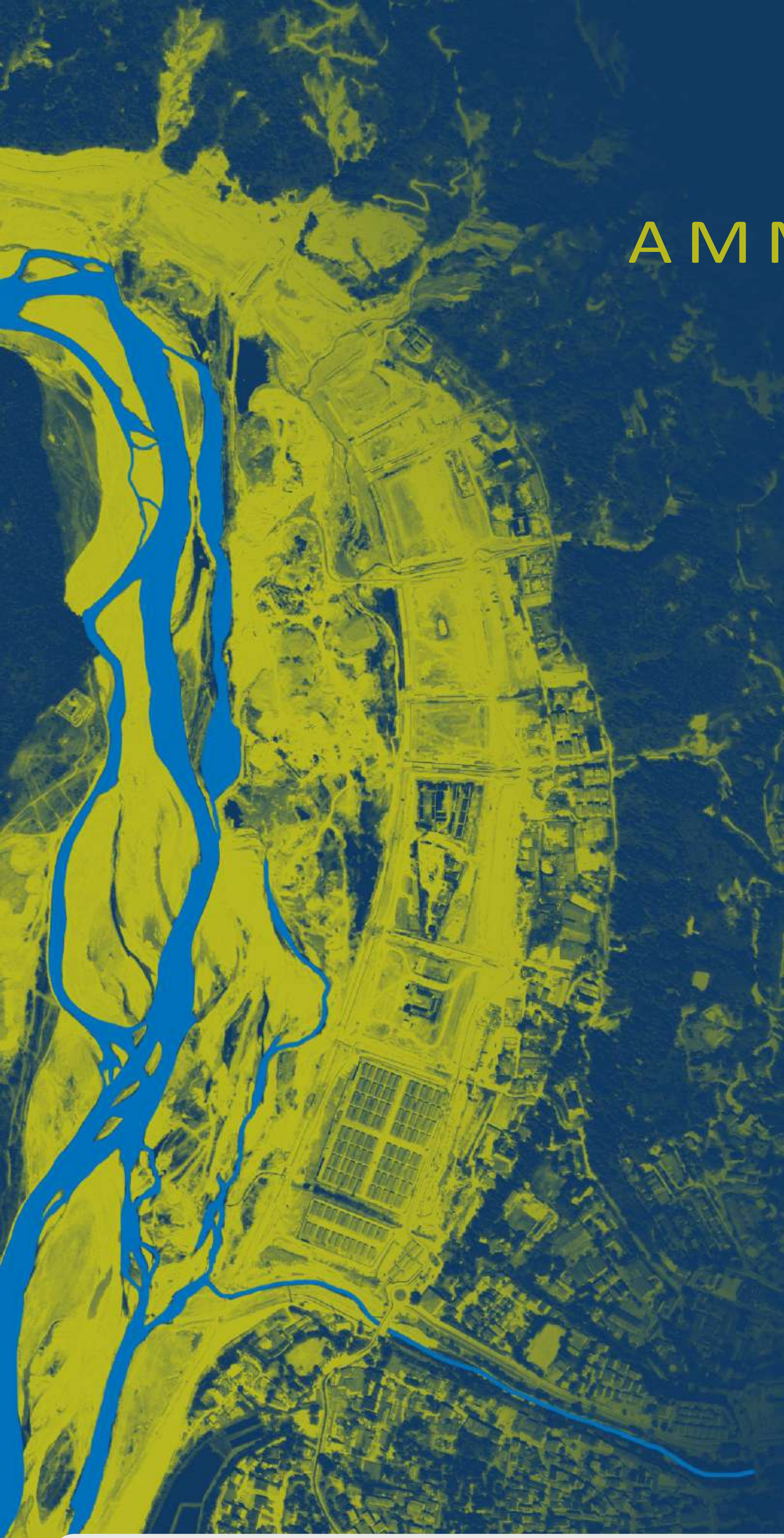

AMMOCHHU

Urban Design and Detailed Infrastructure Design Vol II

FINAL REPORT

CLIENT:
Royal Government of Bhutan
Phuentsholing Thromde

CONSULTANTS:
WHITE CYPRESS ASSOCIATES

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1 LAP FILL ANALYSIS

1.1 Existing Topography

Initially, before the PTDP[1] and PCR[2] projects the LAP was naturally at higher elevation than the PTDP area as indicated in the Figure 1-1. When PTDP project was conceived, plan & design, the design levels of embankment were higher than natural ground levels due to river training works for flood mitigation and land reclamation. With development of PTDP project the certain areas/parts of the LAP will be lower than the design levels of embankment of PTDP project and the PCR project was supposed to do the LAP filling while laying their road embankment and the estimated fill quantity was 3,83,000 m3.

However, due to datum error in PTDP project the design levels of PTDP project was raised by 1.5m and the design levels of PCR road had to be raised by 1.5m. Further, PCR project raised the design level s of road embankment by 1.5m in average with maximum of 3.5m across crossings in the Outfalls to maintain adequate clear height for debris cleaning in the Culverts/Bridges in the future.

Although, there was raise in the design levels of embankment of both PTDP & PCR project, the LAP filling quantity in the scope of PCR project was same. So, Thromde[3] & PCR project team came to understanding that PCR project will do the LAP filling till first internal service road flush with PCR four-lane road.

While carrying out the LAP filling by PCR project, they were not able do the LAP filling in certain stretch as per project scope due to existing warehouses & other temporary structures. Due to all these reasons, a large area of LAP is now low-lying causing water clogging during monsoon and posing high flood and debris sedimentation risk in the future as shown in Figure 1-2. There has also been challenges in designing an efficient Storm Water drainage system in the LAP.

Hill shade model of the Existing Topography of the LAP is shown Figure 1-3 & Figure 1-4 shows the areas that are low-lying i.e., areas below the PCR elevations, highlighted in orange.

