D01 - Drainage Impact Assessment

Proposed Temporary Wholesales Trade (Food) in D.D. 111 and Adjoining Government Land, pat Heung, Yuen Long

Prepared for: Reitar Logtech Group Ltd
21 October 2024
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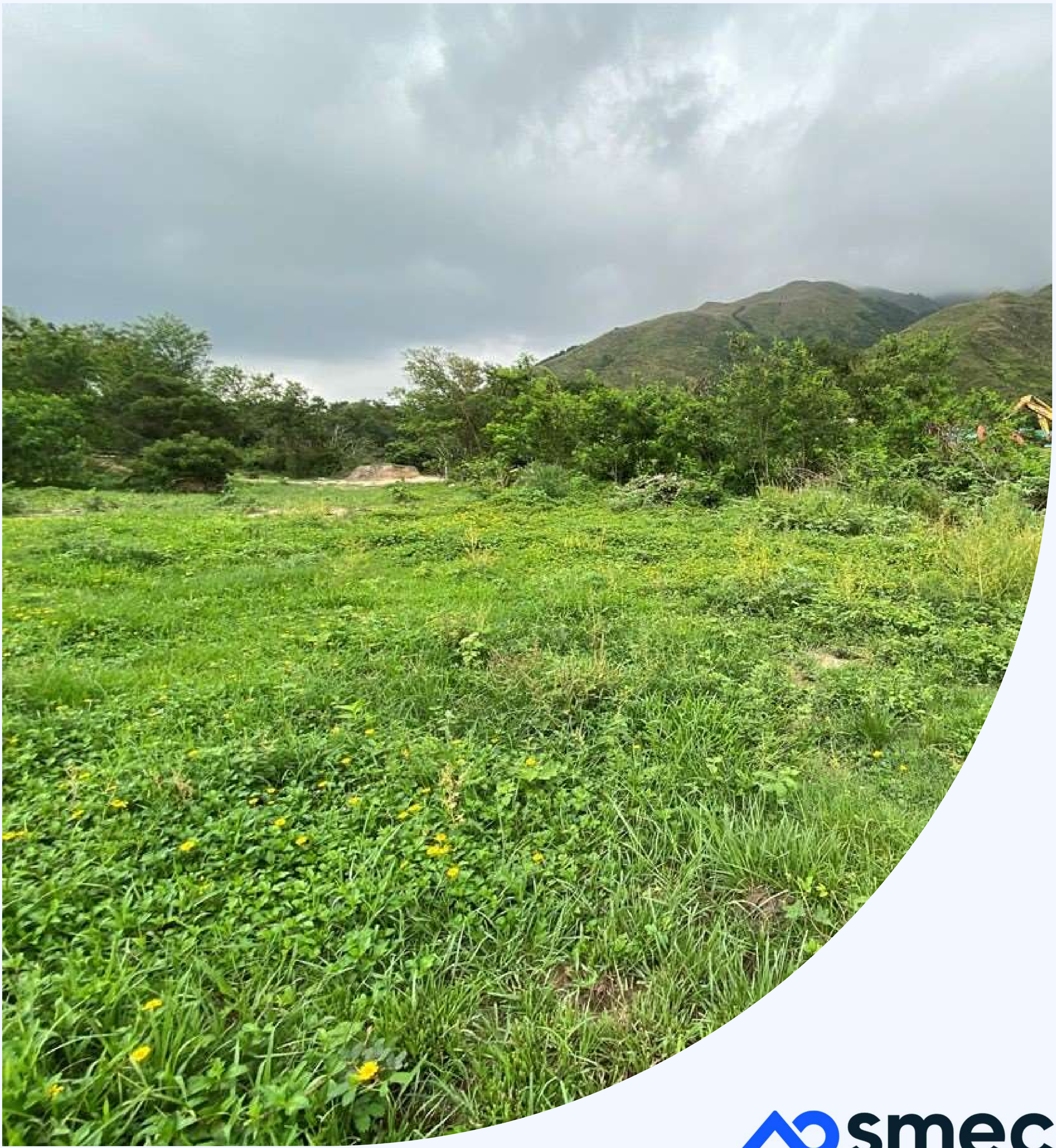

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1. Project Background

1.1 Introduction

1.1.1 A temporary wholesale trade (food) development (the Proposed Use) has been proposed for a period of five years at Lots 872, 873, 875, 876, 877, 878, 880, 881, 882, 883, 884, 885, 886, 887, 888, 889, 890, 891 (Part), 892 (Part), 893 (Part), 3049 and 3050 in DD 111 and adjoining government land, Pat Heung, Yuen Long (“the Site”). The Site is zoned “Open Storage” (OS) on the Approved Pat Heung Outline Zoning Plan (OZP) No. S/YL-PH/11. A planning application (no. A/YL-PH/804) for the Proposed Use was submitted under Section 16 of the Town Planning Ordinance (TPO) and was approved with conditions by the Town Planning Board (TPB) on 12 April 2019. Two of the approval conditions related to drainage issues are as follows:

(c) The submission of drainage proposal within 6 months from the date of planning approval to the satisfaction of the Director of Drainage Services or of the Town Planning Board by 12.10.2019; and

(d) In relation to (c) above, the implementation of drainage proposal within 9 months from the date of planning approval to the satisfaction of the Director of Drainage Services or of the Town Planning Board by 12.10.2019.

1.1.2 A drainage proposal has been submitted to Drainage Service Department (DSD) to discharge Approval condition (c). The submitted drainage proposal with a commitment made in the RtC was considered acceptable by DSD on 11 October 2021.

1.1.3 Further to the commitment in providing supplementary information to address comments from DSD (dated on 29 August 2023) and the comment from DSD dated 6 October 2023, the applicant is required to justify the capacity of the existing watercourse and taking into account the current revised design of the Proposed Development. SMEC Asia Ltd has been commissioned by Reitar Logtech Group Ltd to prepare a new Drainage Impact Assessment for the current revised design of the Proposed Development and justify the capacity of the existing watercourse.

1.2 Site Description

1.2.1 The Site location and its environs are shown on ***Figure 1-1*** in which the uses surrounding the Site include:

- To the North and East: Various open storage / storage yards, workshops, container trailers / tracker park.
- To the South: Village houses in Fu Shing Garden and Ha Che.
- To the West: Vacant land covered with vegetation under “Green Belt” zone.

1.2.2 The Site area is 21,586 m^2 and the General Building Plan (GBP) has been submitted to Building Department in January 2024.

1.3 Objectives of this Report

1.3.1 The objectives of this new Drainage Impact Assessment are to:

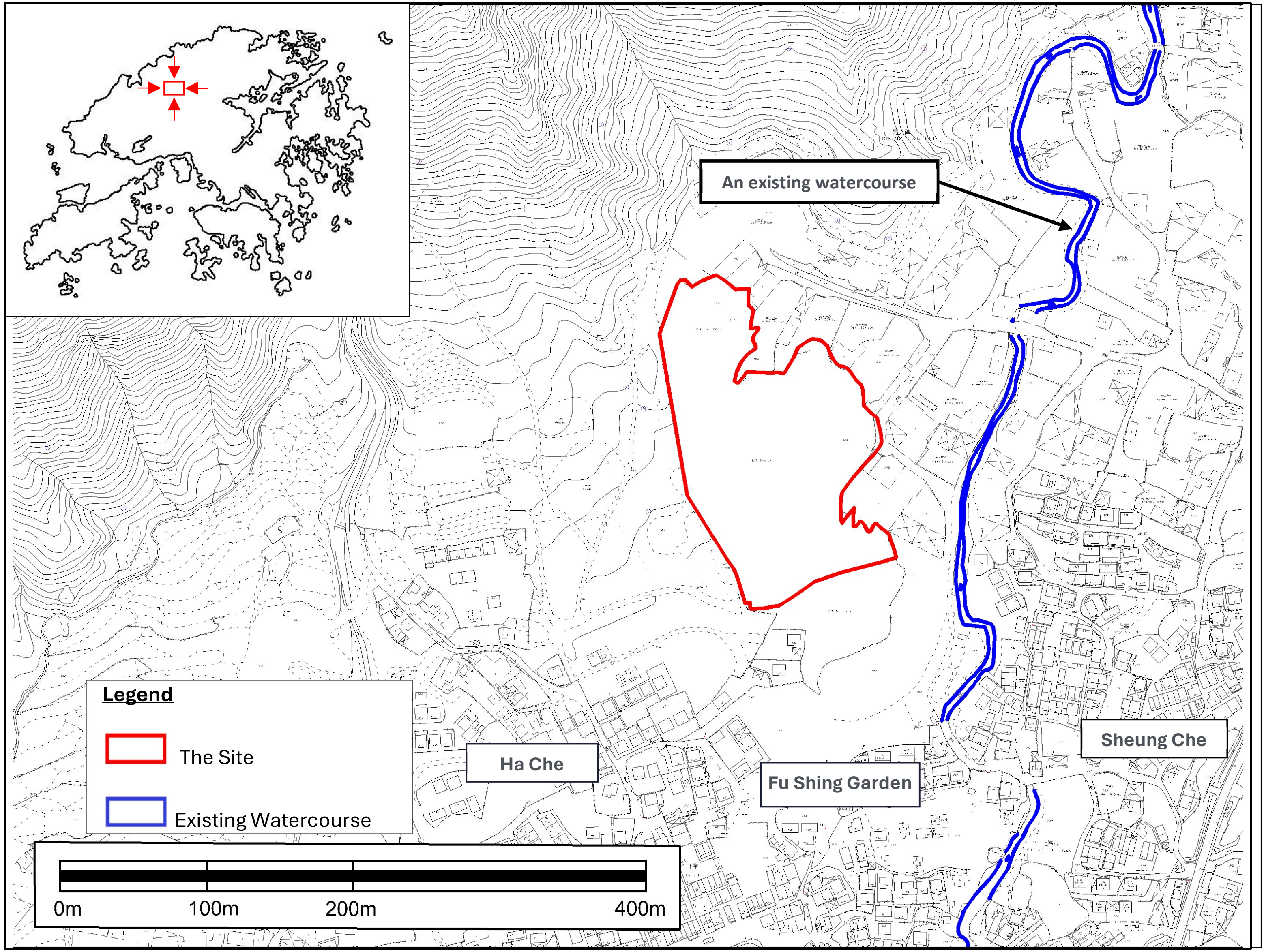
- Assess the potential drainage impact arising from the Proposed Development taking in account the current revised design and justify the capacity of the existing watercourse.
- Recommend the necessary mitigation measures to alleviate any impacts.

1.4 Reference materials

1.4.1 In evaluating the drainage impact arising from the Proposed Use, the following materials have been referred to:

- Drainage Services Department (DSD) publication Stormwater Drainage Manual (with Eurocodes incorporated) – Planning, Design and Management (2018 Edition).
- DSD Stormwater Drainage Manual Corrigendum No. 1/2022.
- DSD Stormwater Drainage Manual Corrigendum No. 1/2024.
- DSD Advice Note No. 1 – Application of the Drainage Impact Assessment Process to Private Sector Projects.
- DSD publication Technical Note to prepare a "Drainage Submission".
- GeoInfo Map reviewed on 05 February 2024.
- Boundary conditions of the existing watercourse provided by DSD on 21 February 2024.
- Pre-CCTV Survey Report carried by Pipeline Drainage Ltd. conducted on 23 September 2020 for the existing pipe near the Site.
- Topographical Survey near Lot No. 858, 861 S. A, 864 S.C, 862, 872-873, 875-878, 880-893, 894 S. A & S. B, 895, 3049-3050, 3083 in D.D.111, Ha Che, Yuen Long, prepared by Keyland Surveying, Planning & GIS Co. Ltd on 24 January 2019.

Figure 1-1: Site Location and its Environs



2. DESCRIPTION OF EXISTING ENVIRONMENT AND DRAINAGE CONDITIONS

2.1 Site Topography and Characteristics

2.1.1 As illustrated on ***Figure 1-1***, the Site is situated on a vacant land to the north of Ha Che in Pat Heung, Yuen Long and surrounded by various open storage / storage yards, workshops, container trailers / tracker park, village houses and vacant land. As the planning application has been approved in 2019, the construction works has already commenced. The site is currently undergoing site formation in February 2024.

2.1.2 The land survey conducted before the commencement of construction indicated that the site surface before proposed development was generally coarse and uneven with small gradient. The elevation ranged from the highest 39.33mPD at the north west corner of the site to the lowest 28.13mPD was at the south east corner. Referring to the site investigation in 2020, the Site was covered with mostly grassland.

