Appendix V

Drainage Proposal

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Planning Application for

Proposed Temporary Concrete Batching Plant in Ping Che, New Territories
Drainage Proposal

Prepared for:

Doran (Hong Kong) Ltd

February 2025

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For and on behalf of
EnviroSolutions & Consulting

Nigel LANDON
Director

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1 PROJECT BACKGROUND

1.1 Introduction

1.1.1 A temporary Concrete Batching Plant ("CBP" or "the Proposed Development") is planned to be erected at part of Lot 153 in D.D. 77 ("the Site") zoned Industrial (Group D) ("I(D)") under the Approved Ping Che and Ta Kwu Ling Outline Zoning Plan ("OZP") No. S/NE-TKL/14.

1.1.2 A Planning Application (Application No.: A/NE-TKL/681) was submitted and approved with conditions under Section 16 of the *Town Planning Ordinance* ("TPO")on 10 June 2022. Thereafter, several changes were made to the General Building Plan ("GBP") compared with the layout submitted in the aforementioned Planning Application. Furthermore, total cementitious material silo capacity was also changed from no more than 450 tonnes to no more than 800 tonnes due to amendment to the definition of silo capacity in accordance with the *Air Pollution Control Ordinance* ("APCO"). Therefore, a planning application for the CBP with the latest layout plans shall be submitted under Section 16 of TPO.

1.1.3 EnviroSolutions & Consulting Ltd ("ESC") has been engaged to carry out a Drainage Proposal to support the abovementioned application.

1.2 Site Description

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

- To the North: Village houses and temporary structures with industrial use
- To the East: Open storage, temporary structures with industrial use and vegetation
- To the South: Workshop (i.e. Hong Kong Wall Testing Centre)
- To the West: Open storage and workshops

1.3 Objectives of this Report

1.3.1 The objectives of this Drainage Proposal are to:

- Review the existing drainage condition of the Site and surroundings.
- Assess the potential drainage impacts arising from the Proposed Development.
- Recommend the necessary mitigation measures to alleviate any impacts, if any.