1.2 Filling Analysis

Filling some areas of the LAP is required to have functioning internal storm water drainage as mentioned earlier. For this filling analysis for three different options were carried out – one each for the two drainage options and one for the PCR Level fill. Filling analysis and volume calculations were done using GIS software, ArcMap 10.8.

[1] Phuentsholing Township Development Project
[2] Phuentsholing Chamkuna Road
[3] Phuentsholing Thromde

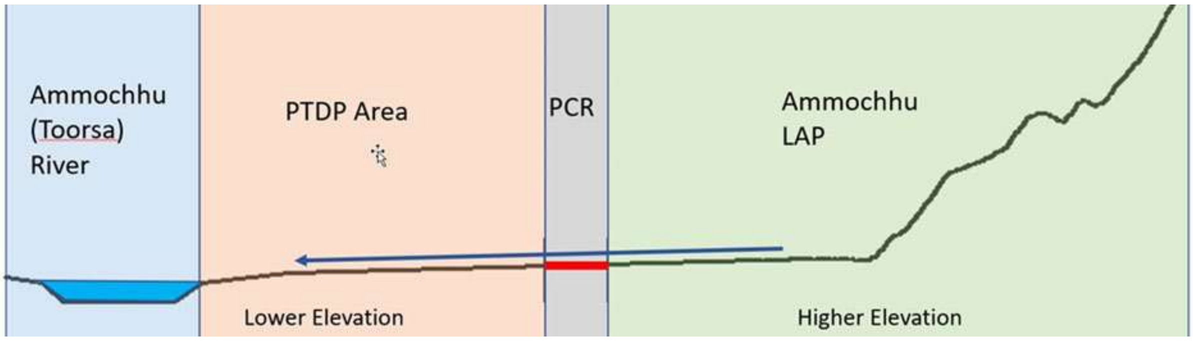

Figure 1-1:Diagrammatic representation (NTS) of the topography of the area before the PTDP and PCR Projects.

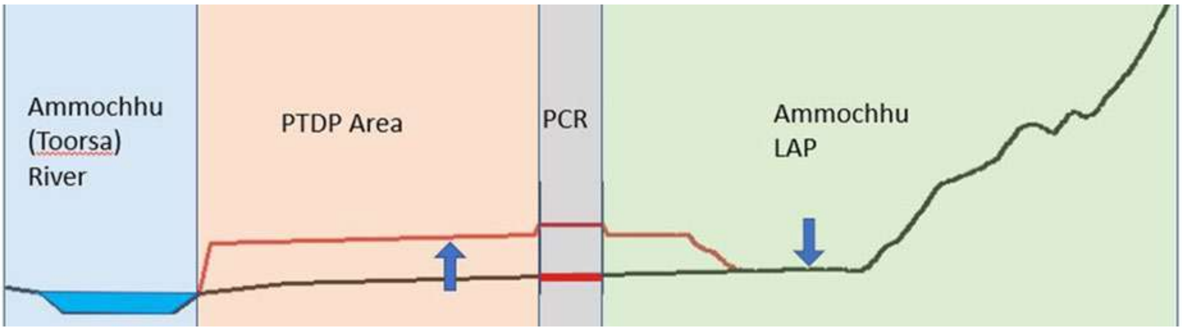

Figure 1-2:Diagrammatic representation (NTS) of the existing topography of the area.

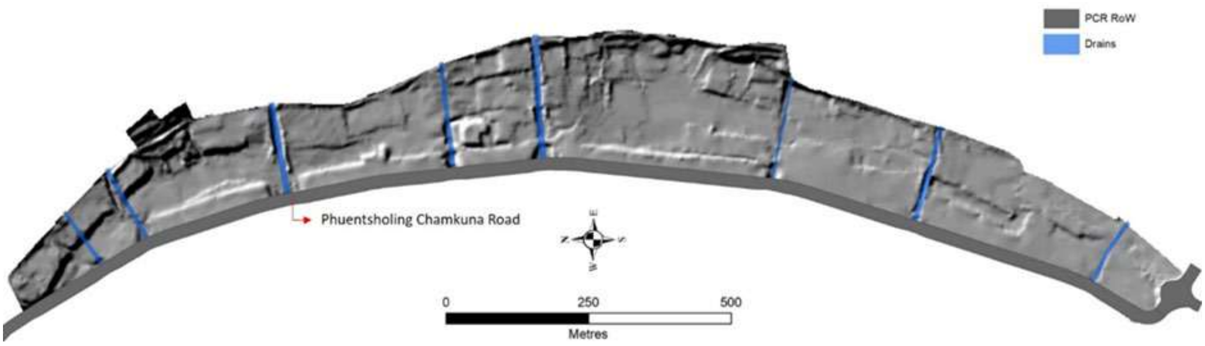

Figure 1-3:Hill shade Model of Existing Topography.

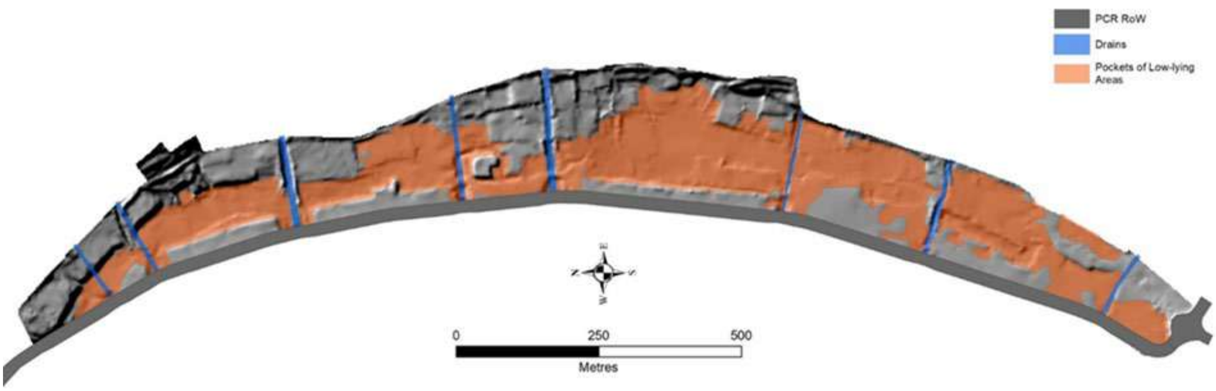

Figure 1-4:Low-lying areas.