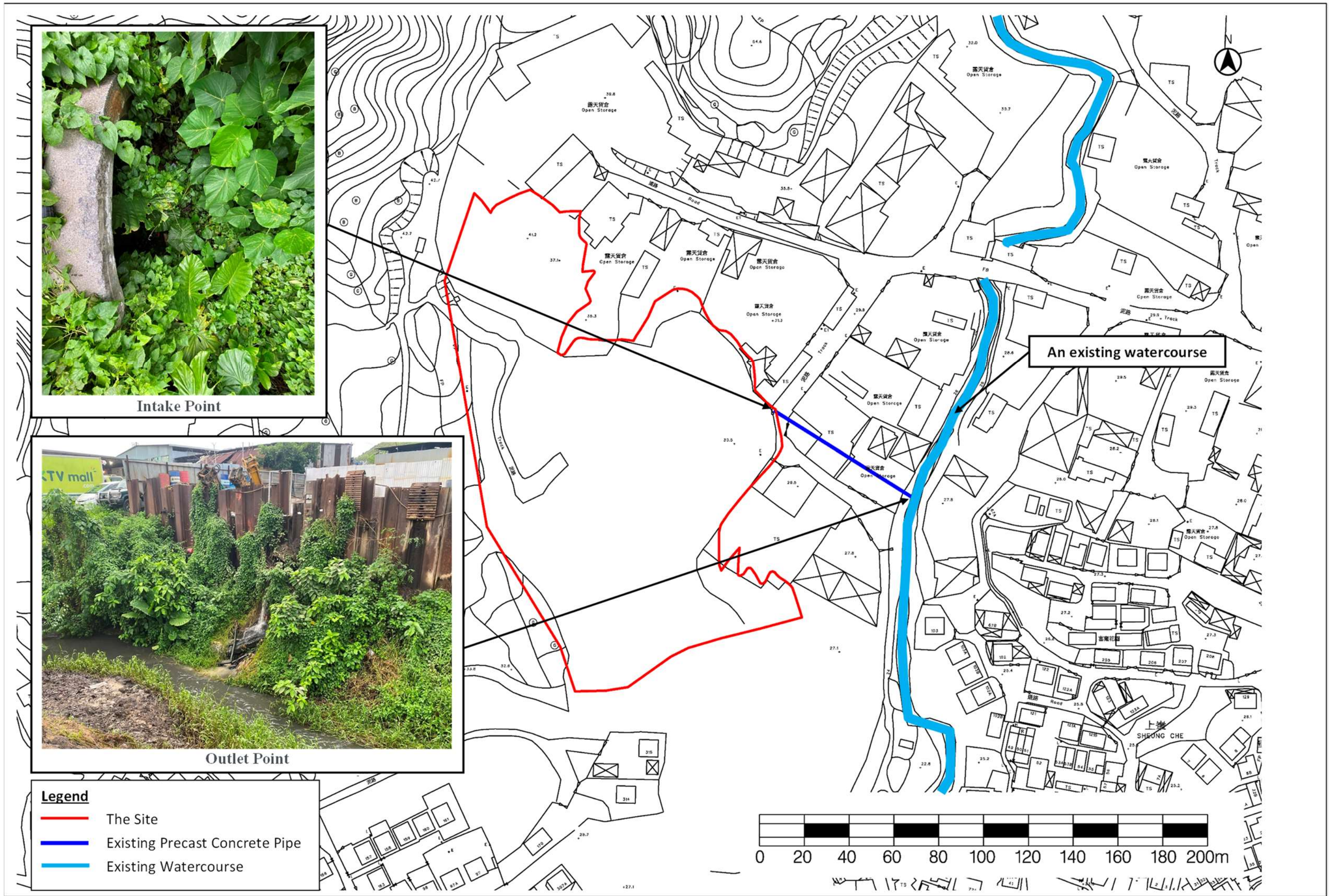
2.1.3 After the proposed development, the site will experience excavation and backfilling to enable the construction of power cap and superstructure. Within the building line, the ground level will be flattened to 34.0mPD while the uneven topographic level along the site boundary will be constructed as an emergency vehicle access (EVA) ramp, enclosing the superstructure of the proposed development. 1,058m^2 of the site area will be reserved for greenery. As reference, the cross sections showing the existing and proposed ground levels of the captioned site are presented as ***Appendix A***.

2.2 Baseline Drainage Conditions

2.2.1 With reference to GeoInfo Map and review on drainage layout records in DSD drawing office in May 2020 and February 2024, there is no municipal drainage system in the vicinity of the Site.

2.2.2 Based on the site observation and CCTV pipe inspection provided as ***Appendix B*** in this report, there is an existing precast concrete pipe connecting the eastern boundary of the Site to an existing watercourse to the east of the Site as shown on ***Figure 2-1***. The dimension of the precast concrete pipe is Ø1,800mm in diameter starting from the Site and then change to Ø600mm in diameter near the outlet at the watercourse. Hence, under the past drainage arrangement of the site, the runoff collected in site would be conveyed to by the precast concrete pipe, and discharged to the existing watercourse at the east of the Site. Siltation and collapse of the existing pipe was observed during the CCTV inspection.

Figure 2-1: Existing Drainage Arrangement of the Site



3. Drainage Impact

3.1 Assumptions and Methodology

3.1.1 Peak instantaneous runoff before and after the Proposed Use was calculated based on the Rational Method. The recommended physical parameters, including runoff coefficient (C) and storm constants for different return periods, are as per the *Stormwater Drainage Manual, and Stormwater Drainage Manual Corrigendum No. 1/2024*.

3.1.2 The Rational Method has been adopted for hydraulic analysis and the peak runoff is given by the following expression:

$$Q_p = 0.278\,C\,iA \qquad \textit{--- Equation 1}$$

where
Q_p = peak runoff in m^3/s
C = runoff coefficient
i = rainfall intensity in mm/hr
A = catchment area in km^2

3.1.3 Rainfall intensity is calculated using the following expression:

$$i = \frac{a}{(t_d + b)^c} \qquad \textit{--- Equation 2}$$

where
i = rainfall intensity in mm/hr
t_d = duration in minutes ($t_d \leq 240$)
a, b, c = storm constants given in Table 3 of *Stormwater Drainage Manual Corrigendum No. 1/2024*

3.1.4 For a single catchment, duration (t_d) can be assumed equal to the time of concentration (tc) which is calculated as follows:

$$t_c = t_0 + t_f \qquad \textit{--- Equation 3}$$

where
t_c = time of concentration
t_0 = inlet time (time taken for flow from the remotest point to reach the most upstream point of the urban drainage system)
t_f = flow time

3.1.5 Generally, t_0 is much larger than t_f. As shown in Equation 2, td is the divisor. Therefore, larger td will result in smaller rainfall intensity (i) as well as smaller Q_p. For the worst-case scenario, tf is assumed to be negligible and so:

$$t_d = t_c = t_0$$

$$t_0 = \frac{0.14465\,L}{H^{0.2}\,A^{0.1}} \qquad \textit{--- Equation 4}$$

where
A = catchment area (m^2)
H = average slope (m per 100 m), measured along the line of natural flow, from the summit of the catchment to the point under consideration
L = distance (on plan) measured on the line of natural flow between the summit and the point under consideration (m)

3.1.6 The capacities of the drainage pipes have been calculated using the Colebrook-Whit Equation, assuming full bore flow with no surcharge, as follows, in accordance with the Stormwater Drainage Manual:

$$V = -\sqrt{32gRs} \times \log\left(\frac{k_s}{14.8R} + \frac{1.25}{R\sqrt{32gRs}}\right) \qquad \textit{--- Equation 5}$$

where
V = mean velocity (m/s)
g = gravitational acceleration (m/s^2)

R = hydraulic radius (m)
k_s = hydraulic pipeline roughness (m)
υ = kinematic viscosity of fluid (m^2/s)
s = hydraulic gradient (energy loss per unit length due to friction)

3.1.7 On the other hand, the capacity of open channel has been calculated using the Manning's Equation:

$$V = \frac{R^{2/3} \times S^{1/2}}{n} \quad \textbf{\textit{--- Equation 6}}$$

where V = mean velocity (m/s)
R = hydraulic radius (m)
n = Manning coefficient ($s/m^{1/3}$)
s = hydraulic gradient (energy loss per unit length due to friction)

3.1.8 Since ***Equation 4*** is derived for natural catchment. It will no longer be valid for the site after the proposed development. Making reference to the rainfall intensity estimation approach in Highway Department *Guidance Notes on Road Pavement Drainage Design*, an arbitrary 5 minutes inlet time will be adopted for the calculation of runoff from the Site after proposed development.

3.2 Assessment Assumption

Identification of Catchment

3.2.1 A total of 8 Catchments were identified in reference to the basemap obtained from Lands Department Hong Kong Map Service website in February 2024. The identified catchments as well as their flow paths has been drawn in ***Figure 3-1***.

3.2.2 As no other identified stream or drainage system at the vicinity of the Site, it is anticipated all the runoff from the identified catchments is collected through the nearby watercourse described in ***Section 2.2*** and presented in ***Figure 2-1***.

3.2.3 Catchment A is the Site. Before the proposed development, its central and south eastern corner was at lower elevation than its edge. The runoff collected in Catchment A was conveyed through the precast concrete pipe. Catchment B, C and D are the hillside and footing of Kai Kung Leung. It is estimated that the runoff generated from Catchment B, C, D will be intercepted by Catchment A. Catchment E, F, G is the open storage area at the vicinity of the site. Since the precast concrete pipe passes through Catchment G and two connection points to the pipe is found within Catchment F. The runoff from Catchment F is very likely conveyed through the concrete pipe to the watercourse. The Catchment H is currently a construction site embraced by large area of greenery before development. Based on desktop study, the site formation on the Site has already commenced, and there is concurrent construction activity within Catchment H. The photo taken in September 2020 showing the previous condition of the Site and the surrounding catchments before commencement of construction are provided in ***Appendix C***.

3.2.4 After the Proposed Development, it is anticipated that surface characteristic of the Site will be changed. After the Proposed Development, the Site will become mostly paved. The runoff coefficient of the Site will change from 0.15 to 0.91 accordingly. Even though there is concurrent construction activity on Catchment H, as a conservative approach, the surface characteristic of the construction area within Catchment H (around 30% of the Catchment H area) is assumed to be paved area. Making reference to Stormwater Drainage Manual, the runoff coefficient of each identified catchment at the vicinity of the site is calculated and shown in ***Table 3.1***.

Table 3.1: Surface Characteristic of Identified Catchments

Catchment ID	Area (km^2)	Surface Characteristics	Runoff Coefficient
A (before proposed development)	21586	100% flat grassland	0.15
A (after proposed development)	21586	5% flat grassland + 95% paved	0.91
B	11806	100% steep grassland	0.2
C	31282	100% steep grassland	0.2
D	10335	100% flat grassland	0.15
E	14805	100% concrete paved	0.95
F	5783	100% concrete paved	0.95
G	4190	100% concrete paved	0.95
H	27154	70% flat grassland + 30% paved	0.39

3.3 Estimation of Runoff

3.3.1 To access the drainage impact of the proposed development, the runoff generated from each catchment has been calculated with rainfall intensity of 2 years, 10 years and 50 years return period coupled with 11.1% rainfall increase projection at the middle 21st century. The design allowance is not considered in this DIA because of the temporary usage nature of the proposed development. The calculation result is shown in ***Table 3.2***. It is estimated that the proposed development will increase the runoff from the Site by $1.241m^3/s$, from $0.206m^3/s$ to $1.447m^3/s$ under rainfall of 50 years returning period. The detailed calculation of runoff from each catchment under different returning period is provided in ***Appendix D***.

Table 3.2: Runoff from each Identified Catchment

Catchment	Runoff during Rainfall of each Returning Period (m^3/s)		
	2 years	10 years	50 years
A (before proposed development)	0.138	0.179	0.206
A (after proposed development)	1.011	1.282	1.447
B	0.123	0.155	0.175
C	0.249	0.326	0.378
D	0.070	0.090	0.103
E	0.649	0.833	0.952
F	0.331	0.413	0.458
G	0.227	0.284	0.317
H	0.467	0.603	0.692

3.4 Proposed Drainage Layout

Internal Drainage System

3.4.1 As mentioned in ***Section 3.2.3***, the runoff generated in Catchment B, C, D are intercepted by Catchment A, the Site. Beside the runoff generated on site after proposed development, the internal drainage design should also take the runoff from Catchment B, C, D into consideration. Therefore, the peak runoff that will be sustained by the internal drainage system of the Site is estimated to be $2.104m^3/s$.

3.4.2 In order to intercept the overland flow from the nearby catchments and convey the collected runoff to terminal manhole, series of channels will be constructed along the periphery of the Site, one from CP09 to CP01 and the other series from CP19 to CP16. One additional channel will be from CP9 to CP10 and will be connected to CP19 via a pipe, and eventually connect to another channel which is to be constructed on the open yard of the proposed development to help convey the runoff collected in roof and rain gutters. The layout of the proposed internal drainage system as well as the cover levels and invert levels of drainage provisions has been shown in ***Figure 3-2***.

3.4.3 Based on the post development topographic level, the site, Catchment A, can be split into 4 sub-catchments. The sub-catchments within the Site are shown in ***Figure 3-3***. Assuming that the distribution of runoff into each section of channels approximately follows the projection of catchment area on each section of channels, the required dimensions of drainage channels/ pipes for the internal drainage system can be estimated. As the lowest level based on topography is at 29.05mPD while the terminal manhole invert level is 29.0mPD, it will require an underground retention tank to collect the stormwater from two other series channels, one from CP10 to CP12 and the other is from CP15 to CP 13. Runoff collected in these two series of channel will eventually discharge to Terminal Manhole via a 400mm diameter underground pipe. The schedule of channels and pipe for the proposed internal drainage system is presented in ***Appendix F***. Sand trap will be provided before the discharge to public drainage facilities. See ***Figure 3-2***. The typical details of the U-channel are referred to the corresponding Civil Engineering Development Department (CEDD) and DSD standard drawings and have been attached in ***Appendix E***. The calculation of channel capacity and sizing of the retention tank is provided in ***Appendix F***. In addition to the proposed internal drainage system, weephole on the retaining walls will be provided in accordance to Geoguide 1 and openings on fence wall will be reserved along the site boundary to intercept the overland flow passing through the site.

External Drainage System

3.4.4 The proposed external drainage system has been drawn in ***Figure 3-4***. As mentioned in previous section, the runoff generated on site as well as the intercepting catchments will be collected by the internal drainage system. The runoff collected in the U-channels will later flow through internal underground connection pipes from CP 16, CP 01 and an underground retention tank to the Terminal Manhole, and will eventually flow through the external drainage system and discharge to the nearby watercourse. Since the existing precast concrete pipe on Site was found damaged during the CCTV inspection, a new external drainage system is proposed. The new discharge point is right under the footbridge across the watercourse. The connection details of the discharge point is shown at ***Figure 3-5***.

3.4.5 The proposed external drainage system consists of a 1500mm underground circular precast concrete pipe in a gradient of 1:200. Because the proposed external drainage system will pass through Catchment E, the runoff generated on Catchment E will be taken into consideration in the hydraulic assessment of the external drainage system at the downstream of manhole RMH-X1. In this regard, the total runoff flow through the 1500mm pipe will be 3.056m^3/s. The calculation of flow capacity of the external drainage system is ***Appendix G***.

Existing Watercourse

3.4.6 The photos of the existing watercourse are presented in ***Figure 3-6***. And the photo of the discharge point underneath the footbridge is presented as ***Figure 3-7***Figure 3-7. The information of the existing watercourse shown in ***Appendix G*** are obtained from DSD.

3.4.7 As the proposed storm water discharge point of the proposed development is at the downstream of Location A and Location B, and at the upstream of Location C and Location D, the hydraulic assessment of the watercourse will be conducted by the estimation of available flow capacity at Location C and Location D under a 10-year sea level in conjunction with a 50-year rainfall, which is the scenario generating the maximum amount of runoff, and resulting in the highest water level in the watercourse.