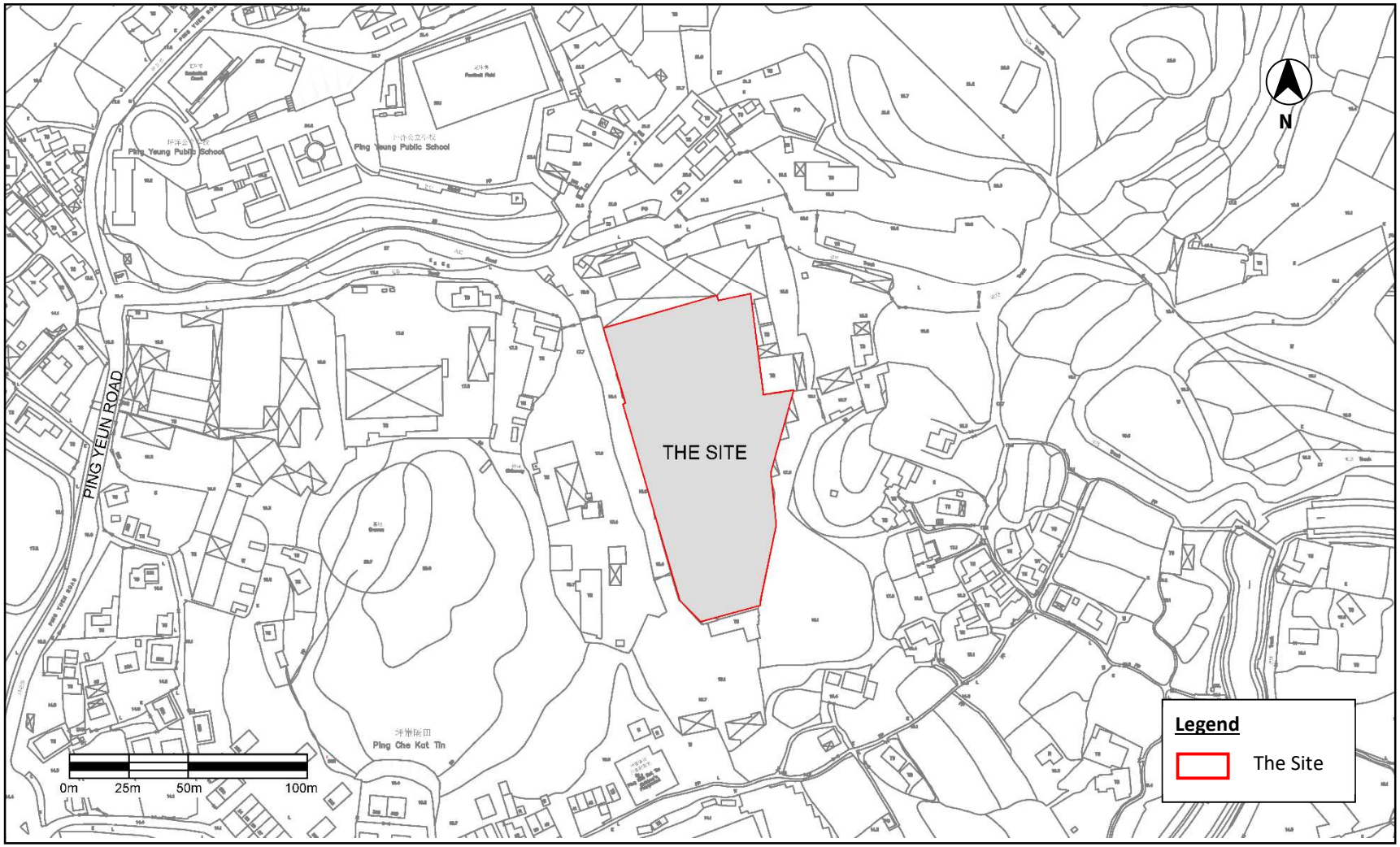
1.4 Reference Materials

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

- Stormwater Drainage Manual (with Eurocodes incorporated) – Planning, Design and Management (Fifth Edition, January 2018)
- Stormwater Drainage Manual – Corrigenda Nos. 1/2022, 1/2024 and 2/2024
- DSD Advice Note No. 1 – Application of the Drainage Impact Assessment Process to Private Sector Projects
- Technical Note to prepare a Drainage Submission
- Drainage Data of GeoInfo Map reviewed on 11 February 2025
- Drainage Proposal dated 20 June 2024 for Application No. A/NE-TKL/681 [Note 1]

[1] The Drainage Proposal dated 20 June 2024 was considered in compliance with Approval Condition (b) of Planning Application No. A/NE-TKL/681 with reference to PlanD's letter dated 8 July 2024.

Figure 1-1 Site Location and its Environs



2 DESCRIPTION OF EXISTING ENVIRONMENT AND DRAINAGE CONDITIONS

2.1 Site Location and Topography

2.1.1 As illustrated in **Figure 1-1**, the Site is surrounded by various temporary structures with industrial use and vegetated slopes to the north and east.

2.1.2 The Site is situated at part of Lots 153 in D.D. 77 in Ping Che, New Territories. The Site area is about 6,957m^2 with elevation ranging from +18.07mPD to +18.43mPD.

2.2 Types of the Site

2.2.1 The Site does not fall within flood prone area. No substantial earth filling was carried out before, as advised by the Applicant. With reference to DSD' Technical Note, the Site is therefore considered as a simple site.