1.2.1 Filling Required for Internal Stormwater Drainage Layout Option 1

Initially, the existing topography survey done by the Consultant as per the scope of work was studied thoroughly to have overall idea of topography of the site.

The design invert levels of eight PCR Outfalls sourced from PCR official were also studied thoroughly to decide on the efficient Storm Water drainage network.

Consultation with the PCR officials was also done to make the necessary minor changes in invert levels of PCR Outfalls to have minimum LAP filling to resolve the Storm Water drainage issues.

After the final design invert levels of Outfalls were fixed, the Storm Water drainage network was proposed running underneath the footpath. The Storm Water drains was placed at critical location, discharge from its catchment were calculated using Rational method & the size of drain was calculated using Manning formula for Open-flow channel. The Slope & velocity (minimum velocity-1.5 m/s)[4] of flows was adjusted to come-up with minimum filling in the LAP to resolve the drainage issues.

The Storm Water drainage network considered here for the ideal condition where consideration is only given to solve the drainage issues & network is shown in Figure 1-5.

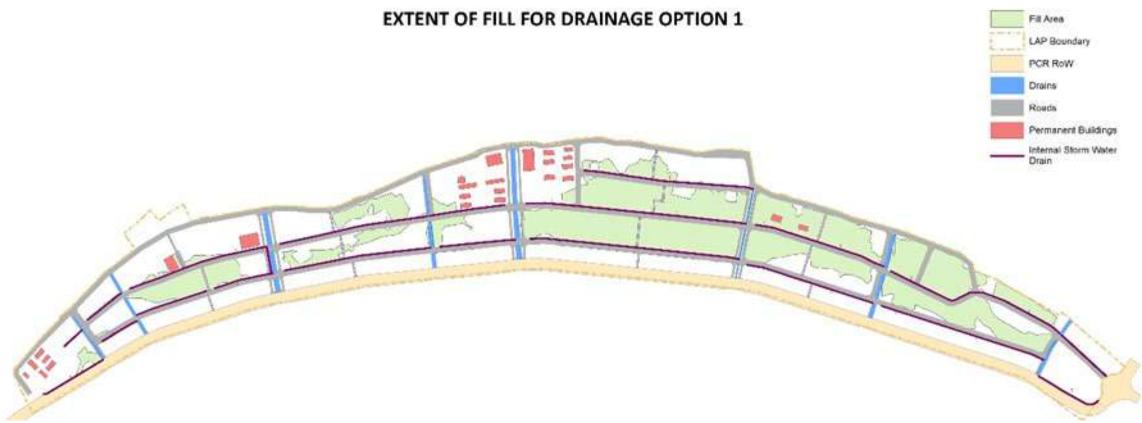

Figure 1-5:Fill Extent for Option 1 Filling.

The total of 146,182 m3 of LAP filling will be required as per this option fill extent.

Three building are affected by this fill option, one of which is under construction and is not inhabited. The buildings considered are the ones which are within the proposed plot boundaries and whose land uses are within the permissible uses under UV-1 (A) precinct in the existing DCR as per the LAP review 2019. The details of

[4] The minimum velocity recommended in IRC SP:50-2013 to ensure self-cleaning of the drain.

affected building are discussed below in Table 1-1.

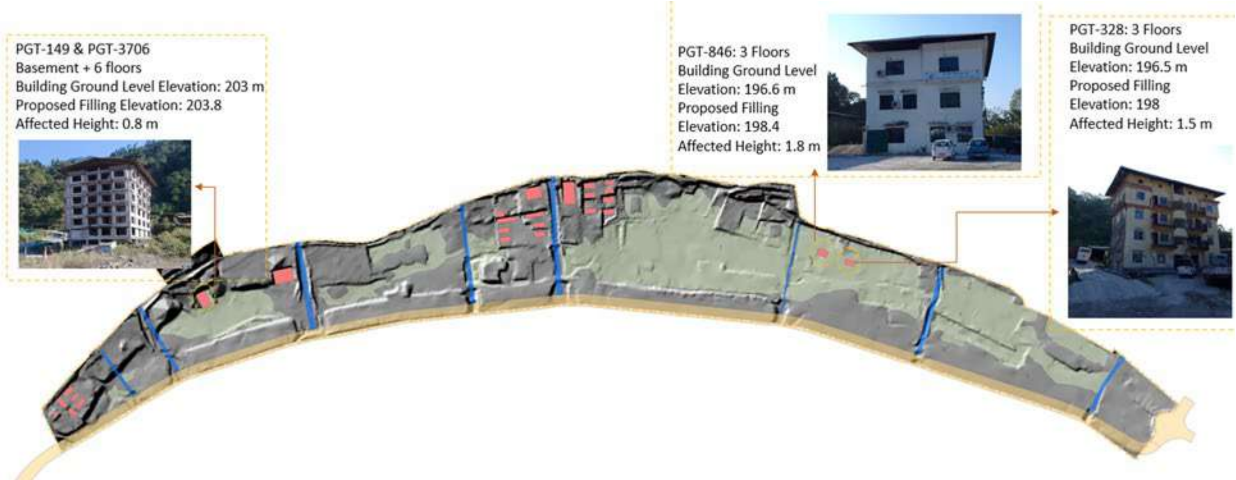

Figure 1-6:Affected Buildings under option 1 filling.