3.4.8 As shown in ***Appendix G***, the peak water level at Location C of the watercourse is at 24.074mPD, and at Location D is at 20.232mPD, and the peak flow is 28.848 m^3/s and 28.990 m^3/s respectively. The bank level of the watercourse at Location C and Location D is at 24.90mPD and 20.70mPD. Under the uniform flow condition, the velocity of an open channel depends on hydraulic radius, surface roughness, and channel gradient. With the information provided by DSD, the peak velocity and the corresponding peak water level is given, the hydraulic property of the watercourse at Location C and D can therefore be back calculated, enabling the estimation of watercourse capacity under different water level. Reserving a 300mm freeboard in reference to Stormwater Drainage Manual, the maximum capacity of watercourse has been calculated to be 62.72 m^3/s at Location C and 37.78 m^3/s at Location D as shown in ***Table 3.3***. The detailed calculation breakdown is shown in ***Appendix H***. Under the scenario of a 10-year sea level in conjunction with a 50-year rainfall, the available flow capacity of the watercourse at Location C and D is 33.88m^3/s and 8.8m^3/s. As Location D is at the downstream of Location C, the maximum allowable stormwater discharge to the watercourse will be 8.8m^3/s.

Table 3.3: Capacity of Watercourse at Location C and D

Location C	Water Level (mPD)	Hydraulic Radius (m)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Available Capacity (m^3/s)
Existing Boundary Condition	24.074	0.71	5.40	28.85	-
300mm freeboard	24.600	0.98	6.67	62.72	33.88
Location D	**Water Level (mPD)**	**Hydraulic Radius (m)**	**Peak Velocity (m/s)**	**Peak Flow (m^3/s)**	**Available Capacity (m^3/s)**
Existing Boundary Condition	20.232	1.25	2.54	28.99	-
300mm freeboard	20.400	1.33	2.65	37.78	8.80

3.4.9 Mentioned in ***Section 3.3.1***, the additional runoff generated from the change of site characteristic during the proposed development has been estimated to be 1.241m^3/s, which is far lower than the allowable discharge 8.8m^3/s. Based on the analysis, the existing watercourse has sufficient capacity to sustain the drainage impact from the proposed development, and no adverse flooding risk due to the proposed development will be anticipated.

3.5 Summary

3.5.1 The runoff generated from the site before and after the proposed development as well as the nearby catchments has been calculated. Under rainfall intensity of 50 years returning period with consideration of climate change effect at the middle 21st century, a total of 2.104m^3/s of runoff will be intercepted by the Site, including the runoff generated on site and the adjacent hillside catchments.

3.5.2 To mitigate the drainage impact from the proposed development, series of channels have been proposed as the internal drainage system to intercept overland flow and collect storm water before discharging to the existing watercourse through underground pipes and retention tank. The total runoff that would be discharged to the watercourse through the proposed external underground pipes is estimated to be 3.056m^3/s under rainfall intensity of 50 years returning period with consideration of climate change effect at the middle 21st century.

3.5.3 The available capacity of the existing watercourse has been assessed from the information provided by DSD. It is expected that the watercourse can handle the increment of runoff caused by the proposed development.

3.5.4 No adverse impact is anticipated from the proposed development after the provision of series of channels and pipes together with a retention tank as internal drainage system and the proposed 1500mm underground pipe as external drainage system.