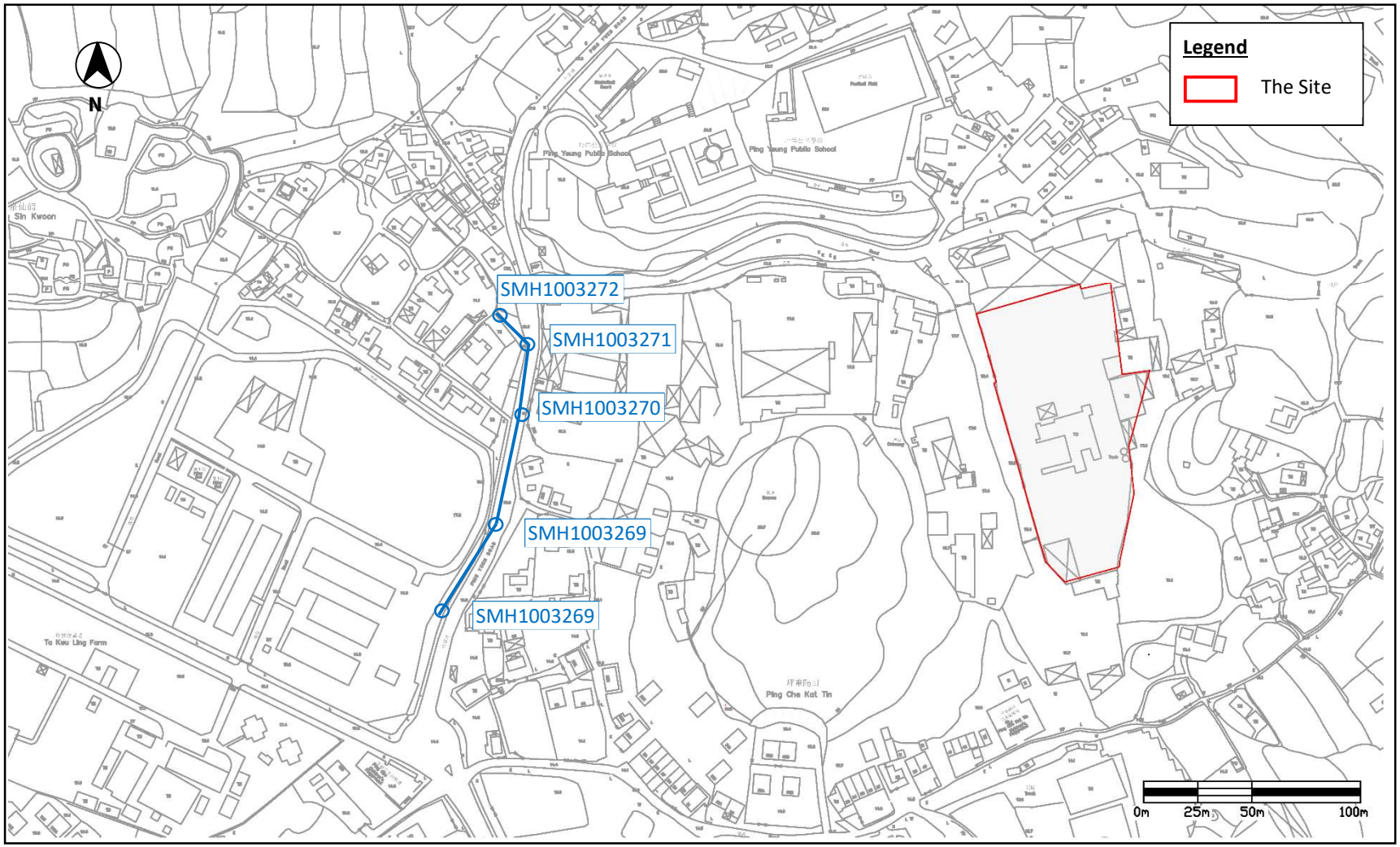
2.3 Existing Baseline Conditions

2.3.1 Based on the site information provided by the Applicant, the ground surface of the Site is entirely paved with concrete including those area to the immediate west of the Site where the existing CBP is located.

2.3.2 With reference to Geoinfo Map, the existing government municipal drainage system is located at approximately 100m to the west of the Site, as shown in **Figure 2-1.** There are some private stormwater drains located at approx. 50m to the northwest of the site and connecting to the existing government municipal drainage system

2.3.3 A survey on the private stormwater drains connecting between the Site and the municipal drainage system was conducted on 20 February 2025. The survey result demonstrated that the runoff from the Site is currently conveyed to the municipal drainage system to the west of the Site as shown in **Figure 2-1**. The survey results are tally with the findings in the Drainage Proposal dated 20 June 2024 listed in **paragraph 1.4.1**. According to Applicant's record, no flooding has occurred before.

Figure 2-1 Existing Drainage Network



3 DRAINAGE ANALYSIS

3.1 Assumptions and Methodology

3.1.1 Peak instantaneous run off before and after the Proposed Development 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("SDM")*.

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

$$Q_p = 0.278\, C\, i\, A \qquad \textbf{--- Equation 1}$$

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:

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

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 SDM

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

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

where t_c = time of correction
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, t_d is the divisor. Therefore, larger t_d will result in smaller rainfall intensity (i) as well as smaller Q_p. For the worst-case scenario, t_f is assumed to be negligible and so:

$$t_c = t_0 = t_f$$

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

where A = catchment area (m^2)
H = average slope (m per 100m), 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 Colebrook-White Equation was adopted for calculation of drainage capacity of pipes. Full bore flow with no surcharge is assumed, as follows, the calculation of drainage capacity in accordance with SDM.

$$V = -\sqrt{32gRs} * \log\left(\frac{ks}{14.8R} + \frac{1.25v}{R\sqrt{32gRs}}\right)$$ ***--- Equation 5***

where V = mean velocity (m/s)
g = gravitational acceleration (m/s^2)
R = hydraulic radius (m)
ks = hydraulic pipeline roughness (m)
v = 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:

$$V = \frac{R^{2/3}}{n} \times \sqrt{Rs}$$ ***--- 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 The proposed peripheral U-Channels is assumed to be reinforced concrete channel with fair condition. Therefore, Manning coefficient n = 0.016 has been adopted for calculations with reference to SDM Table 13. These assumptions were also adopted in the Drainage Proposal dated 20 June 2024 listed in **paragraph 1.4.1**.

3.2 Assessment Assumptions

Project Site

3.2.1 As mentioned in the Drainage Proposal dated 20 June 2024, the Site is currently concrete-paved.

3.2.2 Since the Site is a CBP, the paving condition is assumed to be remain unchanged. With reference to the SDM, the runoff coefficients of paved surface 0.95 has been adopted for the Proposed Development before and after the development summarised in **Table 3-1**.

Table 3-1 Catchment Surface Characteristics and Runoff Coefficient

SCENARIO OF PROJECT	RUNOFF COEFFICIENT	SURFACE CHARACTERISTICS	RUNOFF COEFFICIENT
Before Development	6,957m^2	100% concrete paved	0.95
After Development		100% concrete paved	0.95

Manholes

3.2.3 For the existing storm water manhole on-site and the three new private manholes, it is assumed that the water inlet and outlet would have the same invert level in each manhole.

Identification of Catchments

3.2.4 According to the topographical data on the basemap from the Survey and Mapping Office ("SMO") on 11 February 2025, Catchments A to I have been identified. The site condition and the division of different catchment areas are shown in **Figure 3-2**.

3.2.5 Runoff from the Site (Catchment A2) will be discharged to the existing communal drainage system including underground pipe and channels as shown in **Figure 3-3.**

Cumulative Runoff (Surrounding Catchments)

3.2.6 Based on the topographical data, runoff from Catchment A1 with higher topography shall naturally flow towards Catchment F which has a lower topography and shall not enter the Site. There was manhole in between Catchment A1 and Catchment F but covered (unavailable for survey). As a result, runoff from Catchment A1 has not been included as cumulative runoff to the Site.