Table 1-1:List of affected buildings under option 1 filling.

Plot ID	Floors	Building Ground Elevation (m)	Proposed Filling Elevation (m)	Affected Height (m)
PGT-149 & PGT-3706	Basement + 6 floors	203	203.8	0.8 (Only Basement is affected)
PGT-846	3 Floors	196.6	198.4	1.8
PGT-328	3 Floors	196.5	198	1.5

1.2.2 Filling Required for Internal Stormwater Drainage Option 2

For this option, the Storm Water drainage Network Option 1 was analyzed further to look into possibility of running the certain Storm Water drainage parallel to PCR outfalls to get the absolute minimum filling in the LAP.

The Stormwater drains was placed underneath the footpath and wherever possible drain was diverted to adjacent drain through open spaces & proposed footpath running between plots. For all the drains the Peak discharge from its catchment was calculated using Ration method and the size of drains was calculated using Manning's formula for open-flow channel.

The Slope & velocity (minimum velocity-1.5 m/s) of flows was adjusted to come-up with minimum filling in the LAP to resolve the drainage issues.

The following maps (see Figure 1-7) show the proposed drainage layout option 2 and the extent of fill required if this network is adopted. A total 115,671 m^3 of LAP filling will be required as per this option fill extent.

Two building are affected by the fill, one of which is under construction and is inhabited. The buildings considered are the ones which are within the proposed plot boundaries and whose land uses are within the permissible uses under UV-1 (A) precinct in the existing DCR as per the LAP review 2019. The details of affected building are discussed below in Table 1-2.

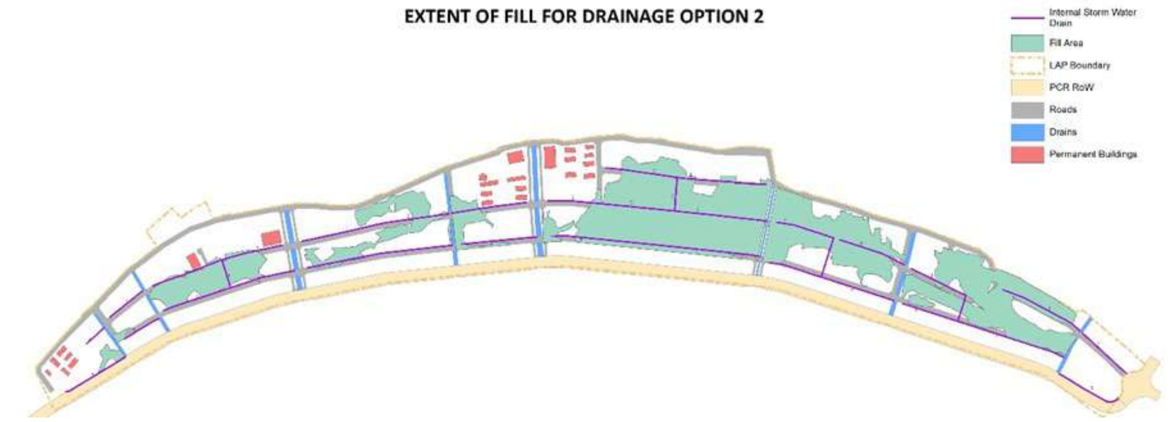

Figure 1-7:Fill Extent for option 2 Filling.

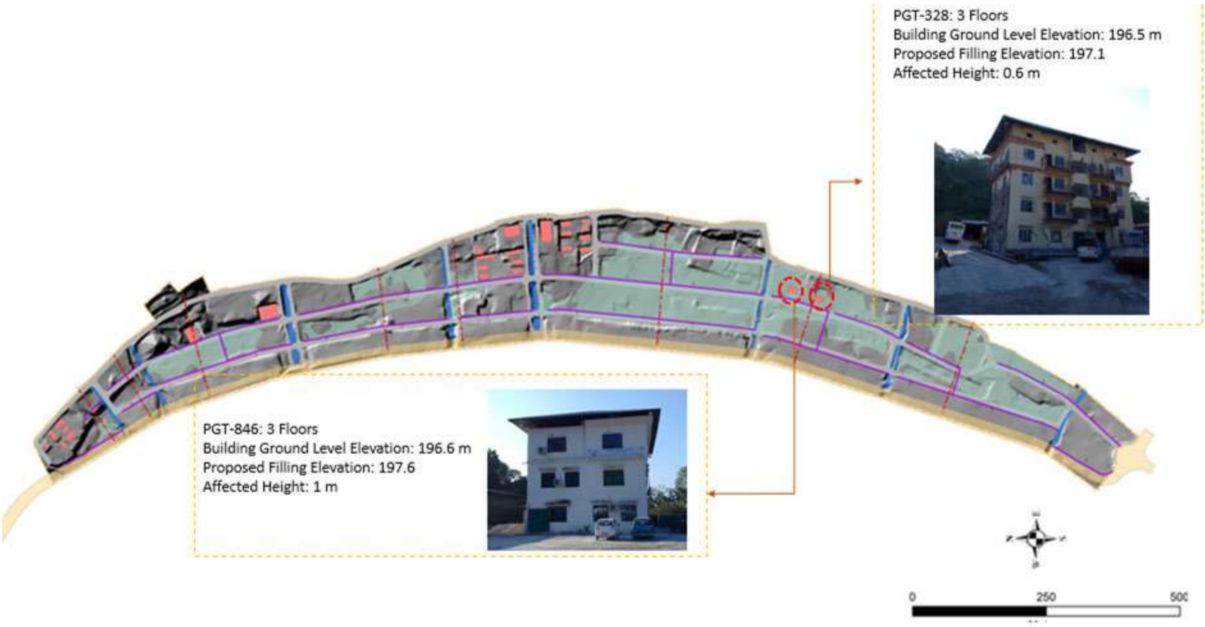

Figure 1-8: Affected buildings under option 2 filling.