Figure 3-1: Identified Catchments

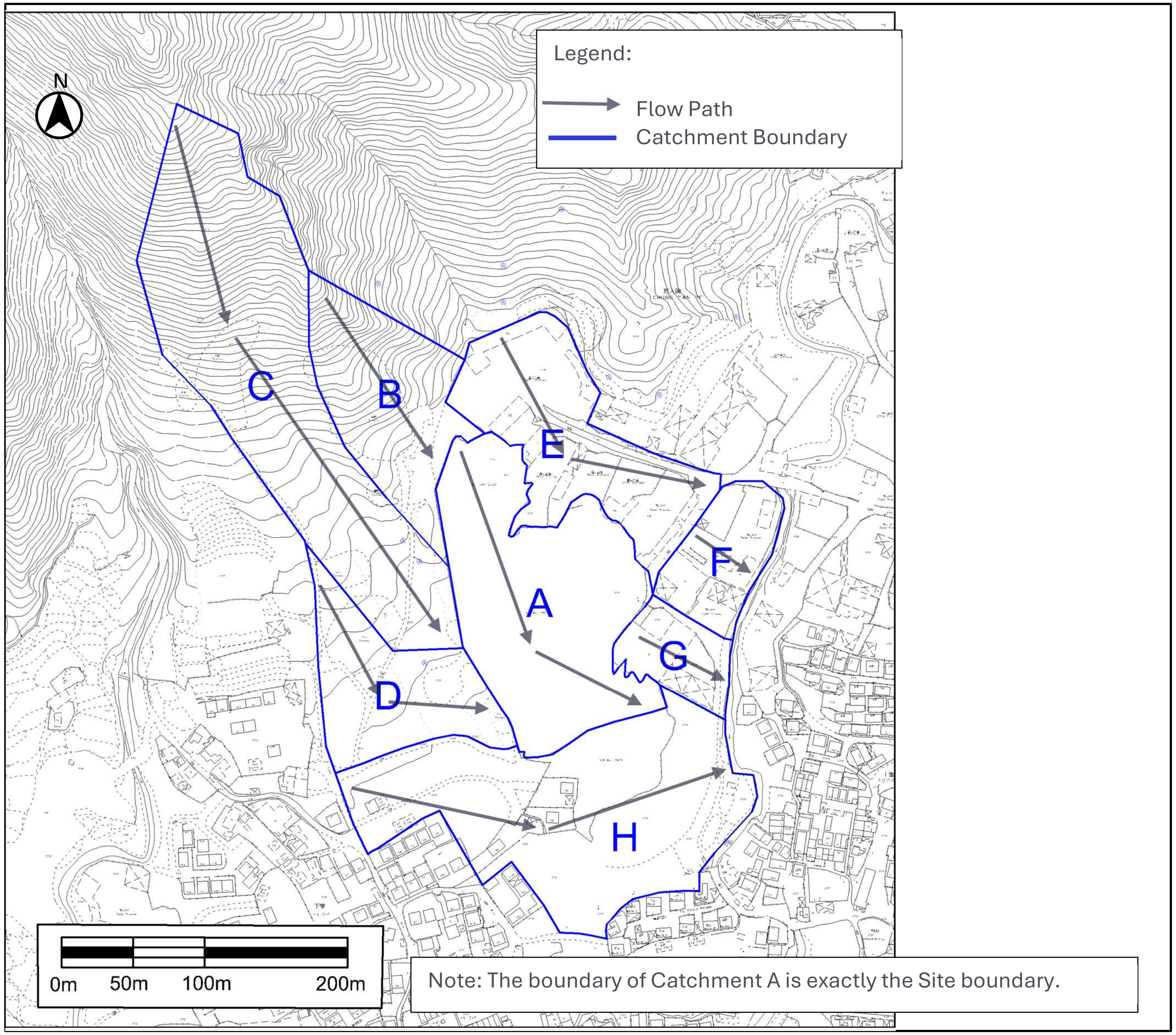


Figure 3-2: Proposed Internal Drainage System

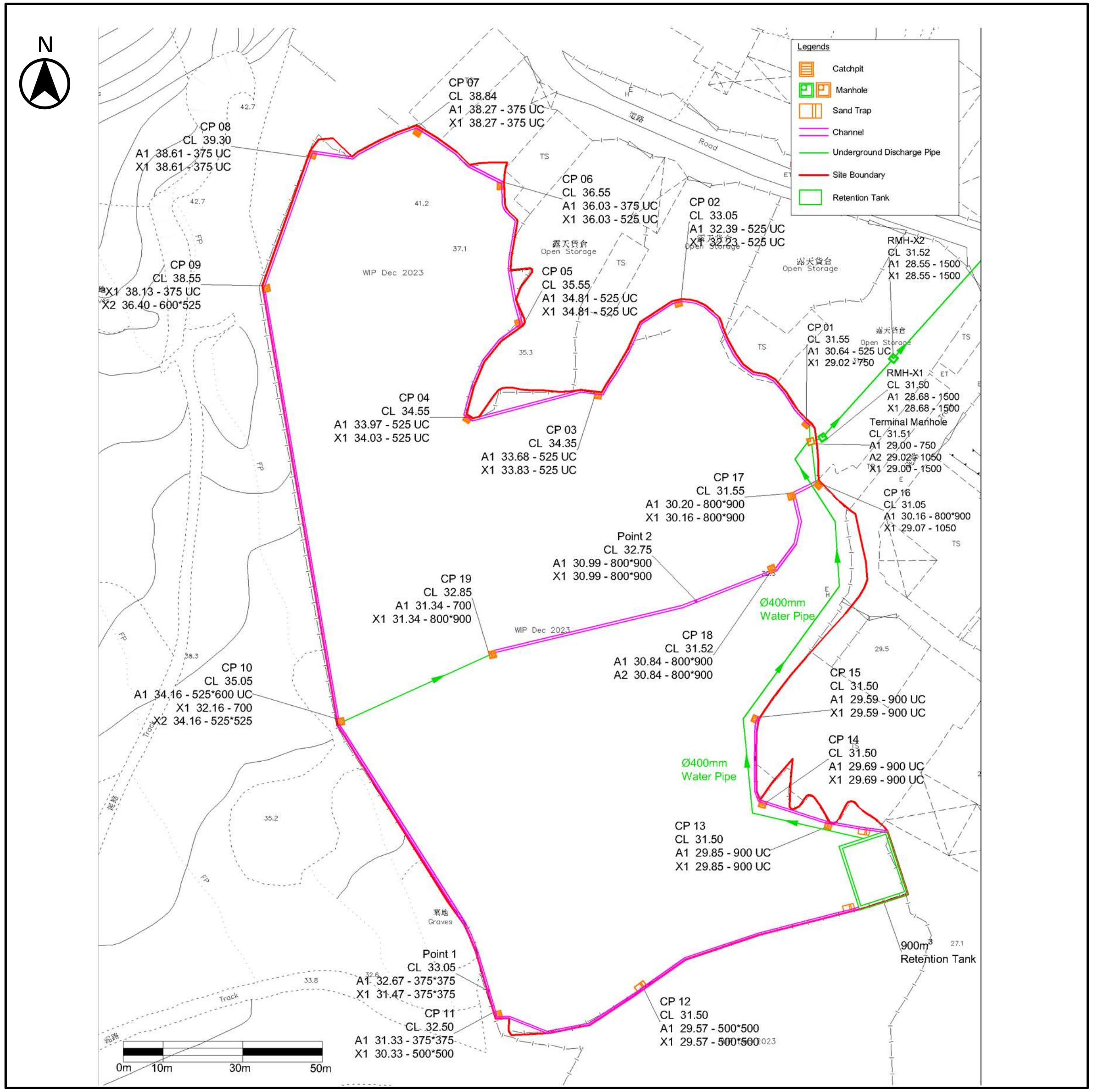


Figure 3-3: Sub-catchment within the Site

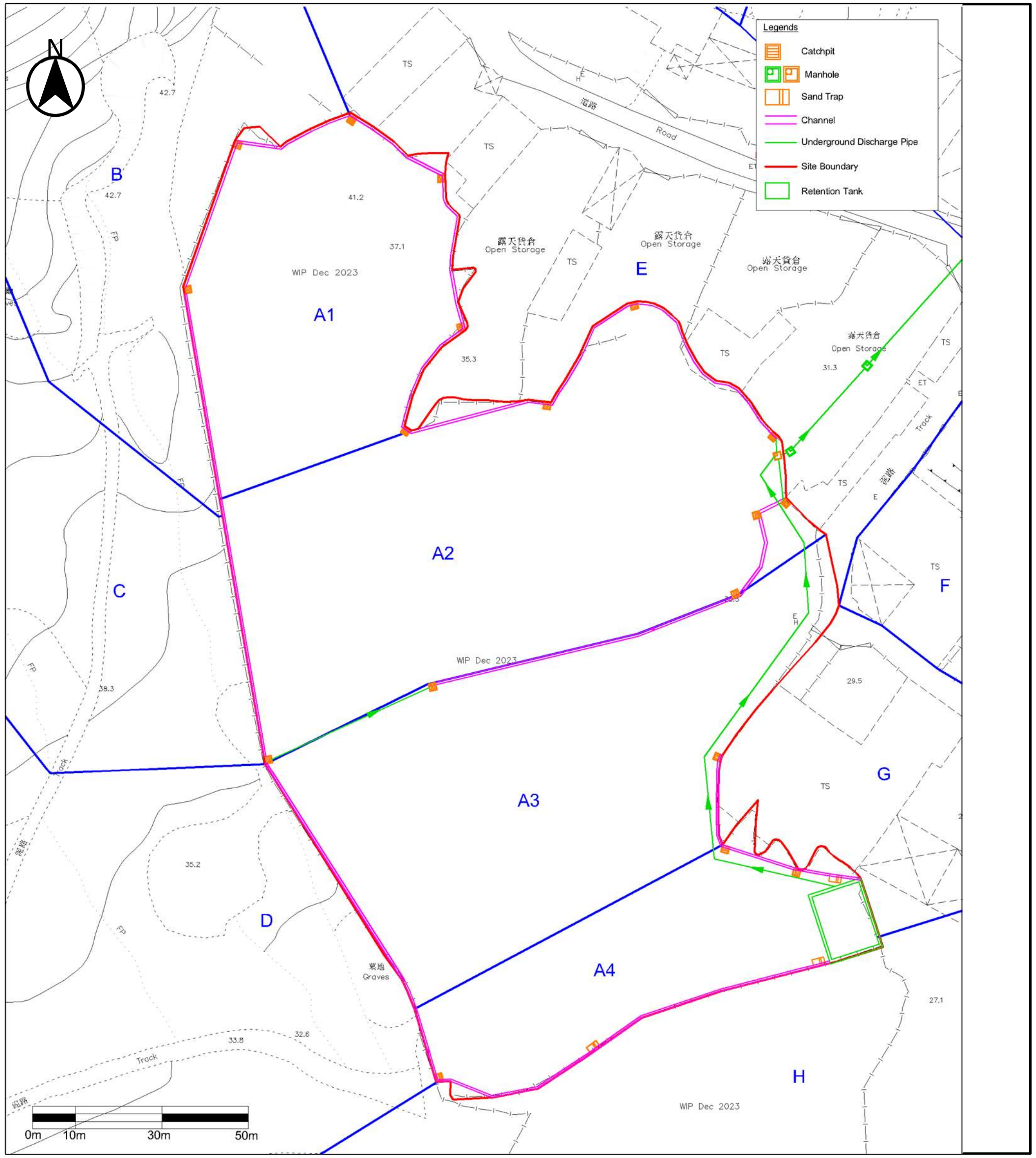


Figure 3-4: Proposed External Drainage System

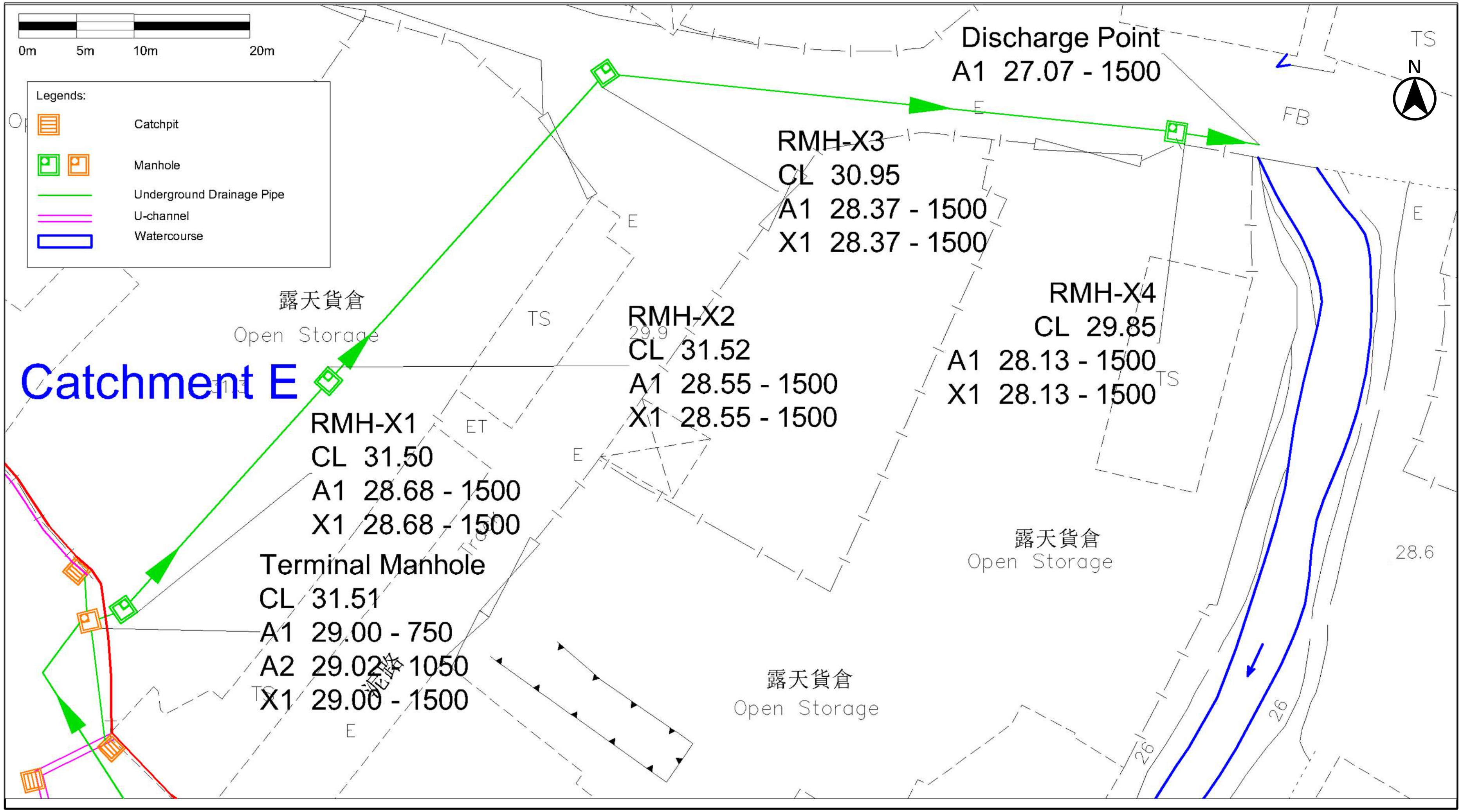


Figure 3-5: Connection Detail of Footbridge

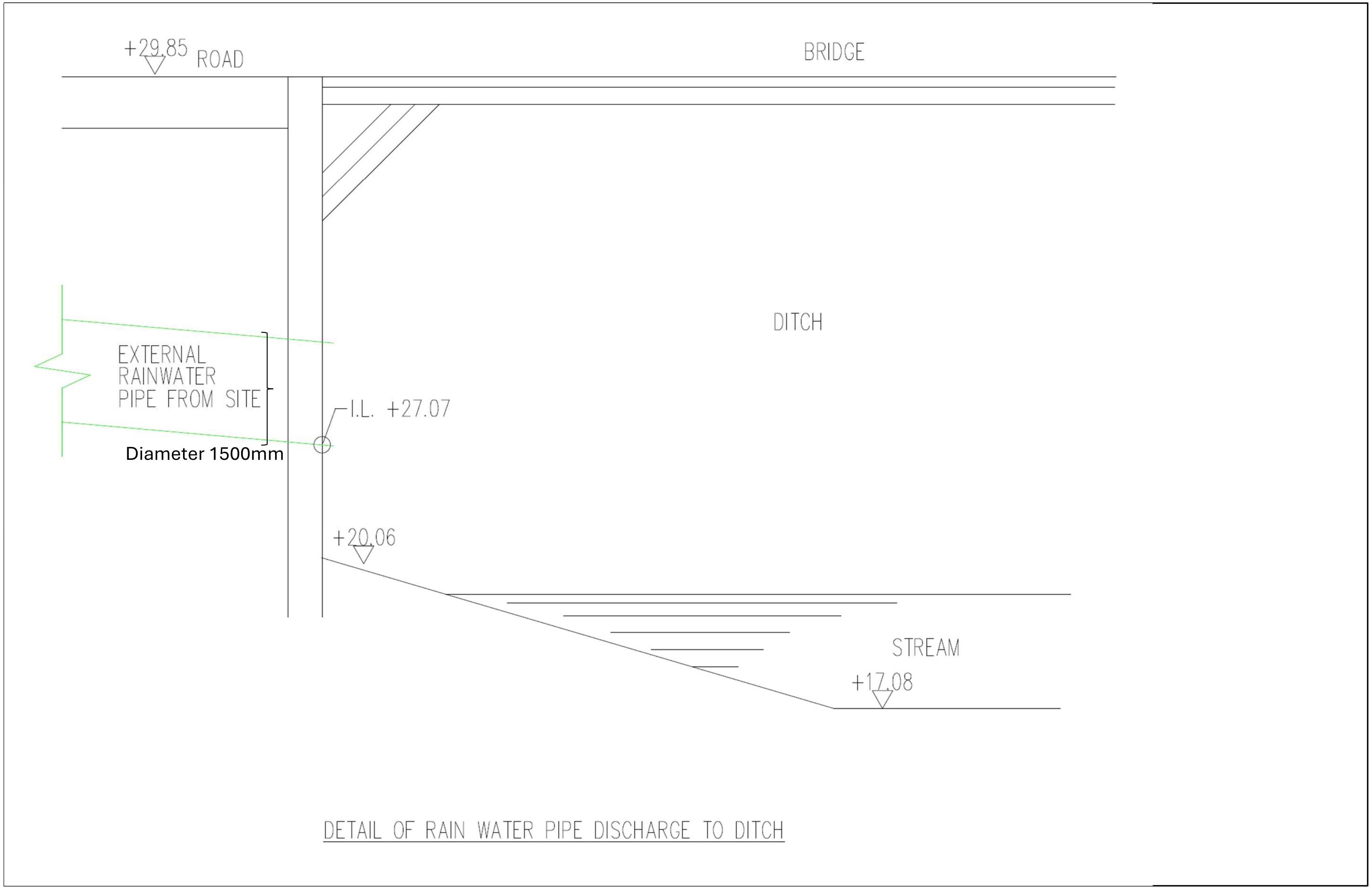


Figure 3-6: Photos of the Existing Watercourse

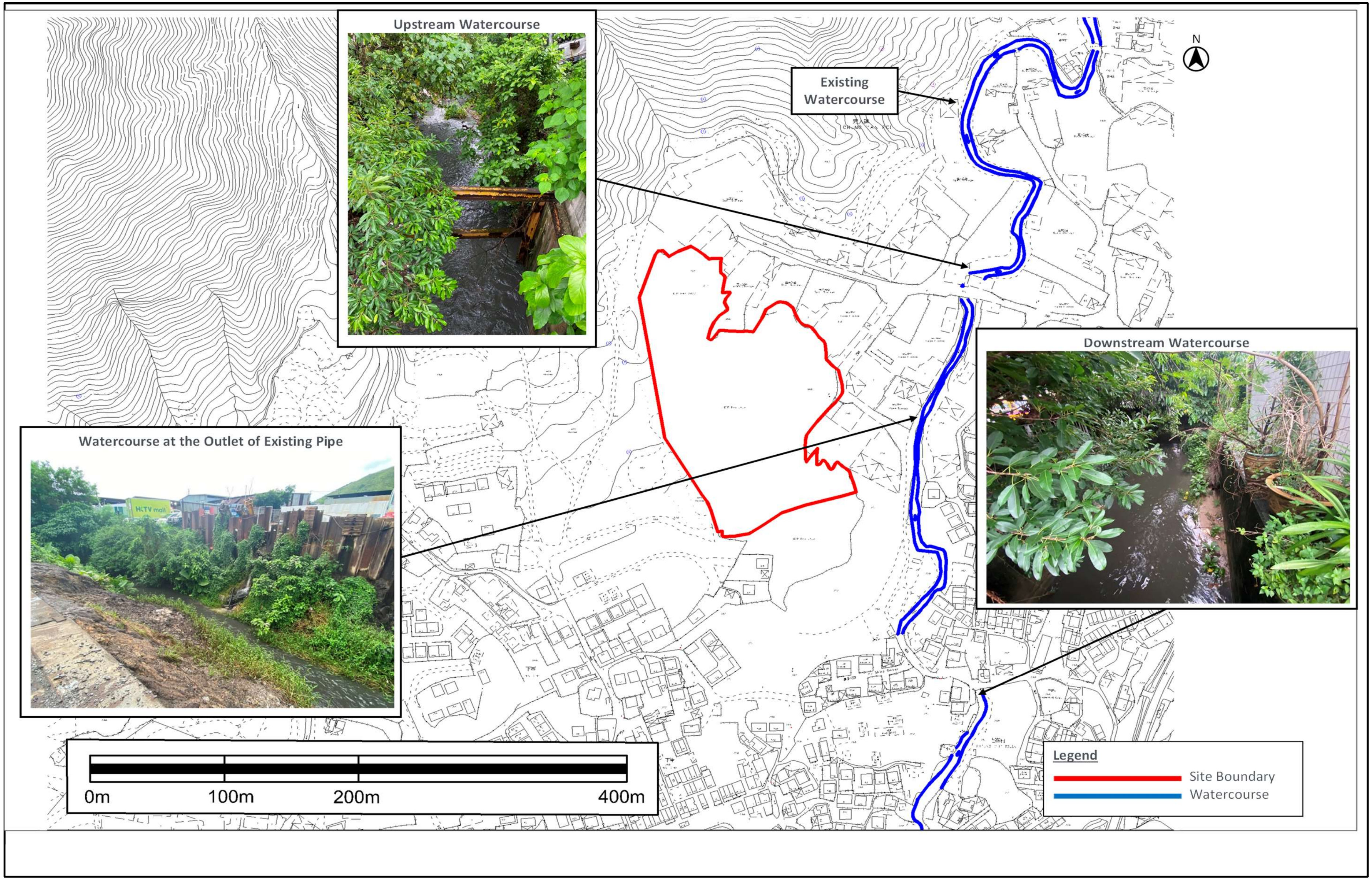


Figure 3-7: Photo of the Existing Watercourse underneath Footbridge

4. Conclusion

4.1.1 The surface characteristic and the drainage arrangement of the site and the nearby catchments has been discussed in this report. Potential drainage impacts that may arise from the Site after construction of the Proposed Development have been assessed.

4.1.2 The peak runoff before and after the development of the Site has been estimated using Rational Method and based on the catchment surface characteristics for the existing environment and the Proposed Development. The total runoff generated from the Site and the intercepting catchments has been estimated to be 2.104m^3/s under the rainfall intensity of 50 years returning period with the consideration of 11.1% rainfall increase projection at the middle 21st century.

4.1.3 To mitigate the drainage impact from the proposed development, series of channels, pipes and a retention tank system have been proposed as the internal drainage system to intercept overland flow and collect storm water before discharging to the existing watercourse through underground pipes. The total runoff that would be discharged through the external underground pipes is estimated to be 3.056m^3/s under rainfall intensity of 50 years returning period with consideration of climate change effect at the middle 21st century.

4.1.4 Based on the information provided by DSD, the available capacity of the existing watercourse has been assessed. It is expected that the watercourse can handle the increment of runoff caused by the proposed development.

