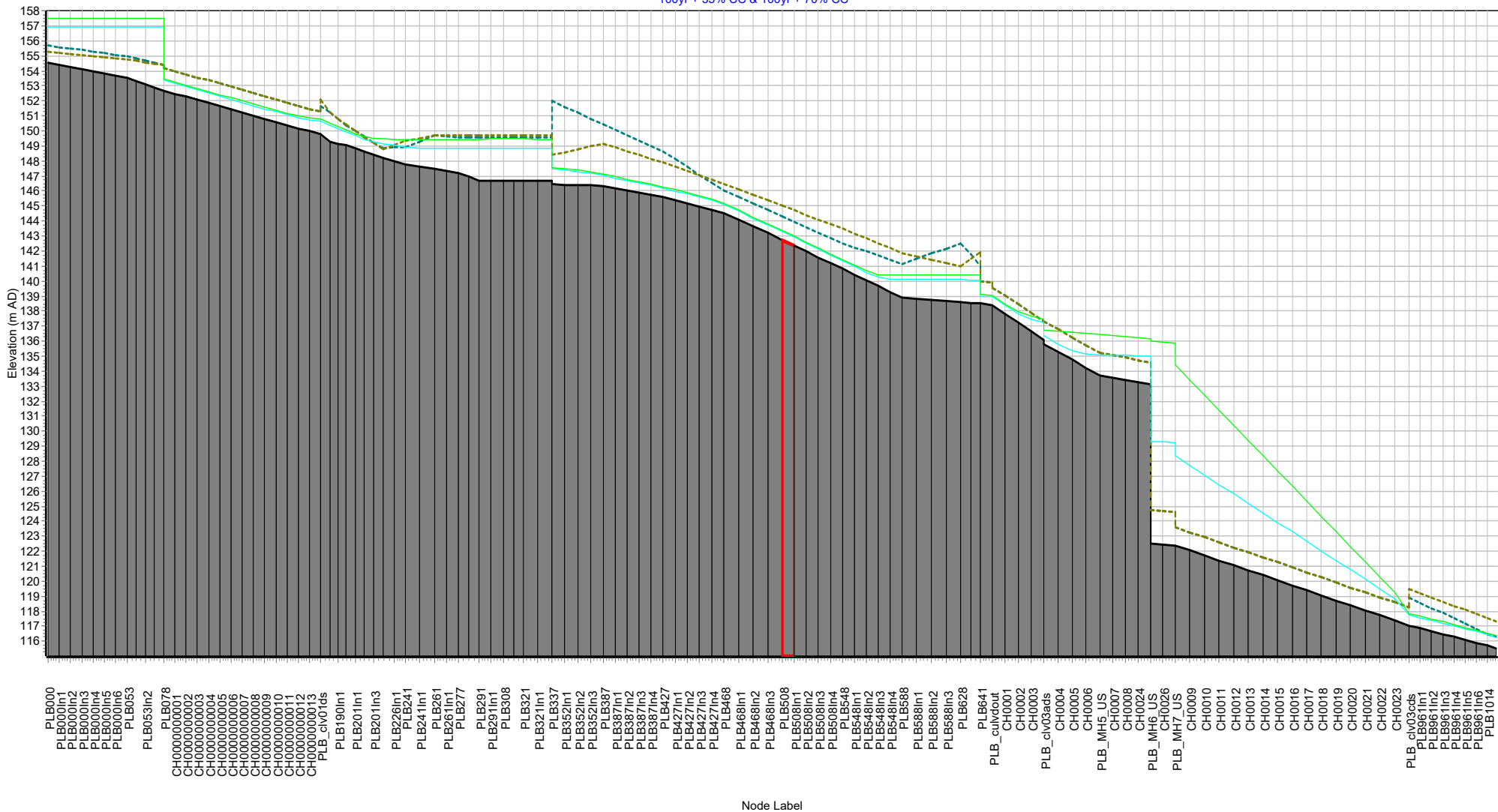


Proposed Model - Long Section - Maximum Stage (mAOD)
20yr



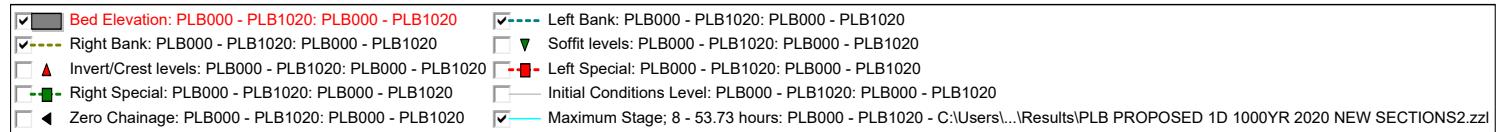
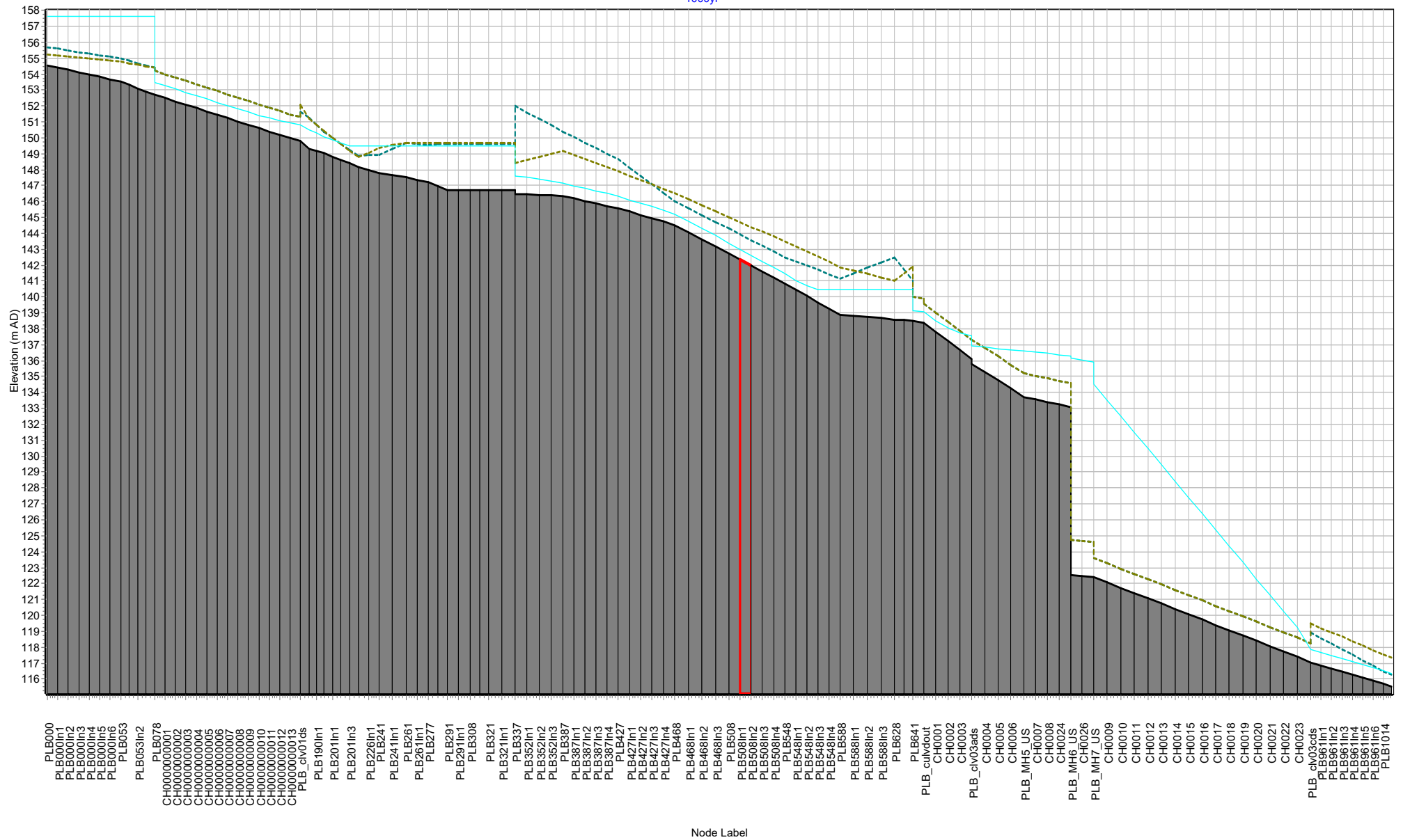
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|---|--|
| ☒ Red Elevation: PLB000 - PLB1020: PLB000 - PLB1020 | ☒ Left Bank: PLB000 - PLB1020: PLB000 - PLB1020 |
| ☒ Right Bank: PLB000 - PLB1020: PLB000 - PLB1020 | ☐ Soffit levels: PLB000 - PLB1020: PLB000 - PLB1020 |
| ☐ Invert/Crest levels: PLB000 - PLB1020: PLB000 - PLB1020 | ☐ Left Special: PLB000 - PLB1020: PLB000 - PLB1020 |
| ☐ Right Special: PLB000 - PLB1020: PLB000 - PLB1020 | ☐ Initial Conditions Level: PLB000 - PLB1020: PLB000 - PLB1020 |
| ☐ Zero Chainage: PLB000 - PLB1020: PLB000 - PLB1020 | ☒ Maximum Stage; 8 - 53.73 hours: PLB000 - PLB1020 - C:\Users\...\Results\PLB PROPOSED 1D 20YR 2020 NEW SECTIONS.zzl |