Table 1-2:List of affected building under option 2 filling.

Plot ID	Floors	Building Ground Elevation (m)	Proposed Filling Elevation (m)	Affected Height (m)
PGT-846	3 Floors	196.6	197.6	1
PGT-328	3 Floors	196.5	197.1	0.6

1.2.3 Filling the LAP at PCR Level

Filling the LAP at PCR Level was explored for the following reasons:

i. To solve the storm water drainage issue in the best way possible. The natural terrain of the area slopes from the hillside (East) towards Ammochhu (West), however with the raised PCR and LAP area along the road, large areas of the LAP fall below the PCR. This has resulted in water clogging within the LAP during monsoons.

ii. With the LAP level maintained at least at the PCR level or higher, the visual connection with the PTDP area and river can be maintained.

PCR project has undertaken filling of the LAP extending up to the first internal road running parallel to the PCR. The filling elevation of the LAP that PCR project has maintained is 600mm below the finished design level of the PCR.

Highlighted in purple (see Figure 1-9) are the areas in the PCR Fill Scope which are higher (at the time of survey in October 2021) than the finished design height. Elevation of other areas within PCR fill scope are equal to or lower than the proposed fill elevation. Detailed Profile Sections at regular intervals are provided in Annex A. It was informed by the PCR Project Implementation unit that filling in these areas are still under process and will be levelled to the Finished Design Level (600 mm below the finished design level of the PCR) when completed. Around 38,000 cu.m of fill in PCR Project scope (10% of their total fill volume) is yet to be completed as of 16 December, 2021. PCR project will fill until the first LAP road (western avenue).

Following the same elevation of fill from the PCR fill extent, further fill in the LAP, beyond PCR scope is modelled as the proposed ground level for the LAP. However, in some areas, for the proposed internal storm water drains to work properly, the filling required is higher than the PCR Fill Level, which has been considered in the final proposed surface (see Figure 1-10 and Figure 1-11). Fill volume under this option is 197,904 cubic meters.

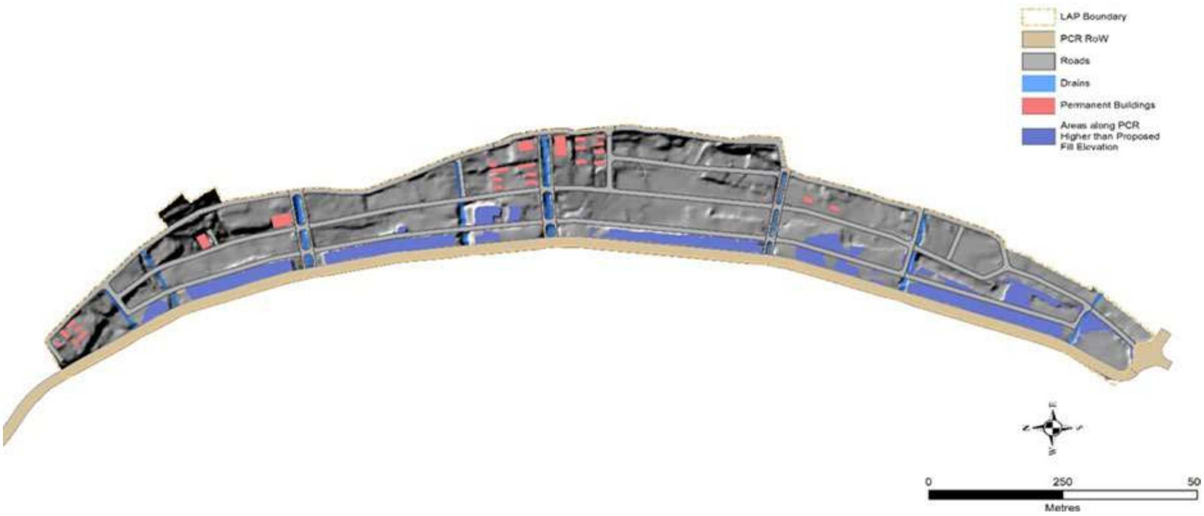

Figure 1-9: Highlighted in Purple are the areas of Fill by PCR Higher than the Proposed Fill Level.

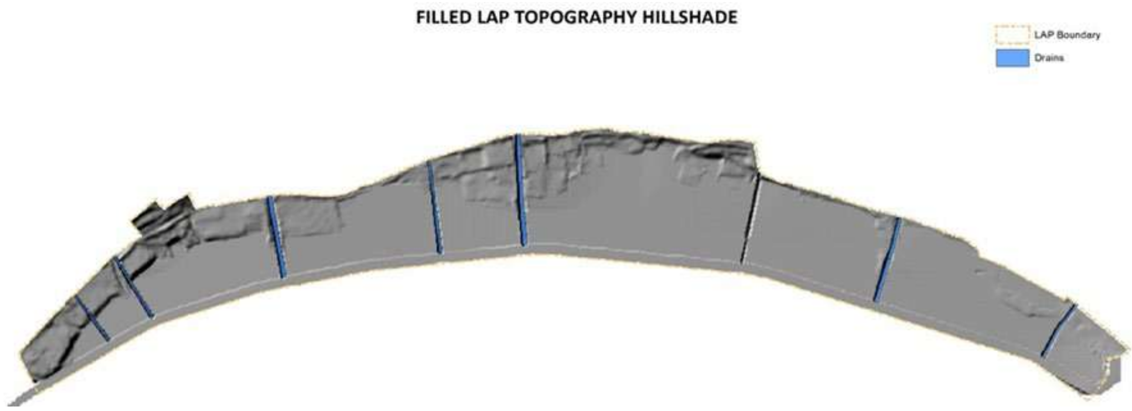

Figure 1-10:Modelled Proposed Surface after Filling at the PCR Level (option 3).