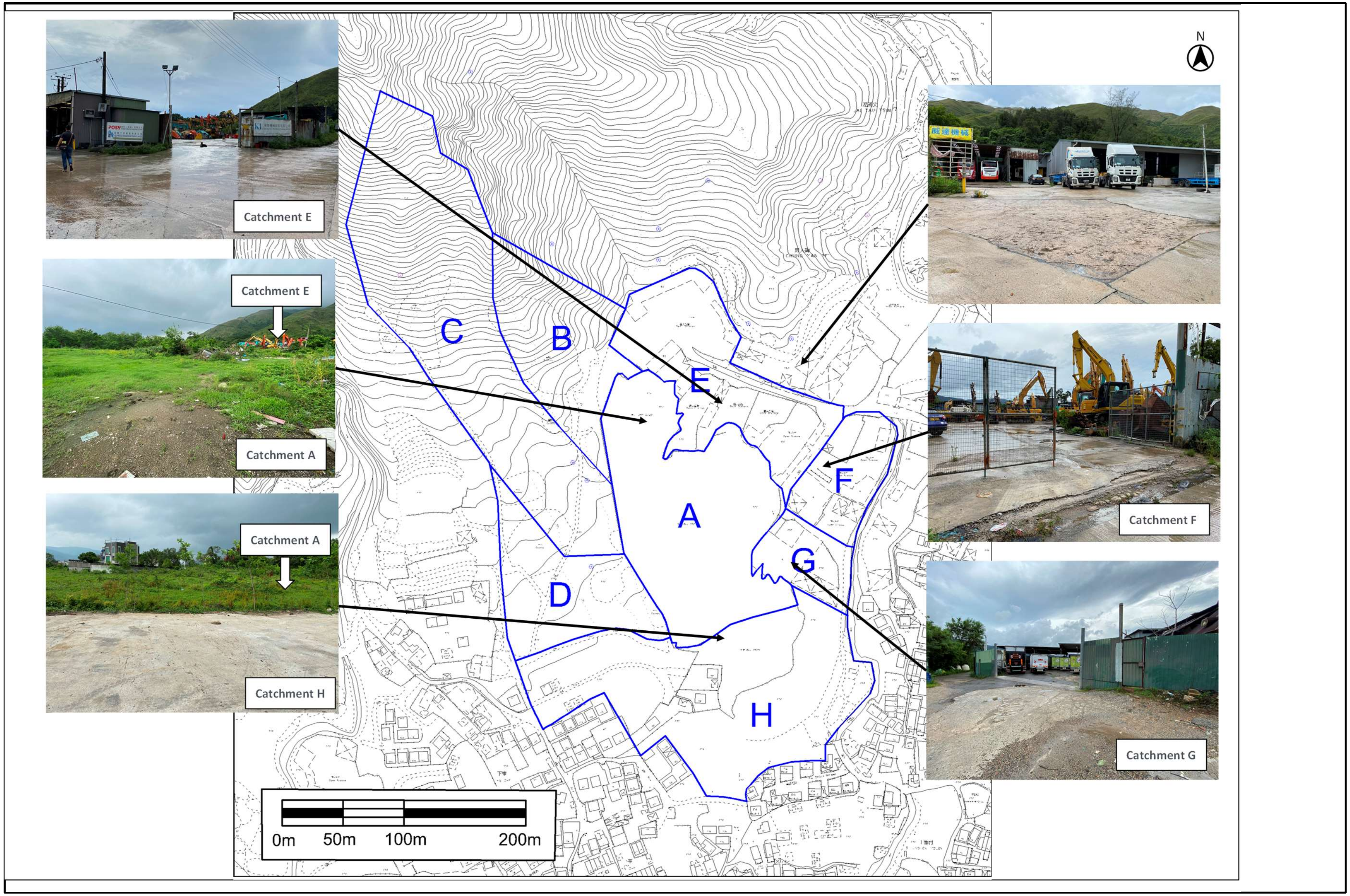
4.1.5 No adverse impact is anticipated from the proposed development after the provision of 3 series of U-channel as internal drainage system and the construction of 1500mm underground pipe as external drainage system.

Appendix A Cross-section of the Site

Appendix B CCTV Pipe Inspection Report

Appendix C Condition of the Site and the Surrounding Catchments

Figure 4-1: Condition of the Site and the Surrounding Catchment Areas



Appendix D Runoff Calculation

Calculation of Runoff for Return Period of 2 Years

Catchment ID	Catchment Area (A), km^2	Average slope (H), m/100m	Flow path length (L), m	Inlet time (t_0), min	Duration (t_d), min	Storm Constants			Runoff intensity (i) mm/hr	Runoff coefficient (C)	C x A	Peak runoff (Q_p), m^3/s
						a	b	c				
Before the Proposed Development												
A (Site)	0.0216	4.77	234.9	9.16	9.16	446.1	3.38	0.463	138.31	0.15	0.0032	0.138
B	0.0118	36.14	175.0	4.84	4.84	446.1	3.38	0.463	168.25	0.20	0.0024	0.123
C	0.0313	28.94	427.0	11.19	11.19	446.1	3.38	0.463	129.04	0.20	0.0063	0.249
D	0.0103	7.66	206.6	7.89	7.89	446.1	3.38	0.463	145.33	0.15	0.0016	0.070
E	0.0148	5.40	183.4	7.25	7.25	446.1	3.38	0.463	149.35	0.95	0.0141	0.649
F	0.0058	6.51	61.5	2.57	2.57	446.1	3.38	0.463	195.35	0.95	0.0055	0.331
G	0.0042	7.82	80.6	3.35	3.35	446.1	3.38	0.463	184.47	0.95	0.0040	0.227
H	0.0272	4.61	216.7	8.32	8.32	446.1	3.38	0.463	142.86	0.39	0.0106	0.467
											Total (General Scenario)	**2.254**

After the Proposed Development												
A (Site)	0.0216	-	-	5.00	5.00	446.1	3.38	0.463	166.71	0.91	0.0196	1.011
B	0.0118	36.14	175.0	4.84	4.84	446.1	3.38	0.463	168.25	0.20	0.0024	0.123
C	0.0313	28.94	427.0	11.19	11.19	446.1	3.38	0.463	129.04	0.20	0.0063	0.249
D	0.0103	7.66	206.6	7.89	7.89	446.1	3.38	0.463	145.33	0.15	0.0016	0.070
E	0.0148	5.40	183.4	7.25	7.25	446.1	3.38	0.463	149.35	0.95	0.0141	0.649
F	0.0058	6.51	61.5	2.57	2.57	446.1	3.38	0.463	195.35	0.95	0.0055	0.331
G	0.0042	7.82	80.6	3.35	3.35	446.1	3.38	0.463	184.47	0.95	0.0040	0.227
H	0.0272	4.61	216.7	8.32	8.32	446.1	3.38	0.463	142.86	0.39	0.0106	0.467
											Total (General Scenario)	**3.127**

Note:

1) Runoff is calculated in accordance with DSD *Stormwater Drainage Manual Planning, Design and Management Fifth Edition, January 2018, Stormwater Drainage Manual Corrigendum No.1/2022, and Stormwater Drainage Manual Corrigendum No.1/2024.*
2) The inlet time of Catchment A after the proposed development is arbitratily taken as 5 minutes in reference to rainfall intensity estimation approach in Highway Department *Guidance Notes on Road Pavement Drainage Design* .

Calculation of Runoff for Return Period of 10 Years

Catchment ID	Catchment Area (A), km^2	Average slope (H), m/100m	Flow path length (L), m	Inlet time (t_0), min	Duration (t_d), min	Storm Constants			Runoff intensity (i) mm/hr	Runoff coefficient (C)	C x A	Peak runoff (Q_p), m^3/s
						a	b	c				
Before the Proposed Development												
A (Site)	0.0216	4.77	234.9	9.16	9.16	485.0	3.11	0.397	179.23	0.15	0.0032	0.179
B	0.0118	36.14	175.0	4.84	4.84	485.0	3.11	0.397	213.00	0.20	0.0024	0.155
C	0.0313	28.94	427.0	11.19	11.19	485.0	3.11	0.397	168.67	0.20	0.0063	0.326
D	0.0103	7.66	206.6	7.89	7.89	485.0	3.11	0.397	187.19	0.15	0.0016	0.090
E	0.0148	5.40	183.4	7.25	7.25	485.0	3.11	0.397	191.73	0.95	0.0141	0.833
F	0.0058	6.51	61.5	2.57	2.57	485.0	3.11	0.397	243.36	0.95	0.0055	0.413
G	0.0042	7.82	80.6	3.35	3.35	485.0	3.11	0.397	231.19	0.95	0.0040	0.284
H	0.0272	4.61	216.7	8.32	8.32	485.0	3.11	0.397	184.39	0.39	0.0106	0.603
											Total (General Scenario)	**2.883**

After the Proposed Development												
A (Site)	0.0216	-	-	5.00	5.00	485.0	3.11	0.397	211.28	0.91	0.0196	1.282
B	0.0118	36.14	175.0	4.84	4.84	485.0	3.11	0.397	213.00	0.20	0.0024	0.155
C	0.0313	28.94	427.0	11.19	11.19	485.0	3.11	0.397	168.67	0.20	0.0063	0.326
D	0.0103	7.66	206.6	7.89	7.89	485.0	3.11	0.397	187.19	0.15	0.0016	0.090
E	0.0148	5.40	183.4	7.25	7.25	485.0	3.11	0.397	191.73	0.95	0.0141	0.833
F	0.0058	6.51	61.5	2.57	2.57	485.0	3.11	0.397	243.36	0.95	0.0055	0.413
G	0.0042	7.82	80.6	3.35	3.35	485.0	3.11	0.397	231.19	0.95	0.0040	0.284
H	0.0272	4.61	216.7	8.32	8.32	485.0	3.11	0.397	184.39	0.39	0.0106	0.603
											Total (General Scenario)	**3.986**

Note:

1) Runoff is calculated in accordance with DSD *Stormwater Drainage Manual Planning, Design and Management Fifth Edition, January 2018, Stormwater Drainage Manual Corrigendum No.1/2022, and Stormwater Drainage Manual Corrigendum No.1/2024.*
2) The inlet time of Catchment A after the proposed development is arbitratily taken as 5 minutes in reference to rainfall intensity estimation approach in Highway Department *Guidance Notes on Road Pavement Drainage Design* .

Calculation of Runoff for Return Period of 50 Years

Catchment ID	Catchment Area (A), km^2	Average slope (H), m/100m	Flow path length (L), m	Inlet time (t_0), min	Duration (t_d), min	Storm Constants a	Storm Constants b	Storm Constants c	Runoff intensity (i) mm/hr	Runoff coefficient (C)	C x A	Peak runoff (Q_p), m^3/s
Before the Proposed Development												
Catchment A (Site)	0.0216	4.77	234.9	9.16	9.16	505.5	3.29	0.355	206.49	0.15	0.0032	0.206
Catchment B	0.0118	36.14	175.0	4.84	4.84	505.5	3.29	0.355	240.28	0.20	0.0024	0.175
Catchment C	0.0313	28.94	427.0	11.19	11.19	505.5	3.29	0.355	195.71	0.20	0.0063	0.378
Catchment D	0.0103	7.66	206.6	7.89	7.89	505.5	3.29	0.355	214.53	0.15	0.0016	0.103
Catchment E	0.0148	5.40	183.4	7.25	7.25	505.5	3.29	0.355	219.11	0.95	0.0141	0.952
Catchment F	0.0058	6.51	61.5	2.57	2.57	505.5	3.29	0.355	269.84	0.95	0.0055	0.458
Catchment G	0.0042	7.82	80.6	3.35	3.35	505.5	3.29	0.355	258.08	0.95	0.0040	0.317
Catchment H	0.0272	4.61	216.7	8.32	8.32	505.5	3.29	0.355	211.71	0.39	0.0106	0.692
											Total (General Scenario)	**3.281**

After the Proposed Development												
Catchment A (Site)	0.0216	-	-	5.00	5.00	505.5	3.29	0.355	238.58	0.91	0.0196	1.447
Catchment B	0.0118	36.14	175.0	4.84	4.84	505.5	3.29	0.355	240.28	0.20	0.0024	0.175
Catchment C	0.0313	28.94	427.0	11.19	11.19	505.5	3.29	0.355	195.71	0.20	0.0063	0.378
Catchment D	0.0103	7.66	206.6	7.89	7.89	505.5	3.29	0.355	214.53	0.15	0.0016	0.103
Catchment E	0.0148	5.40	183.4	7.25	7.25	505.5	3.29	0.355	219.11	0.95	0.0141	0.952
Catchment F	0.0058	6.51	61.5	2.57	2.57	505.5	3.29	0.355	269.84	0.95	0.0055	0.458
Catchment G	0.0042	7.82	80.6	3.35	3.35	505.5	3.29	0.355	258.08	0.95	0.0040	0.317
Catchment H	0.0272	4.61	216.7	8.32	8.32	505.5	3.29	0.355	211.71	0.39	0.0106	0.692
											Total (General Scenario)	**4.522**

Note:
1) Runoff is calculated in accordance with DSD *Stormwater Drainage Manual Planning, Design and Management Fifth Edition, January 2018* and *Stormwater Drainage Manual Corrigendum No.1/2022*
2) The inlet time of Catchment A after the proposed development is arbitratily taken as 5 minutes in reference to rainfall intensity estimation approach in Highway Department *Guidance Notes on Road Pavement Drainage Design* .

Appendix E Standard Details of U-Channel, Catchpit, and Sand Trap

Appendix F Calculation of Proposed Drainage System Capacity

Calculation of Proposed Channel Capacity for Return Period of 50 Years

Drainage Capacity of Internal Drainage System (U-channel)