Proposed Model - Long Section - Maximum Stage (mAOD)
100yr + 35% CC & 100yr + 70% CC

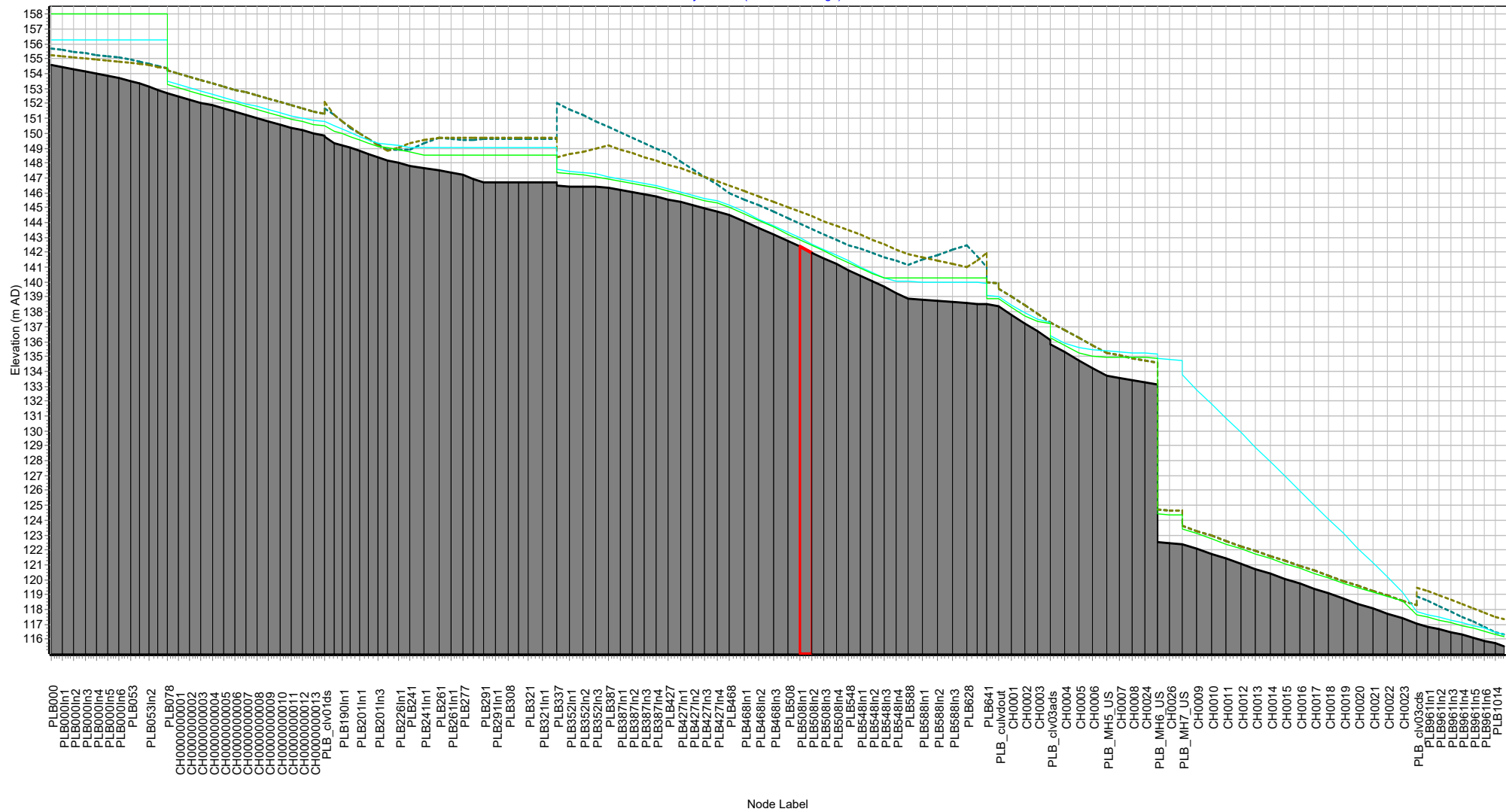


- ☒  Red Elevation: PLB000 - PLB1020: PLB000 - PLB1020
☒  Left Bank: PLB000 - PLB1020: PLB000 - PLB1020
☒  Right Bank: PLB000 - PLB1020: PLB000 - PLB1020
☐  Soffit levels: PLB000 - PLB1020: PLB000 - PLB1020
☐  Invert/Crest levels: PLB000 - PLB1020: PLB000 - PLB1020
☐  Left Special: PLB000 - PLB1020: PLB000 - PLB1020
☐  Right Special: PLB000 - PLB1020: PLB000 - PLB1020
☐ Initial Conditions Level: PLB000 - PLB1020: PLB000 - PLB1020
☐  Zero Chainage: PLB000 - PLB1020: PLB000 - PLB1020
☒  Maximum Stage: 8 - 53.73 hours: PLB000 - PLB1020 - C:\Users\...\Results\PLB PROPOSED 1D 100YR +35% 2020 NEW SECTIONS.zxl