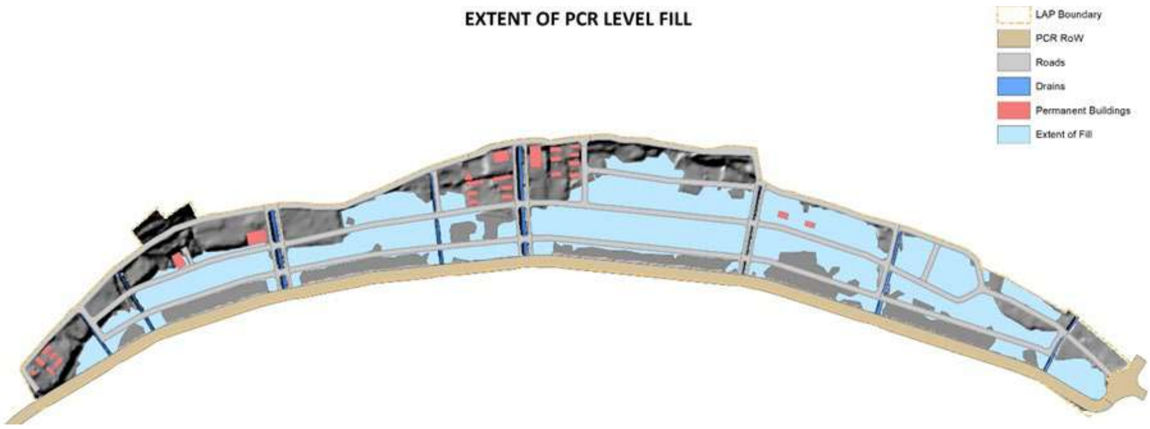

Figure 1-11: Fill Extent Required for Proposed Surface under fill option 3.

Four building are affected by this fill option, two of which is under construction and is not inhabited. The buildings considered are the ones which are within the proposed plot boundaries and whose land uses are within the permissible uses under UV-1 (A) precinct in the existing DCR as per the LAP review 2019. The details of affected building are discussed below Table 1-3.

Table 1-3: List of affected building under option 3 filling.

Plot ID	Floors	Building Ground Elevation (m)	Proposed Filling Elevation (m)	Affected Height (m)
PGT-149 &PGT-3706	Basement + 6 floors	203	204.2	1.2(Only Basement is affected)
PGT-1477	Basement + 6 floors	203.8	204.2	0.4 (Only Basement is affected)
PGT-846	3 Floors	196.6	198.4	1.8
PGT-328	3 Floors	196.5	198.2	1.7

Figure 1-12:Buildings affected under fill option 3.

1.3 Final Fill Option Adopted

Thromde decided to opt for fill option 3 on the following recommendation by the experts:

a) High intensity of rainfall received in the area during monsoon coupled with huge amount of sediment that flows down to the LAP from the areas immediately upstream pose a great flash flood and sedimentation risk in the LAP. Filling the LAP at PCR level or higher, removes pockets of low-lying area hence eliminating the ponding issue in future. Moreover, due to climate change, the rainfall pattern has been erratic and unpredictable which are only going to get worse in the coming years.

b) The estimated cost difference in filling options is around Nu. 10 million only while the long-term benefits in terms of operation and maintenance will be immense.

c) Only the ground floors of two buildings are affected by this fill by less than 2 meters. These two buildings are affected in all three fill options.

d) Initial intent was to fill the LAP at PCR level. Later, levels in PTDP and PCR projects were raised by about 1.5 m. However, fill level in the LAP was not raised causing it to be lower than the downstream areas.

1.4 Additional LAP filling

Although the LAP filling at the PCR level resolved the drainage issues in the LAP. While carrying the road design, the road R4 had to be raised to maintain the gradient with the Eastern Avenue Road R1.

With the raised in the design level of road R4, the plots within the Eastern Avenue Road R1 & Road R4 are lying 3m on an average below the design road level. In order to resolve the drainage of these area, the additional fill volume of 40,555.50 m^3 will be required and the quantity has been included in the LAP filling.

1.5 Bill of Quantities and Estimate

The total volume of 238,459.50m^3 of filling is required in the LAP and the total cost of implementing the LAP filling is Nu. 70,731,734.44.

2 TECHNICAL REVIEW OF OUTFALL

2.1 Current Design of the Outfalls

Eight of the twelve outfalls of the PCR project fall under Thromde Boundary on the east side of Phuentsholing Chamkuna Road (PCR). The slope of the drainage channel is steeper for the sections in the Ammochhu Local Area Plan (LAP), while the slope is lesser for the sections that are under the finished level of PTDP fill up level.

The seven outfalls namely OF1, OF2, OF3, OF4, OF5, OF6 & OF7 are designed as open channels. The cross section of these outfalls is trapezoidal. Its bottom width varies from 0.4 m to 2.0m, its top width varies from 4.1m to 9.3m & its height varies 0.9m to 2.0m. While outfall 2a is designed as a ducted u-shape drain & its width and height are 2.5m & 2.1m respectively.

The Trapezoidal section of outfalls is 100mm thick M20 Reinforced Cement concrete (RCC) slab with nominal reinforcement. It is designed as an RCC lining with nominal reinforcement because there isn't significant active pressure acting on it. Moreover, it is only designed to withstand pressure from discharge flowing through it. The ducted section of outfall is 250mm thick M20 RCC wall with 150mm thick covered slab. It is designed to withstand active pressure due to backfill/ earth fill.