Channel	Upstream Catchpit	Downstream Catchpit	Upstream Ground Level (mPD)	Upstream Invert Level (mPD)	Downstream Ground Level (mPD)	Downstream Invert Level (mPD)	Shape	Depth (m)	Width	Diameter (m)	gradient (1 over)	Length (m)	s	A_w	P_w	R	n	V	Q_C	Involved Catchment	Q_p (m^3/s)	% of capacity	Remark
Northern Channel	CP 09	CP 08	38.55	38.13	39.30	38.61	U-Shape	0.375	Not Applicable	0.375	50	34.4	0.0200	0.1255	0.9640	0.1302	0.016	2.271	0.257	B	0.140	55%	OK
	CP 08	CP 07	39.30	38.61	38.84	38.27	U-Shape	0.375		0.375	50	28.3	0.0200	0.1255	0.9640	0.1302	0.016	2.271	0.257	B	0.140	55%	OK
	CP 07	CP 06	38.84	38.27	36.55	36.03	U-Shape	0.375		0.375	50	25.8	0.0200	0.1255	0.9640	0.1302	0.016	2.271	0.257	B	0.140	55%	OK
	CP 06	CP 05	36.55	36.03	35.55	34.81	U-Shape	0.525		0.525	50	37.1	0.0200	0.2461	1.3497	0.1823	0.016	2.842	0.629	A1,B	0.365	58%	OK
	CP 05	CP 04	35.55	34.81	34.55	33.97	U-Shape	0.525		0.525	50	28.9	0.0200	0.2461	1.3497	0.1823	0.016	2.842	0.629	A1,B	0.365	58%	OK
	CP 04	CP 03	34.55	33.97	34.35	33.68	U-Shape	0.7		0.525	50	33.7	0.0200	0.2920	1.5247	0.1915	0.016	2.937	0.772	A1, A2, B	0.627	81%	OK
	CP 03	CP 02	34.35	33.68	33.05	32.39	U-Shape	0.7		0.525	50	33.1	0.0200	0.2920	1.5247	0.1915	0.016	2.937	0.772	A1, A2, B	0.627	81%	OK
	CP 02	CP 01	33.05	32.39	31.55	30.64	U-Shape	0.7		0.525	50	45.7	0.0200	0.2920	1.5247	0.1915	0.016	2.937	0.772	A1, A2, B	0.627	81%	OK
Southern Channel	CP 10	Point 1	35.05	34.16	33.05	32.67	rectangular	0.375	0.375	Not Applicable	50	74.5	0.0200	0.1406	1.1250	0.1250	0.016	2.210	0.280	D	0.103	37%	OK
	Point 1	CP11	33.05	31.47	32.50	31.33	rectangular	0.375	0.375		50	7.2	0.0200	0.1406	1.1250	0.1250	0.016	2.210	0.280	D	0.103	37%	OK
	CP11	CP12	32.50	30.33	31.05	29.57	rectangular	0.5	0.5		50	38.1	0.0200	0.2500	1.5000	0.1667	0.016	2.677	0.602	A4,D	0.308	51%	OK
	CP12	Tank	31.05	29.57	29.05	28.12	rectangular	0.5	0.5		50	72.5	0.0200	0.2500	1.5000	0.1667	0.016	2.677	0.602	A4,D	0.308	51%	OK
	CP15	CP14	34.05	32.40	32.50	31.98	rectangular	0.4	0.3		50	20.9	0.0200	0.1200	1.1000	0.1091	0.016	2.018	0.218	A3	0.110	50%	OK
	CP14	CP13	32.50	31.98	31.50	31.33	rectangular	0.4	0.3		50	32.6	0.0200	0.1200	1.1000	0.1091	0.016	2.018	0.218	A3	0.110	50%	OK
	CP13	Tank	31.50	29.33	29.05	28.93	rectangular	0.4	0.3		50	20.0	0.0200	0.1200	1.1000	0.1091	0.016	2.018	0.218	A3	0.110	50%	OK
Central Channel	CP 09	CP 10	38.55	36.40	35.05	34.16	rectangular	0.6	0.525		50	111.8	0.0200	0.3150	1.7250	0.1826	0.016	2.845	0.807	A1,B,C	0.469	58%	OK
	CP10	CP19	35.05	32.16	32.85	31.34	circular pipe	Not Applicable		0.7	50	41.0	0.0200	0.3848	2.1991	0.1750	0.016	2.765	0.958	A1,B,C	0.469	49%	OK
	CP19	Point 2	32.85	31.34	32.75	30.99	rectangular	0.9	0.8	Not Applicable	150	53.4	0.0067	0.7200	2.6000	0.2769	0.016	2.168	1.405	A1,A2,A3,B,C	1.060	75%	OK
	Point 2	CP18	32.75	30.99	31.52	30.84	rectangular	0.9	0.8		150	21.6	0.0067	0.7200	2.6000	0.2769	0.016	2.168	1.405	A1,A2,A3,B,C	1.060	75%	OK
	CP18	CP17	31.52	30.84	31.55	30.70	rectangular	0.9	0.8		150	21.0	0.0067	0.7200	2.6000	0.2769	0.016	2.168	1.405	A1,A2,A3,B,C	1.060	75%	OK
	CP17	CP16	31.55	30.20	31.05	30.16	rectangular	0.9	0.8		150	6.6	0.0067	0.7200	2.6000	0.2769	0.016	2.168	1.405	A1,A2,A3,B,C	1.060	75%	OK

Remark: In reference to *Stormwater Drainage Manual* Table 13, the manning's roughness coefficient 0.016 is taken as concrete line surface under fair condition, the effect of sedimentation is considered through deducting flow capacity by 10%.

Legend

D = diameter, m

A_w = Cross Section Area of Flow, m^2

P_w = Wetted Perimeter, m

R = Hydraulic Radius = A_w/P_w, m

s = Hydraulic Gradient

n = Manning's roughness coefficient

V = Mean Velocity, m/s

Q_c = Flow Capacity, m^3/s

Q_p = Estimated Peak Flow, m^3/s

Calculation of Proposed Pipe Capacity for Return Period of 50 Years

Drainage Capacity of Proposed External Drainage System

From	To	Upstream Invert Level	Downstream Invert Level	Description	length	d	r	Aw	Pw	R	s	ks	V	Q_c	Q_p	% of capacity	Remark
		mPD	mPD		m	m	m	m^2	m	m	-	mm	m/s	m^3/s	m3/s	%	
Catchpit 01	Terminal Manhole	29.02	29	Internal drainage 01	2.96	0.75	0.375	0.442	2.356	0.188	0.005	0.15	2.267	0.9019	0.6268	70%	OK
Catchpit 16	Terminal Manhole	29.07	29.02	Internal drainage 02	9.50	1.05	0.525	0.866	3.299	0.263	0.005	0.15	2.784	2.1695	1.0597	49%	OK
Terminal Manhole	RMH-X1	29	28.68	external drainage 01	1.36	1.5	0.75	1.767	4.712	0.375	0.005	0.15	3.455	5.494	2.104	38%	OK
RMH-X1	RMH-X2	28.68	28.55	external drainage 02	24.82	1.5	0.75	1.767	4.712	0.375	0.005	0.15	3.455	5.494	3.056	56%	OK
RMH-X2	RMH-X3	28.55	28.37	external drainage 03	34.09	1.5	0.75	1.767	4.712	0.375	0.005	0.15	3.455	5.494	3.056	56%	OK
RMH-X3	RMH-X4	28.37	28.13	external drainage 04	47.65	1.5	0.75	1.767	4.712	0.375	0.005	0.15	3.455	5.494	3.056	56%	OK
RMH-X4	Discharge Point	28.13	27.07	external drainage 05	6.41	1.5	0.75	1.767	4.712	0.375	0.005	0.15	3.455	5.494	3.056	56%	OK

Remark: In reference to *Stormwater Drainage Manual* Table 14, the surface roughness value is taken as precast concrete pipe with 'O' ring joints under normal condition, the effect of sedimentation is considered by deducting flow capacity by 10%.

Legend

d = pipe diameter, m
r = pipe radius (m) = 0.5d
A_w = wetted area (m^2) = p r^2
P_w = wetted perimeter (m) = 2pr
R = Hydraulic radius (m) = A_w/P_w

s = Slope of the total energy line
k_s = equivalent sand roughness, mm
V = Velocity of flow calculated based on Colebrook White Equation, m/s
Q_c = Flow Capacity (10% sedimentation incorporated), m^3/s
Q_p = Estimated total peak flow from the pipe

Tank Sizing for Stormwater Storage Tank

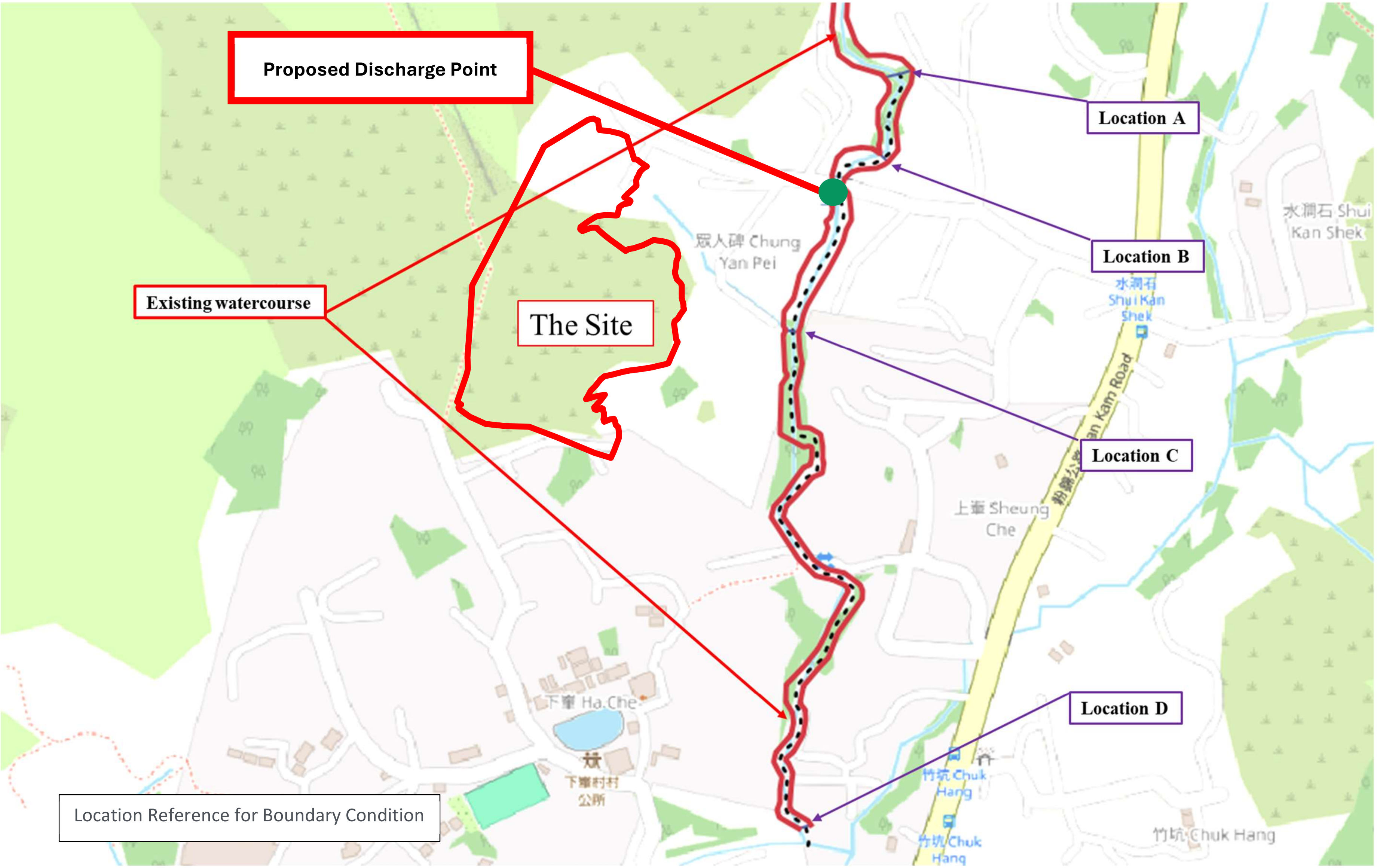
Description	Catchment Area (A), km^2	Runoff intensity (i), mm/hr [2]	Runoff coefficient (C)	C x A	Peak runoff (Q_p), m^3/s	Duration of Storm, hours	Runoff Volume, m^3	Length (L)	Width (W)	Depth (D)	Design Volume
From CP10 to Tank (Catchment D)	**0.01034**	134.00	0.15	0.0016	0.058	1.0	233				
From CP10 to Tank (Catchment A4)	**0.00305**	134.00	0.91	0.0028	0.103	1.0	411				
From CP15-13 (25% Catchment A3)	**0.00653**	134.00	0.91	0.0059	0.055	1.0	222				
Total runoff volume needs to be stored							**867**	20.0	15.0	3.0	900

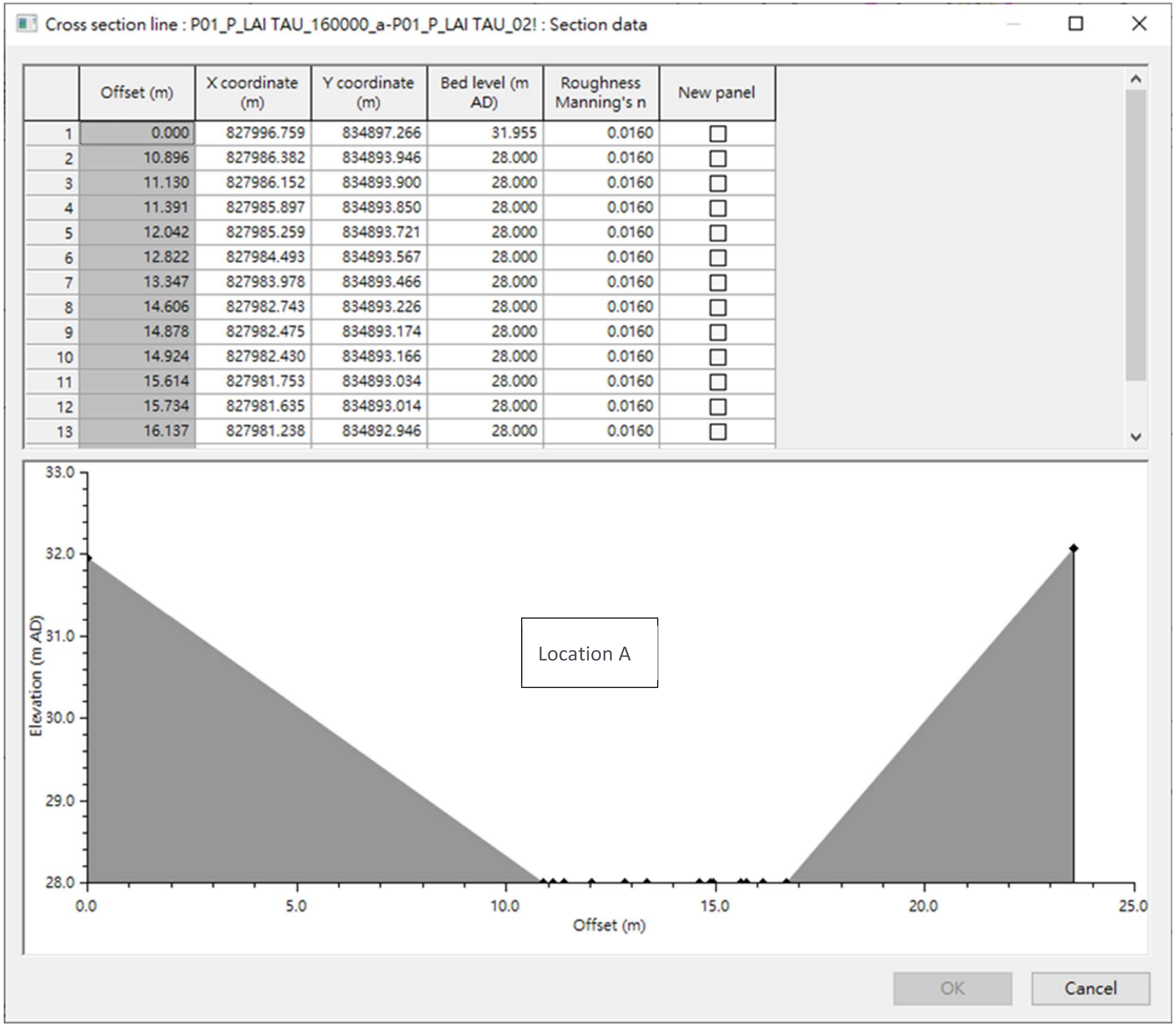
Note:

1) Runoff is calculated in accordance with DSD's "*Stormwater Drainage Manual (with Eurocodes incorporated) - Planning, Design and Managemen* t" (SDM), fifth edition, January 2018. .