3.2.7 Catchment C spans an elevation range from 19.3mPD to 21mPD, with higher elevations located on the western side. A significant portion of the northwestern area is occupied by houses and workshops, while the eastern side is predominantly covered with vegetation. According to applicant's information, a drainage U-channel was identified along the roadside of Catchment C (refer to **Figure 3-1**). It is probable that runoff from Catchment C is collected into this U-channel, which is not linked to the communal village drain and does not flow through the Site. As a result, runoff from Catchment C has been excluded from the cumulative runoff calculations for the development site.

Figure 3-1 U-channel along Catchment C

3.2.8 With reference to the topographical data on survey map, the lowest level of Catchment B is 17.9mPD located at southern side of Catchment B, which is much lower than Catchment A2 and unlikely to enter the Site. Since runoff from Catchment B is unlikely to flow toward the Site, it has been excluded from cumulative runoff to the development Site.

3.2.9 On the other hand, there is no obvious drainage connection between Catchments D, F, G, H and I to the Site. Based on topography, runoff from Catchments D, F and G should be discharged to the channel along Ping Yuen Road. Catchments H and I should be drained to Ping Yuen River along the northeast to southwest direction.

3.2.10 Calculation methods of corresponding catchments are summarised in **Table 3-2** and the photos of relevant channels and watercourse were shown in **Figure 3-3**.

Table 3-2 Method for Estimating the Surface Runoff from the Site

CATCHMENT	ESTIMATING METHOD FOR SURFACE RUNOFF
Catchment A2	Rational Method

3.3 Estimated Existing and Future Runoff

Peak Runoff from the Site

3.3.1 Based on the assumption as described **paragraphs 3.2.1 to 3.2.2** the runoff from the Proposed Development before and after development was estimated based on the return periods of 2 and 10 years.

3.3.2 As shown in **Table 3-3** the estimated peak runoff generated from the Proposed Development is 0.284m^3/s under 10 years return period. There will be no increment in the estimated peak runoff after the proposed development under all assessed return periods. Detailed calculations are provided in **Appendix A.**

Table 3-3 Estimated Peak Runoff of the Proposed Development

RETURN PERIOD	ESTIMATED PEAK RUNOFF (m^3/s)		
	BEFORE DEVELOPMENT	AFTER DEVELOPMENT	INCREMENT
2 Years	0.221	0.219	-0.9%
10 Years	0.286	0.284	-0.9%

Peak Runoff from the Surrounding Catchment

3.3.3 Based on the observation from site visit and topography information, runoff from surrounding catchment into the communal drain is not anticipated.

3.4 Existing Drainage Layout

3.4.1 Since the Proposed Development will not result in an increase in peak runoff under the worst-case scenario across all assessed return periods, no additional drainage impact is expected due to the development.

3.4.2 Runoff from the site will be directed into the village communal drainage system through the discharge point situated at the northwestern corner of the Application Site, adjacent to Lot 154. This runoff will ultimately connect to the municipal drainage system via SMH1003271, as illustrated in **Figure 3-2**. The proposed discharge location has been in use for over 20 years by the Applicant, the landlord, and former tenants. Photographs depicting the existing drainage and the proposed discharge point are provided in **Figure 3-4.**

3.5 Proposed Drainage Arrangement and Capacity Assessment

3.5.1 Three proposed manholes will be installed connecting the discharge point and the existing private manhole in communal drainage.

3.5.2 Drawing of the drainage arrangement of the Site is presented in **Figure 3-3**. It shows that terminal storm water manhole STMH_T1(D.T.I.L 17.55) is connected to a proposed private manhole ST01(I.L. 17.45). ST01 is further connected to a proposed manhole ST02(I.L. 17.37). ST02 is further connected to another proposed manhole ST03 (I.L. 17.25). ST03 will eventually connect to the downstream existing private manhole S-05(I.L. 17.16). The drainage section between STMH_T1 and S-05 is connected via an underground stormwater pipe with a size of Ø450mm. The proposed manholes and drains will pass through Lot 153, Lot 154, Lot 155, Lot 157 and Lot 158.

3.5.3 Assessment on the flow capacity of the proposed circular pipe has been conducted as shown in ***Appendix B.*** The proposed circular pipe should not less than Ø 450mm. Detailed assessment is provided in ***Appendix B*** shown that the utilization for from manhole STMH_T1 to S-05 ranges from 91% to 93% with the consideration of 10% of sedimentation in the calculation of drainage flow capacity..

Proposed Parameter Drains

3.5.4 Two sets of U-channels (i.e. Section 1A to 1H and Section 2A to 2B) with grating cover are proposed to be constructed along the perimeter of the Site in order to facilitate the drainage collection within the Site. The Ø 600mm diameter U-channel with a gradient range from 1 in 200 to 1 in 500 gradients will collect the runoff generating from the Site. Each of the two U-channels will be connected to existing manhole STMH_T1 and eventually connect to proposed new manholes and further the SMH1003271. The indicative parameter drain is shown in ***Figure 3-3***.

3.5.5 To prevent severe runoff to the communal drainage system during rainy season, two 70 tonnes water silo will be equipped to store stormwater temporarily during heavy rain period for one hour and will gradually release the stored stormwater into the stormwater pipe until heavy raining stops.