All the Outfalls except outfall 2a are designed as trapezoidal cross section & the designed invert levels of Outfall the follow & lay below the existing ground profile. These outfalls are ethically pleasing from the Urban design aspect & is safer for the public in case of an accident as it has a gentle side slope. Moreover, from an engineering aspect it is desired because the ratio of flow area to contact area with the wall is very high thereby reducing the probability of debris accumulation along it.

The typical detail of trapezoidal drain is shown in the Figure 2-1 .

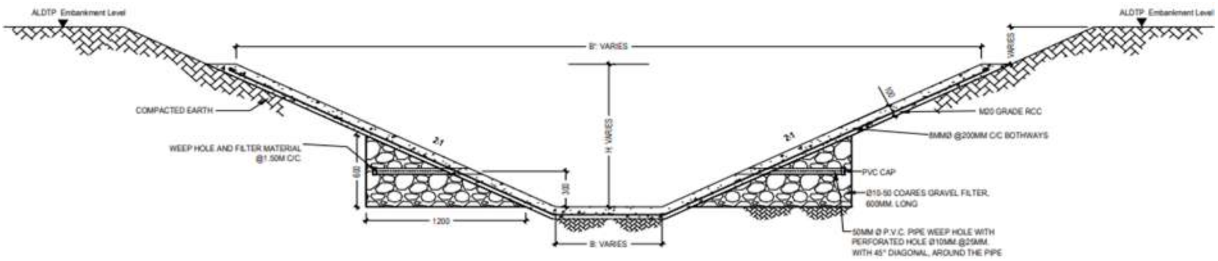

Figure 2-1: Typical Section of Trapezoidal Outfall.

The Outfall 2a is ducted & covered U-shape drain. Although the designed section of Outfall 2a is adequate, its designed invert levels run above the existing ground especially toward the end of the Outfall as shown in the figure 1-2. Consulting with the PCR team it was decided & agreed that we cannot further reduce the designed invert level because of the fixed Culvert 2a's invert level & the constraint on the slope of Outfall.

With the LAP filling, the Outfall 2a's profile on left side will be fully covered & drainage issues on that side will be resolved but on the right side the Outfall 2a's profile toward the end will protrude out of the proposed

LAP filling as shown in **Figure 2-3**. However, the LAP filling was proposed considering that the drainage of the area between Second & third service road will drain storm water to the Outfall 2. Hence there is no issue with the Outfall 2a.

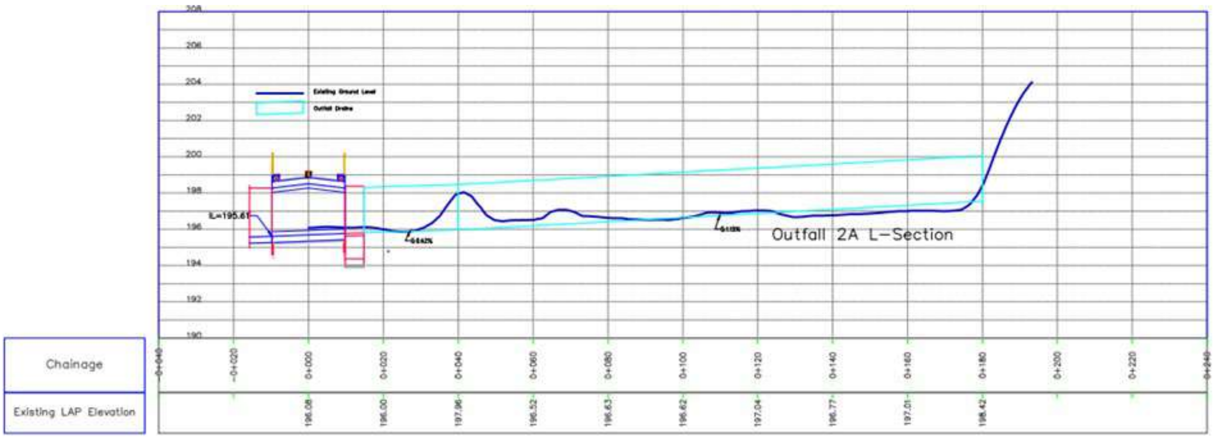

Figure 2-2: Longitudinal Profile of Outfall 2a.

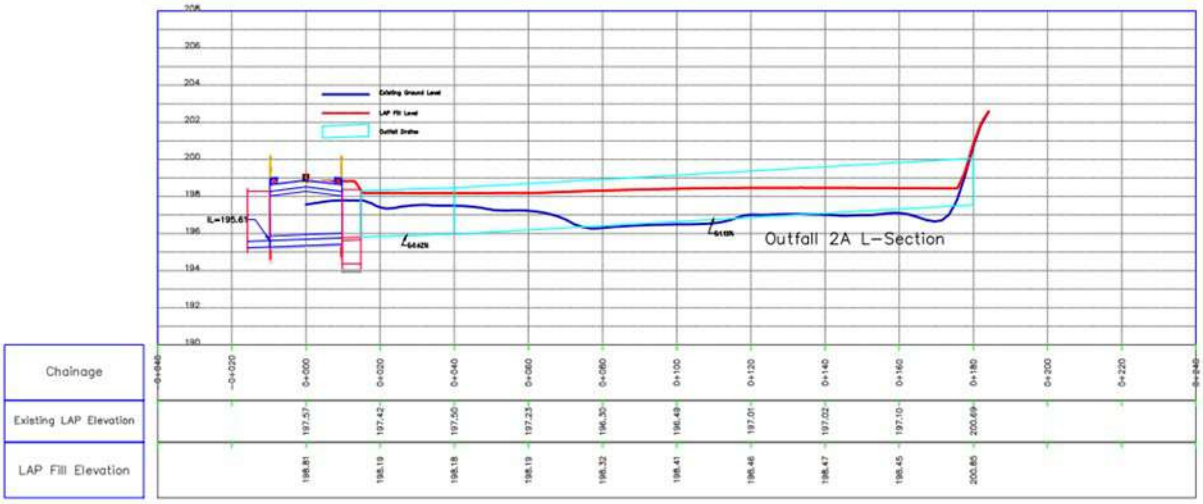

Figure 2-3: Longitudinal profile 1-2m away (right side) from longitudinal section of Outfall 2a.