Proposed Model - Long Section - Maximum Stage (mAOD)
1000yr

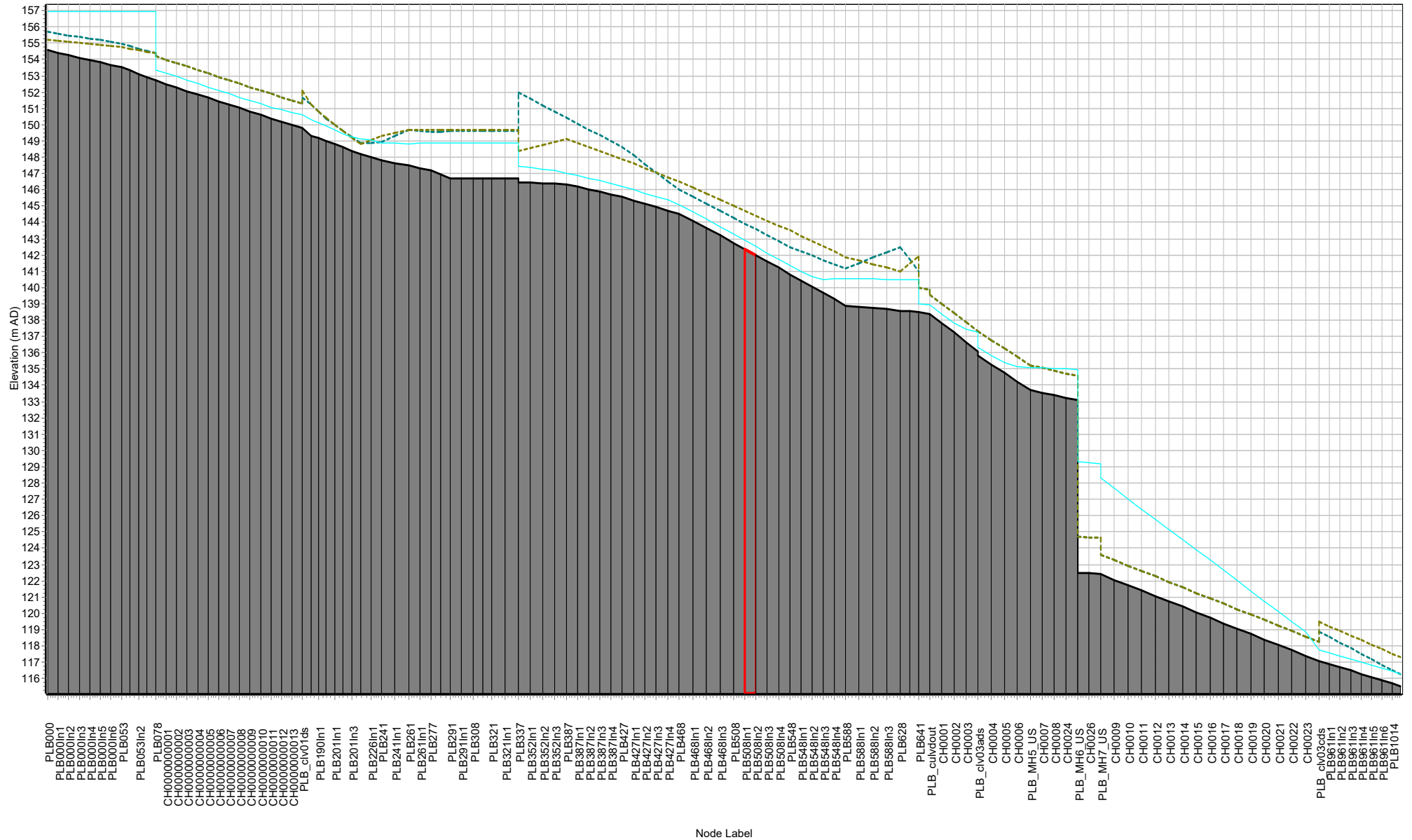


Proposed Model - Long Section - Maximum Stage (mAOD)
100yr + 35% (+20% Mannings)



- ☒  Red Elevation: PLB000 - PLB1020: PLB000 - PLB1020
☒  Left Bank: PLB000 - PLB1020: PLB000 - PLB1020
☒  Right Bank: PLB000 - PLB1020: PLB000 - PLB1020
☐  Soffit levels: PLB000 - PLB1020: PLB000 - PLB1020
☐  Invert/Crest levels: PLB000 - PLB1020: PLB000 - PLB1020
☒  Left Special: PLB000 - PLB1020: PLB000 - PLB1020
☐  Right Special: PLB000 - PLB1020: PLB000 - PLB1020
☐  Initial Conditions Level: PLB000 - PLB1020: PLB000 - PLB1020
☐  Zero Chainage: PLB000 - PLB1020: PLB000 - PLB1020
☒  Maximum Stage: 8 - 53.75 hours: PLB000 - PLB1020 - C:\Users\...Results\PLB PROPOSED 1D 100YR +35% 2020 NEW SECTIONS +20% MANNINGS.zzi

Proposed Model - Long Section - Maximum Stage (mAOD)
100yr + 35% CC (50% Blockage)



- | | |
|---|---|
| ☒ Bed Elevation: PLB000 - PLB1020: PLB000 - PLB1020 | ☒ Left Bank: PLB000 - PLB1020: PLB000 - PLB1020 |
| ☒ Right Bank: PLB000 - PLB1020: PLB000 - PLB1020 | ☐ Soffit levels: PLB000 - PLB1020: PLB000 - PLB1020 |
| ☐ Invert/Crest levels: PLB000 - PLB1020: PLB000 - PLB1020 | ☒ Left Special: PLB000 - PLB1020: PLB000 - PLB1020 |
| ☒ Right Special: PLB000 - PLB1020: PLB000 - PLB1020 | ☐ Initial Conditions Level: PLB000 - PLB1020: PLB000 - PLB1020 |
| ☐ Zero Chainage: PLB000 - PLB1020: PLB000 - PLB1020 | ☒ Maximum Stage; 8 - 53.75 hours: PLB000 - PLB1020 - C:\Users\...\Results\PLB PROPOSED 1D 100YR +35% 2020 NEW SECTIONS 50% BLOCKAGE.zzi |