2) Extreme intensity under 50 years return period is based on Table 3a of Corrigendum 1/2024.

Appendix G Information of Existing Watercourse


Proposed Discharge Point
Location A
Location B
Existing watercourse
The Site
Location C
Location D
冠人碑 Chung Yan Pei
水澗石 Shui Kan Shek
上輋 Sheung Che
下輋 Ha Che
竹坑 Chuk Hang
Location Reference for Boundary Condition

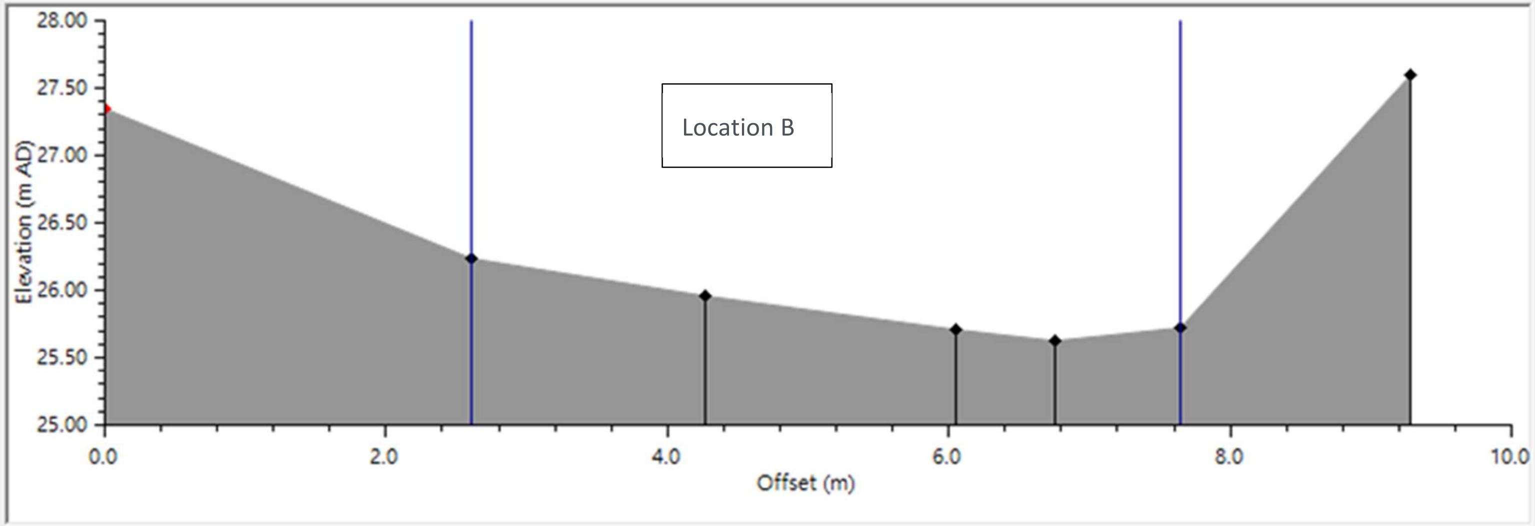


Cross section line : P01_P_LAI TAU_160000_a-P01_P_LAI TAU_02! : Section data

	Offset (m)	X coordinate (m)	Y coordinate (m)	Bed level (m AD)	Roughness Manning's n	New panel
1	0.000	827996.759	834897.266	31.955	0.0160	☐
2	10.896	827986.382	834893.946	28.000	0.0160	☐
3	11.130	827986.152	834893.900	28.000	0.0160	☐
4	11.391	827985.897	834893.850	28.000	0.0160	☐
5	12.042	827985.259	834893.721	28.000	0.0160	☐
6	12.822	827984.493	834893.567	28.000	0.0160	☐
7	13.347	827983.978	834893.466	28.000	0.0160	☐
8	14.606	827982.743	834893.226	28.000	0.0160	☐
9	14.878	827982.475	834893.174	28.000	0.0160	☐
10	14.924	827982.430	834893.166	28.000	0.0160	☐
11	15.614	827981.753	834893.034	28.000	0.0160	☐
12	15.734	827981.635	834893.014	28.000	0.0160	☐
13	16.137	827981.238	834892.946	28.000	0.0160	☐

Cross section line : P01_P_LAI TAU_160530_a : Section data

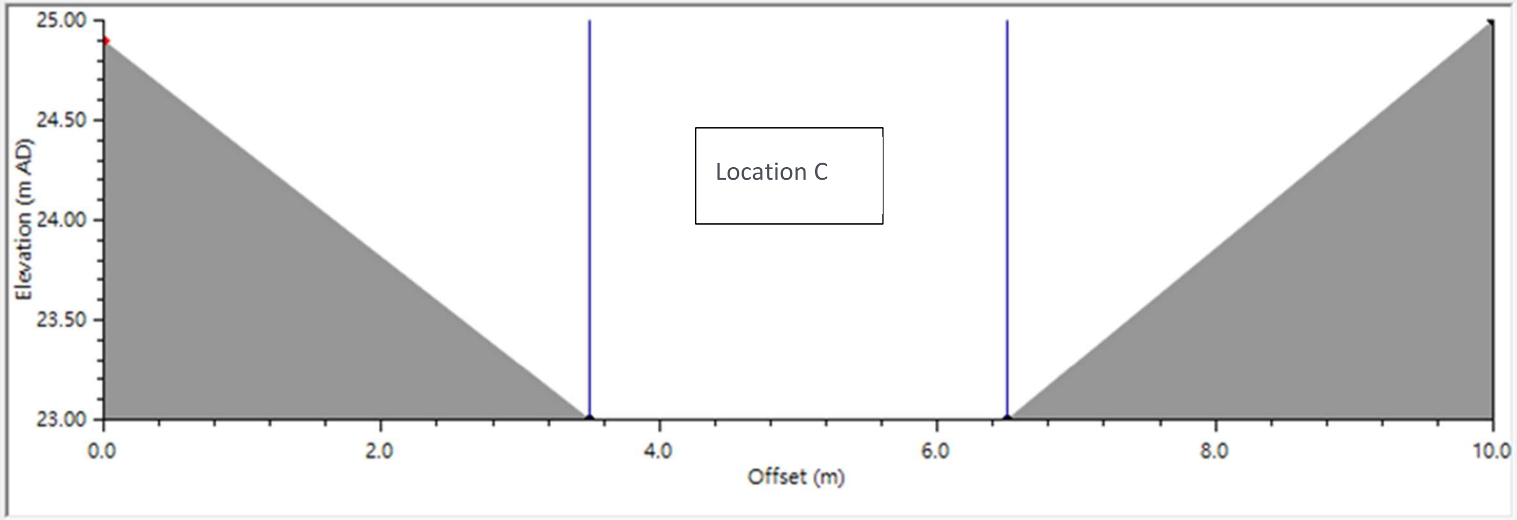
	Offset (m)	X coordinate (m)	Y coordinate (m)	Bed level (m AD)	Roughness Manning's n	New panel
1	0.000	827976.997	834823.522	27.350	0.0160	☐
2	2.600	827975.173	834825.375	26.240	0.0160	☒
3	4.270	827974.002	834826.565	25.960	0.0160	☐
4	6.050	827972.753	834827.834	25.710	0.0160	☐
5	6.750	827972.262	834828.333	25.620	0.0160	☐
6	7.640	827971.638	834828.967	25.720	0.0160	☒
7	9.280	827970.488	834830.136	27.600	0.0160	☐



OK
Cancel

Cross section line : P01_P_LAI TAU_160900 : Section data

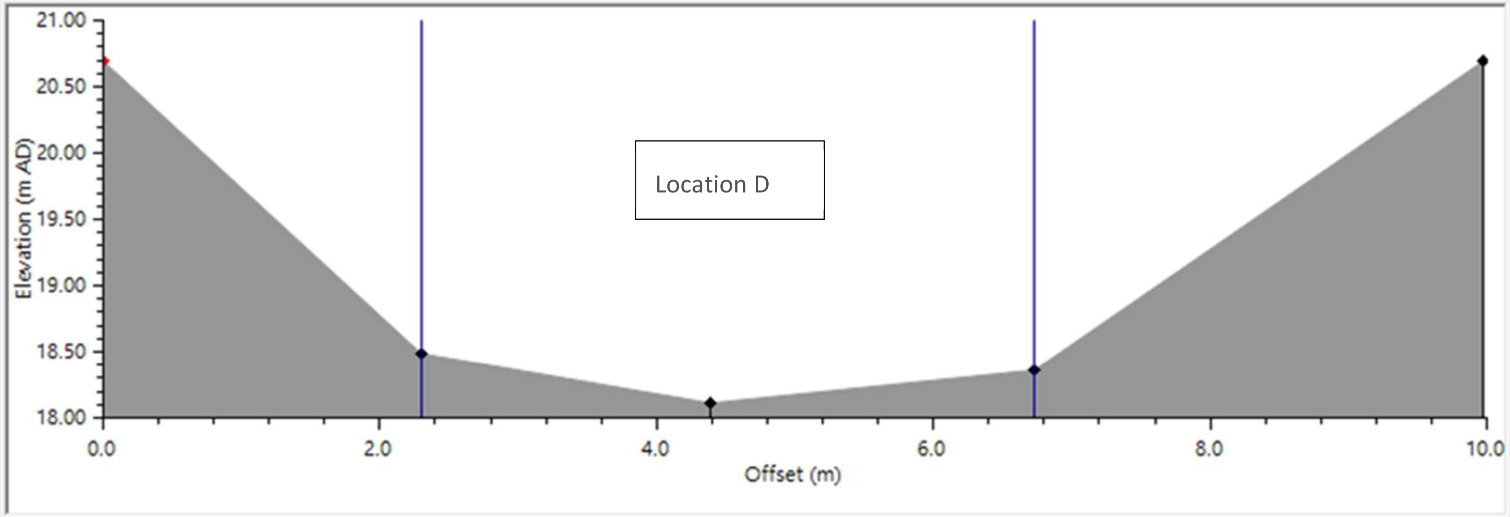
	Offset (m)	X coordinate (m)	Y coordinate (m)	Bed level (m AD)	Roughness Manning's n	New panel
1	0.000	827906.112	834687.532	24.900	0.0160	☐
2	3.500	827902.649	834688.040	23.000	0.0160	☒
3	6.500	827899.681	834688.475	23.000	0.0160	☒
4	10.000	827896.218	834688.982	25.000	0.0160	☐



OK Cancel

Cross section line : P01_P_LAI TAU_161180_a : Section data

	Offset (m)	X coordinate (m)	Y coordinate (m)	Bed level (m AD)	Roughness Manning's n	New panel
1	0.000	827915.891	834292.642	20.700	0.0160	☐
2	2.305	827913.697	834291.934	18.480	0.0160	☒
3	4.386	827911.702	834291.343	18.110	0.0160	☐
4	6.733	827909.453	834290.669	18.360	0.0160	☒
5	9.972	827906.380	834289.645	20.700	0.0160	☐



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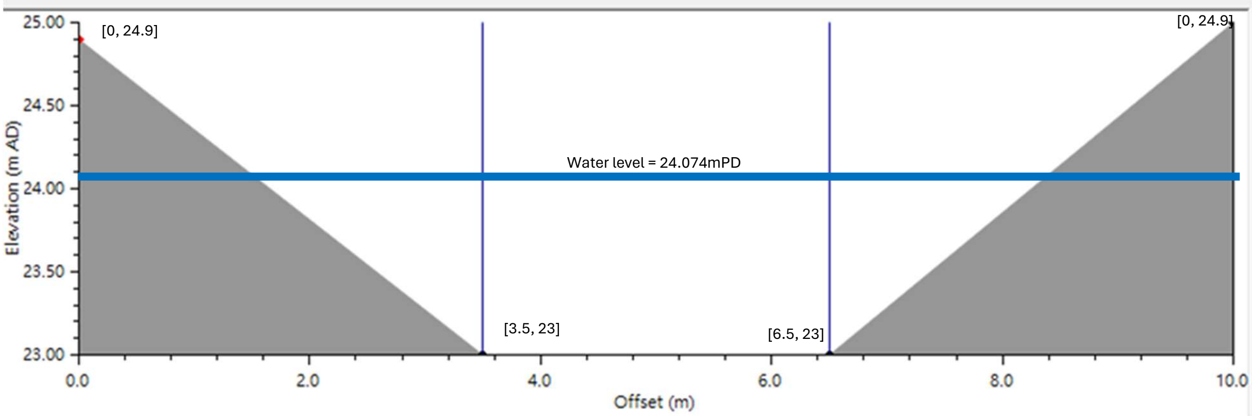
Boundary Condition at each Section of Watercourse

		Return Period														
Location	Section ID	2AB			10A			10B			50A			50B		
		Peak Water Level (mPD)	Peak Flow (m3/s)	Peak Velocity (m/s)	Peak Water Level (mPD)	Peak Flow (m3/s)	Peak Velocity (m/s)	Peak Water Level (mPD)	Peak Flow (m3/s)	Peak Velocity (m/s)	Peak Water Level (mPD)	Peak Flow (m3/s)	Peak Velocity (m/s)	Peak Water Level (mPD)	Peak Flow (m3/s)	Peak Velocity (m/s)
A	P01_P_LAI TAU_160000_a-P01_P_LAI TAU_02!	28.41300	14.80727	5.25000	28.53400	22.90300	6.06600	28.41300	14.80724	5.25000	28.59600	28.07553	6.54000	28.53400	22.90267	6.06600
B	P01_P_LAI TAU_160530_a	26.41500	14.80655	4.86400	26.57800	21.15100	5.68200	26.41500	14.80648	4.86400	26.67300	28.82516	6.12100	26.57800	23.15104	5.68200
C	P01_P_LAI TAU_160900	23.75600	14.80303	4.42800	23.95900	23.14600	5.05500	23.75600	14.80309	4.42800	24.07400	28.84762	5.39900	23.95900	23.14692	5.05500
D	P01_P_LAI TAU_161180_a	19.53400	14.84949	2.11800	19.96600	23.21900	2.39500	19.53400	14.85032	2.11800	20.23200	28.98928	2.54300	19.96600	23.21988	2.39500

Appendix H Calculation of Watercourse Capacity

Calculation of Flow Capacity of Watercourse at Location C

Referring to the information provided by DSD, the peak water level at Location C has reached 24.074mPD under the scenario of a 10-year sea level in conjunction with a 50-year rainfall. The flow area as well as the hydraulic radius at the peak water level can be calculated from the geometry of the cross section of the watercourse at Location C:



As shown, the geometry of the watercourse cross-section at Location C can be portrayed by its bed level and the corresponding offset from shore line. Listed in the format as [offset, bed level], the 4 points used for describing cross-section geometry are:

[0, 24.9]; [3.5, 23]; [6.5, 23]; [10, 25].