3.5.6 The flow capacities of the internal perimeter drains were calculated using Manning's Equation. The results show that the utilisation capacity ranges from 48% to 97%.

3.5.7 A supplementary treatment system containing a sedimentation tank will be equipped on Site to filter sand and rock of the wastewater. No significantly large amount of sediment/ sand/ rock will enter village communal drainage system.

3.5.8 Since there will be no increase in runoff generation, no additional drainage impact is anticipated. The findings of this Drainage Proposal are tally with those of the Drainage Proposal dated 20 June 2024 (**paragraph 1.4.1** refers). Therefore, there is no need to further submit an updated Drainage Proposal.

3.5.9 The calculation shows that both the proposed Ø450mm circular drains and Ø600mm U-shape parameter drains have sufficient capacity for the cumulative runoff with the operation of two 70 tonnes water tank to temporarily store runoff during heavy rain. Therefore, no adverse drainage impact due to the Proposed Development is anticipated.

Figure 3-2 Identification of Catchments

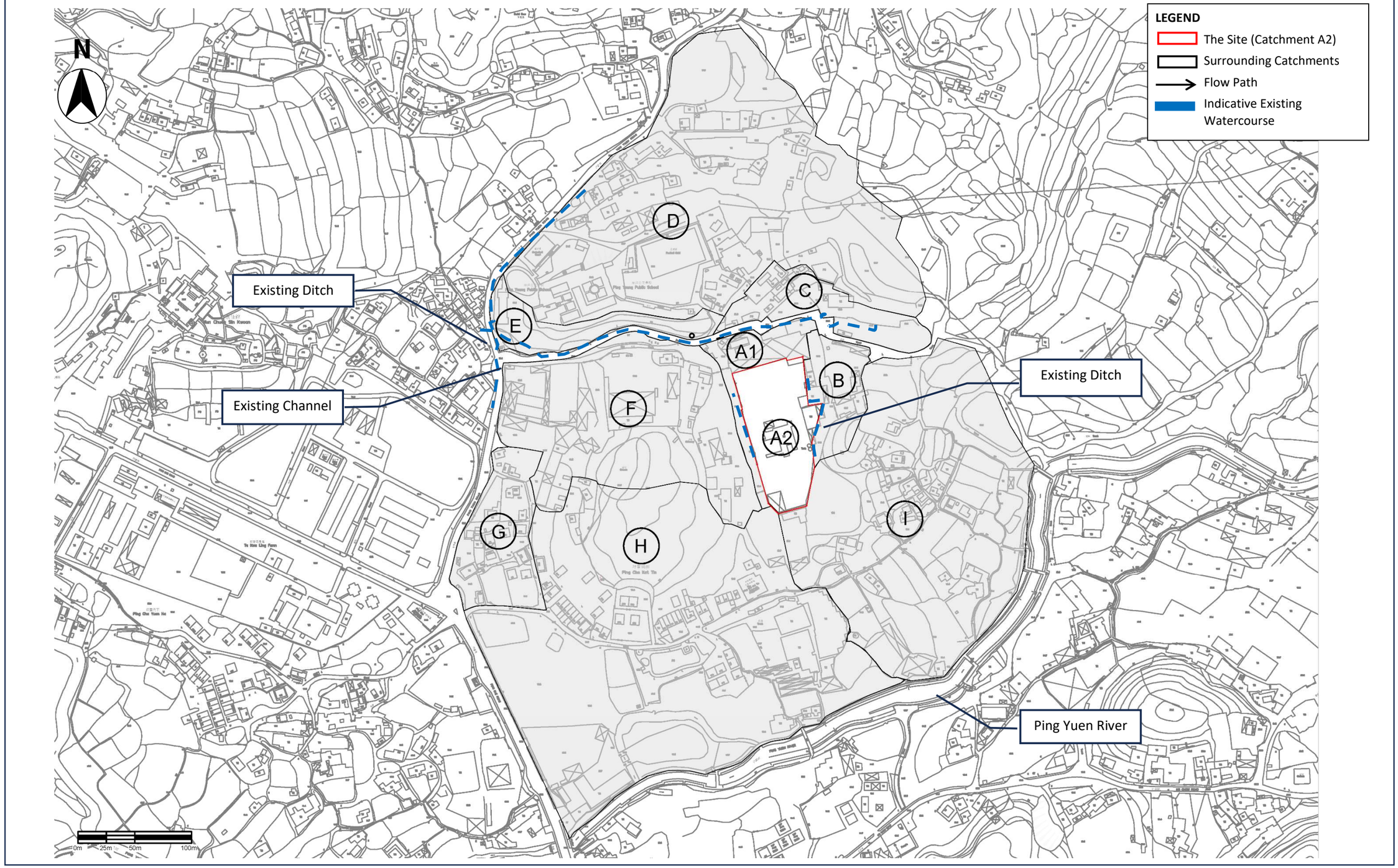


Figure 3-3 Proposed Discharge Point

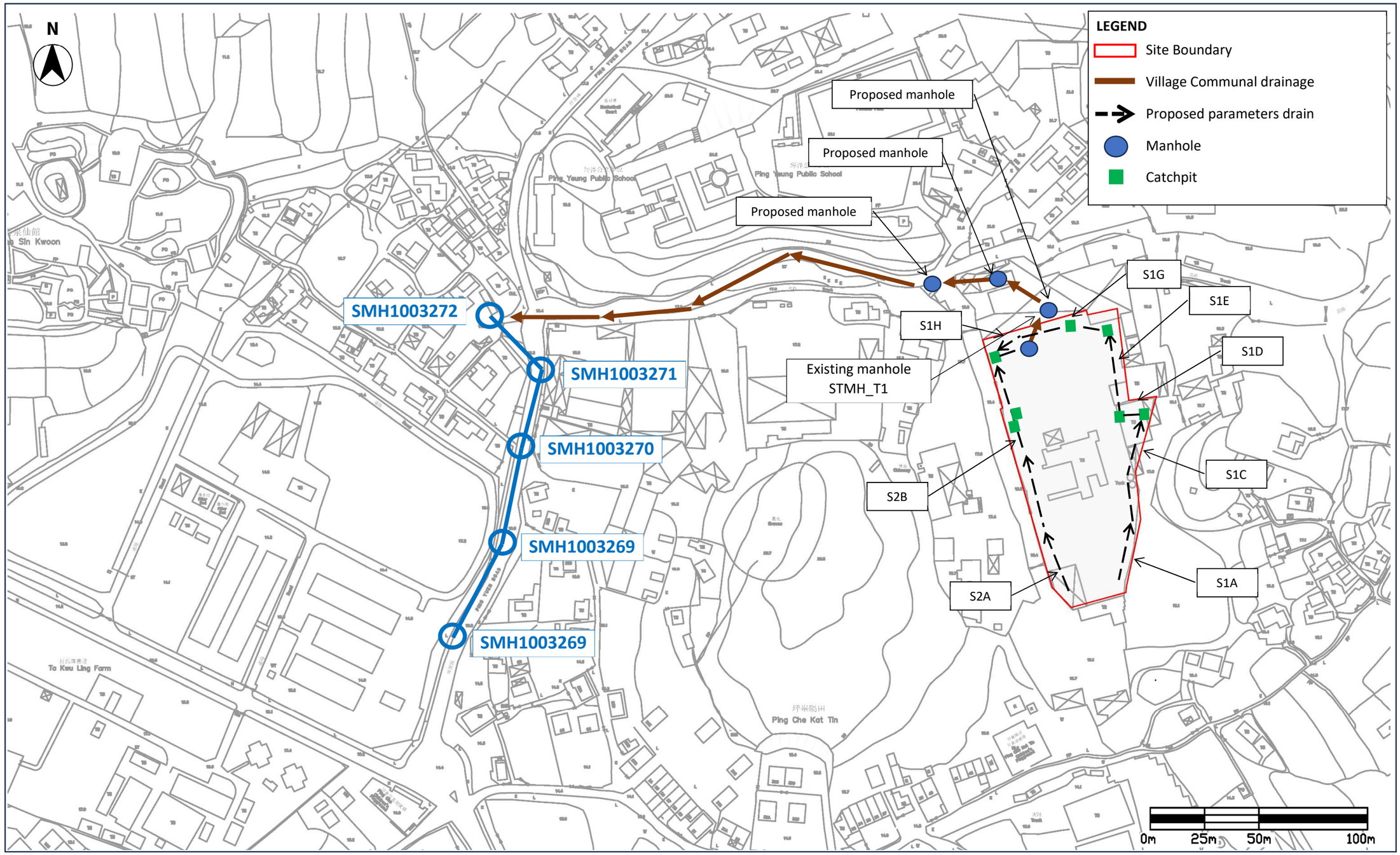
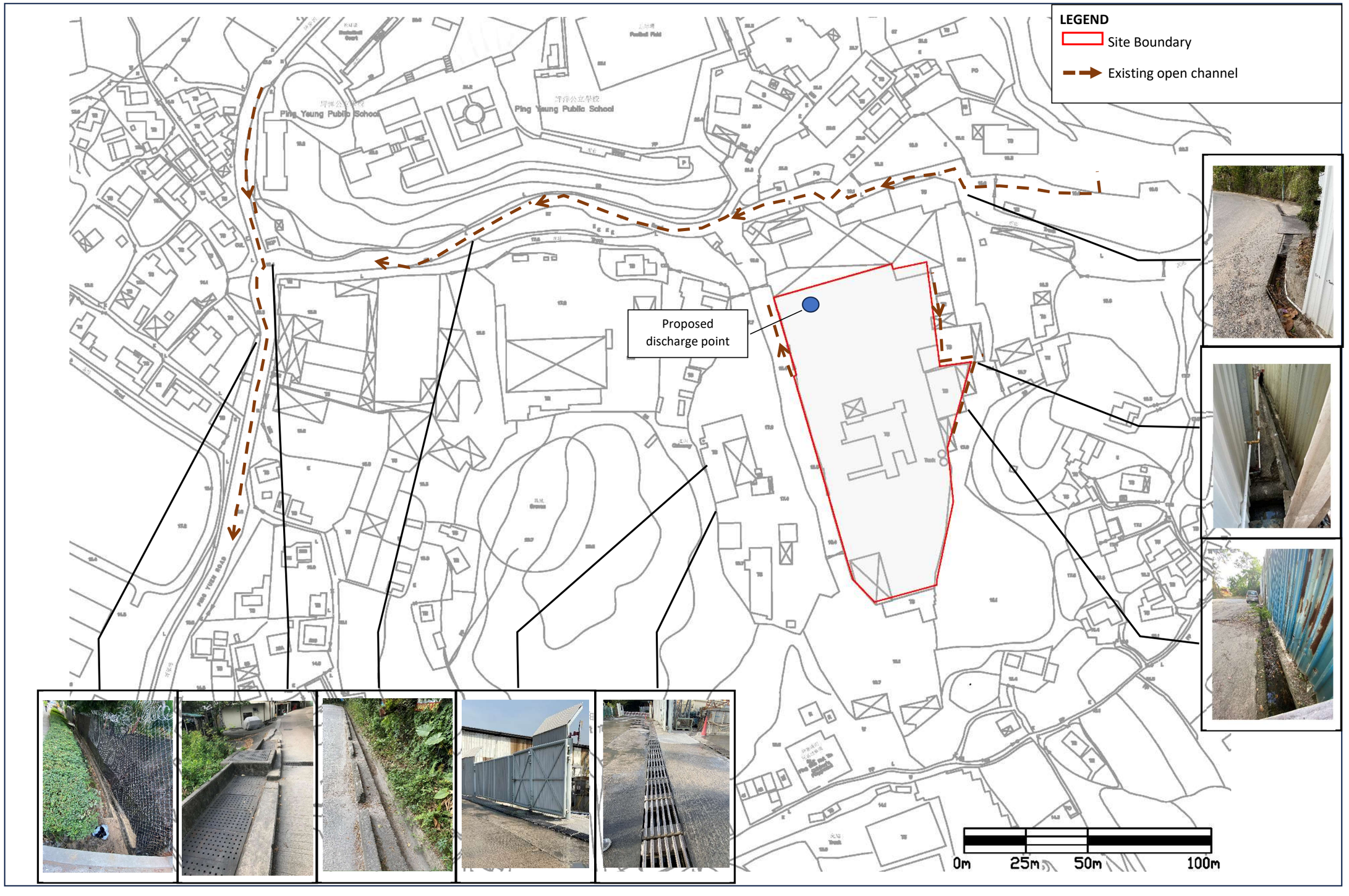


Figure 3-4 Photographs Showing locations of the Existing Drainage



4 CONCLUSION

4.1.1 Potential drainage impacts that may arise from the Site after the Proposed Development have been assessed.

4.1.2 The peak runoff for the Development Site, both before and after development, was calculated using the Rational Method, taking into account the surface characteristics under existing and future conditions. Under a 10-year return period, the estimated peak runoff from the Site is 0.284 m^3/s. Since the paving conditions will remain unchanged before and after development, no additional surface runoff is expected as a result of the Proposed Development.

4.1.3 The indicative location of the proposed Ø600mm periphery U-channels shown in **Figure 3-3** will properly divert the runoff arising from the Site which there is currently no runoff from any upper catchments overland-flowing to the Site to the terminal manhole (i.e. STMH_T1). The diverted runoff should be collected at the existing STMH_T1 manhole and eventually discharged to the village communal drainage system via three proposed manholes (ST-01, ST-02 and ST-03) and Ø450mm circular drains.

4.1.4 The capacity of proposed parameter surface drains and proposed Ø450mm circular drains has been checked as shown in **Appendix B**. The flow capacity of proposed periphery drains will range from 48% to 94%; the flow capacity of proposed Ø450mm circular drains ranges from 91 to 94% with the consideration of 10% of sedimentation in the calculation and the operation of two 70 tonnes water tank during heavy raining. The calculation shows that it can handle the runoff from the Site. As such, no adverse drainage impact is anticipated.

4.1.5 This Drainage impact assessment indicates the initial findings regarding drainage impact and indicative drainage layout. A qualified engineer should be engaged by the Architect/Contractor of the Proposed Development to review and provide detailed designs for the internal Site drainage layout.

4.1.6 In conclusion, no adverse drainage impact from the Proposed Development is anticipated.

Appendix A Runoff Calculations

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												
Site Area	0.0070	0.31	130.0	9.83	9.83	1004.5	17.24	0.644	120.08	0.95	0.0066	0.221
After the Proposed Development												
Site Area	0.0070	0.27	130.0	10.09	10.09	1004.5	17.24	0.644	119.32	0.95	0.0066	0.219

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												
Site Area	0.0070	0.31	130.0	9.83	9.83	1157.7	19.04	0.597	155.50	0.95	0.0066	0.286
After the Proposed Development												
Site Area	0.0070	0.27	130.0	10.09	10.09	1157.7	19.04	0.597	154.65	0.95	0.0066	0.284

Note:

1. Runoff is calculated in accordance with DSD's "*Stormwater Drainage Manual (with Eurocodes incorporated) - Planning, Design and Management*" (SDM), fifth edition, January 2018.
2. Storm Constants were adopted from Table 3a Storm Constants for Different Return Periods of HKO Headquarters of DSD's *Corrigendum No. 1/2024.*