Therefore,

$$flow\ area = (\frac{24.074-23.0}{24.9-23.0})^2 \times (24.9-23.0) \times (3.5-0) \times 0.5 + (24.074-23.0) \times (6.5-3.5) + \left(\frac{24.074-23.0}{25.0-23.0}\right)^2 \times (25.0-23.0) \times (10.0-6.5) \times 0.5 = 5.29370(m^2)$$

$$wet\ perimeter = \frac{24.074-23.0}{24.9-23.0} \times \sqrt[2]{(24.9-23.0)^2+(3.5-0)^2} + (6.5-3.5) + \frac{24.074-23.0}{25.0-23.0} \times \sqrt[2]{(25.0-23.0)^2+(10.0-6.5)^2} = 7.41585\ (m)$$

$$hydraulic\ radius = \frac{flow\ area}{wet\ perimeter} = \frac{5.29370}{7.41585} = 0.71384\ (m)$$

Under assumption of uniform flow condition, the flow velocity of an open channel will subject to its roughness, channel gradient, and hydraulic radius as express as Manning's equation:

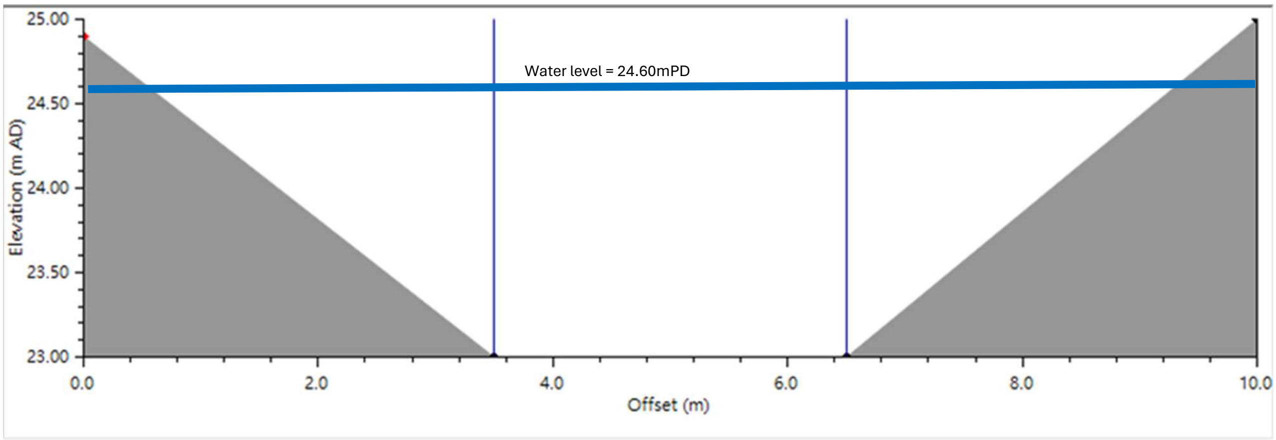
$$flow\ velocity = \frac{channel\ gradient^{\frac{1}{2}}}{Manning\ coefficient} \times hydraulic\ radius^{\frac{2}{3}}$$

The peak velocity of watercourse at Location C corresponding to the water level of 24.074m has been given by DSD as 5.399m/s. The hydraulic property of watercourse at Location C can be back calculated as a constant.

$$5.399 = \frac{channel\ gradient^{\frac{1}{2}}}{Manning\ coefficient} \times 0.71384^{\frac{2}{3}}$$

$$\frac{channel\ gradient^{\frac{1}{2}}}{Manning\ coefficient} = \frac{5.399}{0.71384^{\frac{2}{3}}} = 6.75947\ (m^{\frac{1}{3}}/s)$$

When a 300mm freeboard is reserved, the water level at Location C will reach 24.6mPD.



The corresponding flow area and hydraulic radius can be calculated.

$$flow\ area = \left(\frac{24.60-23.0}{24.9-23.0}\right)^2 \times (24.9-23.0)\times(3.5-0)\times 0.5 + (24.60-23.0)\times(6.5-3.5) + \left(\frac{24.60-23.0}{25.0-23.0}\right)^2 \times (25.0-23.0)\times(10.0-6.5)\times 0.5 = 9.39790\ (m^2)$$

$$wet\ perimeter = \frac{24.60-23.0}{24.9-23.0} \times \sqrt[2]{(24.9-23.0)^2+(3.5-0)^2} + (6.5-3.5) + \frac{24.60-23.0}{25.0-23.0} \times \sqrt[2]{(25.0-23.0)^2+(10.0-6.5)^2} = 9.57855\ (m)$$

$$hydraulic\ radius = \frac{flow\ area}{wet\ perimeter} = \frac{9.39790}{9.57855} = 0.98114\ (m)$$

With the hydraulic radius and flow area known, the capacity of watercourse at Location C when a 300mm freeboard is reserved can be estimated as below:

$$flow\ velocity = \frac{channel\ gradient^{\frac{1}{2}}}{Manning\ coefficient} \times hydraulic\ radius^{\frac{2}{3}} = 6.75947 \times 0.98114^{\frac{2}{3}} = 6.67421\ (m/s)$$

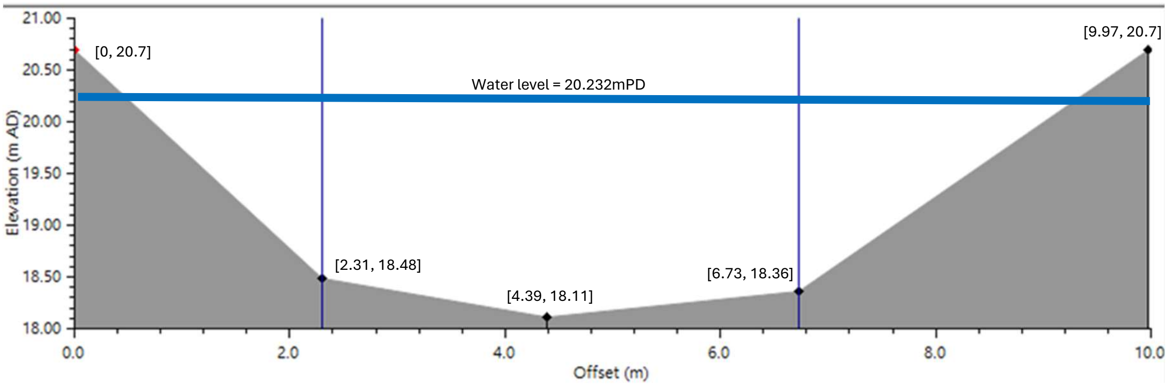
$$flow\ capacity = flow\ area \times flow\ velocity = 9.39790 \times 6.67421 = 62.72357\ (m^3/s)$$

Comparing the peak flow under current boundary condition, 28.84762m^3/s, and the flow capacity under 300mm freeboard, the available capacity of watercourse at Location C can be estimated:

$$available\ capacity = 62.72357 - 28.84762 = 33.87595\ (m^3/s)$$

Calculation of Watercourse Capacity at Location D

Referring to the information provided by DSD, the peak water level at Location D has reached 20.232mPD under the scenario of a 10-year sea level in conjunction with a 50-year rainfall. The flow area as well as the hydraulic radius at the peak water level can be calculated from the geometry of the cross section of the watercourse at Location D:



As shown, the geometry of the watercourse cross-section at Location D can be portrayed by its bed level and the corresponding offset from shore line. Listed in the format as [offset, bed level], the 5 points used for describing cross-section geometry are:

[0, 20.7]; [2.31, 18.48]; [4.39, 18.11]; [6.73, 18.36]; [9.97, 20.7].

Therefore,

$$flow\ area = (\frac{20.232-18.48}{20.70-18.48})^2 \times (20.70-18.48)\times(2.31-0)\times 0.5 + (20.232-18.48)\times(4.39-2.31) + (18.48-18.11)\times(4.39-2.31)\times 0.5 + (20.232-18.36)\times(6.73-4.39)\times 0.5 + (18.36-18.11)\times(6.73-4.39)\times 0.5 + \left(\frac{20.232-18.36}{20.70-18.36}\right)^2 \times (20.70-18.36)\times(9.97-6.73)\times 0.5 = 12.73673(m^2)$$

$$wet\ perimeter = \frac{20.232-18.48}{20.70-18.48}\times\sqrt[2]{(20.70-18.48)^2+(2.31-0)^2} + \sqrt[2]{(18.48-18.11)^2+(4.39-2.31)^2} + \sqrt[2]{(18.36-18.11)^2+(6.73-4.39)^2} + \frac{20.232-18.36}{20.70-18.36}\times\sqrt[2]{(20.70-18.36)^2+(9.97-6.73)^2} = 10.19617\ (m)$$

$$hydraulic\ radius = \frac{flow\ area}{wet\ perimeter} = \frac{12.73673}{10.19617} = 1.24917\ (m)$$

Under assumption of uniform flow condition, the flow velocity of an open channel will subject to its roughness, channel gradient, and hydraulic radius as express as Manning's equation:

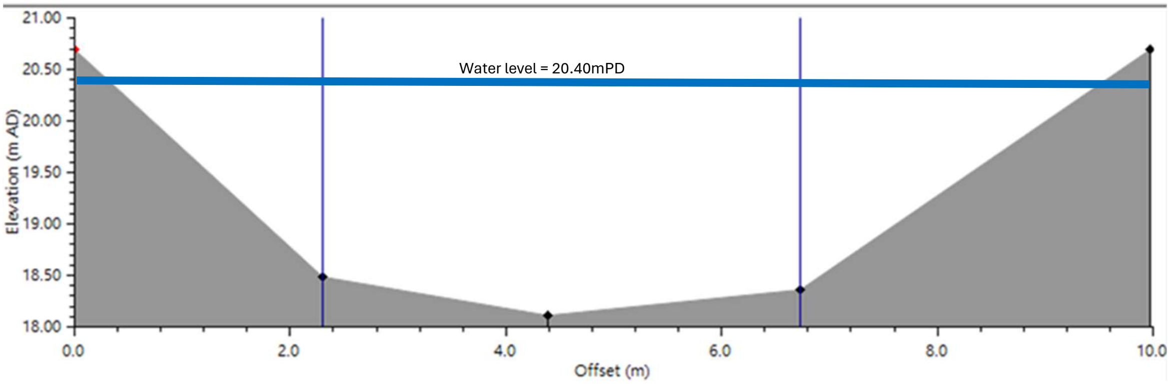
$$flow\ velocity = \frac{channel\ gradient^{\frac{1}{2}}}{Manning\ coefficient} \times hydraulic\ radius^{\frac{2}{3}}$$

The peak velocity of watercourse at Location D corresponding to the water level of 20.232m has been given by DSD as 2.543m/s. The hydraulic property of watercourse at Location D can be back calculated as a constant.

$$2.543 = \frac{channel\ gradient^{\frac{1}{2}}}{Manning\ coefficient} \times 1.24917^{\frac{2}{3}}$$

$$\frac{channel\ gradient^{\frac{1}{2}}}{surface\ roughness} = \frac{2.543}{1.24917^{\frac{2}{3}}} = 2.19246\ (m^{\frac{1}{3}}/s)$$

When a 300mm freeboard is reserved, the water level at Location D will reach 20.40mPD.



The corresponding flow area and hydraulic radius can be calculated.

$$flow\ area = (\frac{20.40-18.48}{20.70-18.48})^2 \times (20.70-18.48)\times(2.31-0)\times 0.5 + (20.40-18.48)\times(4.39-2.31) + (18.48-18.11)\times(4.39-2.31)\times 0.5 + (20.40-18.36)\times(6.73-4.39)\times 0.5 + (18.36-18.11)\times(6.73-4.39)\times 0.5 + \left(\frac{20.40-18.36}{20.70-18.36}\right)^2 \times (20.70-18.36)\times(9.97-6.73)\times 0.5 = 14.25575\ (m^2)$$

$$wet\ perimeter = \frac{20.40-18.48}{20.70-18.48}\times\sqrt[2]{(20.40-18.48)^2+(2.31-0)^2} + \sqrt[2]{(18.48-18.11)^2+(4.39-2.31)^2} + \sqrt[2]{(18.36-18.11)^2+(6.73-4.39)^2} + \frac{20.40-18.36}{20.70-18.36}\times\sqrt[2]{(20.70-18.36)^2+(9.97-6.73)^2} = 10.72523\ (m)$$

$$hydraulic\ radius = \frac{flow\ area}{wet\ perimeter} = \frac{14.25575}{10.72523} = 1.32918\ (m)$$

With the hydraulic radius and flow area known, the capacity of watercourse at Location D when a 300mm freeboard is reserved can be estimated as below:

$$flow\ velocity = \frac{channel\ gradient^{\frac{1}{2}}}{Manning\ coefficient} \times hydraulic\ radius^{\frac{2}{3}} = 2.19246 \times 1.32918^{\frac{2}{3}} = 2.65046\ (m/s)$$
$$flow\ capacity = flow\ area \times flow\ velocity = 14.25575 \times 2.65046 = 37.78431\ (m^3/s)$$

In summary, the flow capacity of watercourse at Location D with 300mm freeboard is estimated to be 37.78m^3/s.

Comparing the peak flow under current boundary condition, 28.98928m^3/s, and the flow capacity under 300mm freeboard, the available capacity of watercourse at Location D can be estimated:

$$available\ capacity = 37.78431 - 28.98928 = 8.79503\ (m^3/s)$$

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