Appendix B Calculation of Drainage Capacity and Number of Stormwater Storage Tank

Drainage Capacity of Proposed U-Channels within the Site

From	To	Description	Shape	Diameter	Start Level	End Level	Slope (s)	Cross Section Area, m2	Wetted Perimeter	Hydaralius Radius, m	Manning Roughness Coefficient	Mean Velocity, m/s	Capacity Flow, m3/s	Total Runoff, m3/s	% of capacity	Remark
S1A	S1C	Proposed U-Channel	U-shape	0.6	18.5	18.26	0.0022	0.32	1.54	0.21	0.016	1.03	0.33	0.255	77%	OK
S1D(Catch pit 1)	S1D(Catch pit 2)	Proposed U-Channel	U-shape	0.6	18.26	18.24	0.0015	0.32	1.54	0.21	0.016	0.85	0.27	0.255	94%	OK
S1E	S1E	Proposed U-Channel	U-shape	0.6	18.24	18.10	0.0033	0.32	1.54	0.21	0.016	1.27	0.41	0.255	63%	OK
S1G	S1G	Proposed U-Channel	U-shape	0.6	18.10	17.96	0.0036	0.32	1.54	0.21	0.016	1.32	0.42	0.255	60%	OK
S1H	STMH_H1	Proposed U-Channel	U-shape	0.6	17.96	17.9	0.0030	0.32	1.54	0.21	0.016	1.20	0.39	0.255	66%	OK
S2A	S2A	Proposed U-Channel	U-shape	0.6	18.55	18.50	0.0033	0.32	1.54	0.21	0.016	1.27	0.41	0.255	63%	OK
S2B	STMH_T1	Proposed U-Channel	U-shape	0.6	18.5	17.9	0.0058	0.32	1.54	0.21	0.016	1.67	0.54	0.255	48%	OK

Legend

d = pipe diameter, m

r = pipe radius (m) = 0.5d

A_w = wetted area (m^2) = πr^2

P_w = wetted perimeter (m) = $2\pi r$

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 Site during peak season, m^3/s

Drainage Capacity of Proposed Stormwater Drainage Pipe outside the Site

Description	Shape	Catchment Description	d	r	Aw	Pw	R	s	ks	V	Qc	Qp'	Is Qc > Qp' ?	% of capacity
Proposed Stormwater Pipe Connecting to Proposed Discharge Point	Circular Pipe	STMH_T1 to ST01	0.450	0.225	0.159	1.414	0.112	0.0061	0.06	1.953	0.279	0.255	Y	91%
Proposed Stormwater Pipe Connecting to Proposed Discharge Point	Circular Pipe	ST01 to ST02	0.450	0.225	0.159	1.414	0.112	0.0057	0.06	1.894	0.271	0.255	Y	94%
Proposed Stormwater Pipe Connecting to Proposed Discharge Point	Circular Pipe	ST02 to ST03	0.450	0.225	0.159	1.414	0.112	0.0057	0.06	1.894	0.271	0.255	Y	94%
Proposed Stormwater Pipe Connecting to Proposed Discharge Point	Circular Pipe	ST03 to S-05	0.450	0.225	0.159	1.414	0.112	0.0058	0.06	1.910	0.273	0.255	Y	93%
Proposed Stormwater Pipe Connecting to Proposed Discharge Point	Circular Pipe	S05 to S-07	0.450	0.225	0.159	1.414	0.112	0.0187	0.06	3.498	0.501	0.255	Y	51%

Where

d = pipe diameter, m

r = pipe radius (m) = 0.5d

A_w = wetted area (m^2) = (r^2/2) (b + sinq)

P_w = wetted perimeter (m) = br

R = Hydraulic radius (m) = A_w/P_w

k_s = hydraulic pipeline roughness, mm

V = Velocity of flow calculated based on Colebrook-White Equation, m/s

Q_c = Flow Capacity including 10% for siltation, m^3/s

Q_p = Estimated total peak flow from the Site during peak season, m^3/s

Note

1. Flow capacity of pipe segment is calculated based on Colebrook-White Equation.
2. The diameter and gradient of the proposed stormwater pipe is indicative only. Its details will be subject to change during the detailed design stage.
3. The ks value of 0.06 in good condition for precast concrete pipes with 'O' ring joints recommended in Table 14 of the SDM for design purpose is adopted.

Required number of Stormwater Storage Tank

Catchment ID	Peak runoff (Q_p), m^3/s	Maximum peak runoff enter the drain m^3/s	Surplus runoff (Q_p), m^3/s	Duration of Storm, hours	Runoff Volume Required, m^3/s
Catchment A1 within the Site	0.284	0.255	0.029	1	104
Required Number of 70-Tonne Tanks					2

Accountability

We understand the importance of being accountable to each other and our clients.

Passion

We are completely passionate about providing practical solutions and outcomes that deliver for our clients.

Insight

We work in an environment that encourages and values insight as a critical quality which informs our decisions and our clients and supports practical solutions and project delivery.

Integrity

We behave with respect and honesty toward each other, our clients and our stakeholders.

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