Moreover, if we are to propose the LAP filling that would fully cover the Outfall 2a's profile on right side still we wouldn't be able to drain the storm water to Outfall 2a because the area between Outfall 2 & 2a is relatively flat and the designed invert levels of outfall 2a is slightly higher than the designed invert levels of Outfall 2. For the following reason as stated above there are no issues with the Outfall 2a & section details of Outfall 2a is given in the **Figure 2-4**.

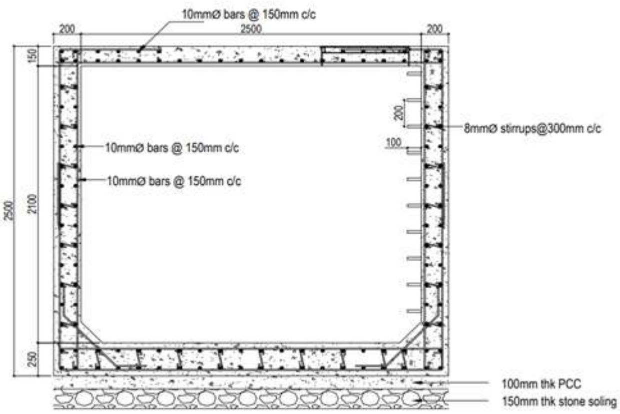

Figure 2-4: Typical Section of Ducted Outfall.

2.2 Issues

2.2.1 BACKFILLING

The final design level of PCR Highway was achieved after two consecutive level raises. Initially, PTDP raised the level by 1.5m whereby the design level of PCR four-lane had to be raised by 1.5m. Further the level was raised by 1.5m to 3.5m to provide clearances under Culvert/bridge by PCR project. This rise in level of PCR four-lane Highway starts from Outfall 2a and ends at Bridge BR-04. Similarly, the design invert level of Outfalls was raised by 1m from initial design invert level. Due to all these raises in the level, Ammochhu LAP's area between first, second & third internal roads were laying below the design level of PCR four-lane Highway creating drainage issues in these areas. Hence, back filling is necessary and recommended to resolve drainage issues in Ammochhu LAP.

When back filling is done in the LAP, the difference between LAP fill levels & top level of PCR Outfalls in LAP will be as follow;

a) At the gentle slope section of Outfall 1 & Outfall 2, the embankment height above drain varies from 0.5m to 1.1m on an average as shown in the figure 2-1. There isn't a significant difference between the proposed LAP filling level & the existing ground level.

b) At the gentle slope section of Outfall 3, Outfall 4, Outfall 5, Outfall 6 & Outfall 7, the embankment height above the drain varies from 2.0 to 3.5m on an average as shown in the figure 2-2. The difference between the proposed LAP filling & the existing ground level for all the above Outfall varies from 1.0m to 3.72m. The maximum difference of 2.5m to 3.72m were found at the start of the Outfalls & this difference represent the level raised to achieved the proposed LAP filling level at Outfalls.

c) Similarly, toward the steeper slope section of Outfall 3, Outfall 5, Outfall 6 & Outfall 7 the difference between the existing topography & top of Trapezoidal drain is 2m average.

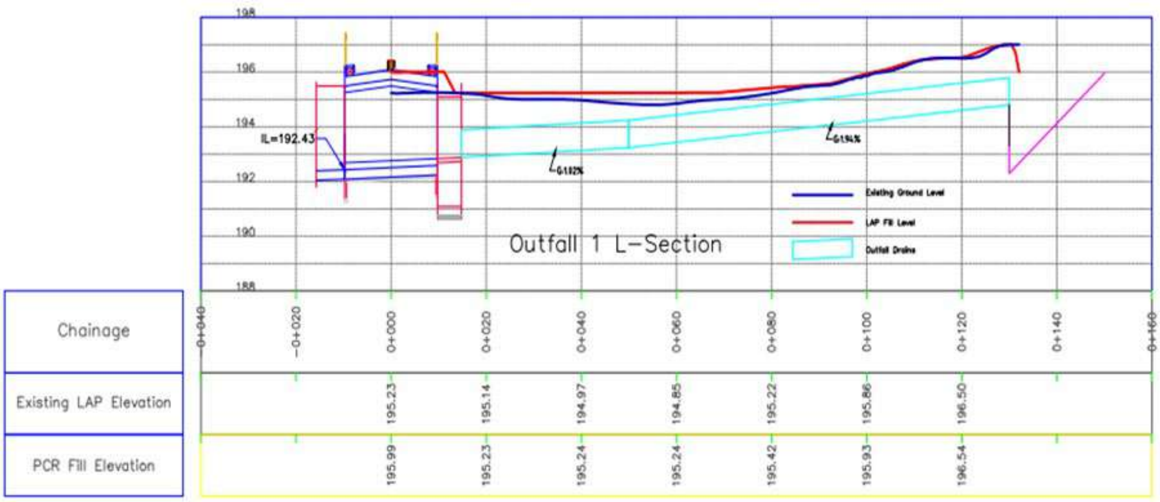

Figure 2-5: Longitudinal profile at 1-2m away from Longitudinal section of Outfall1.

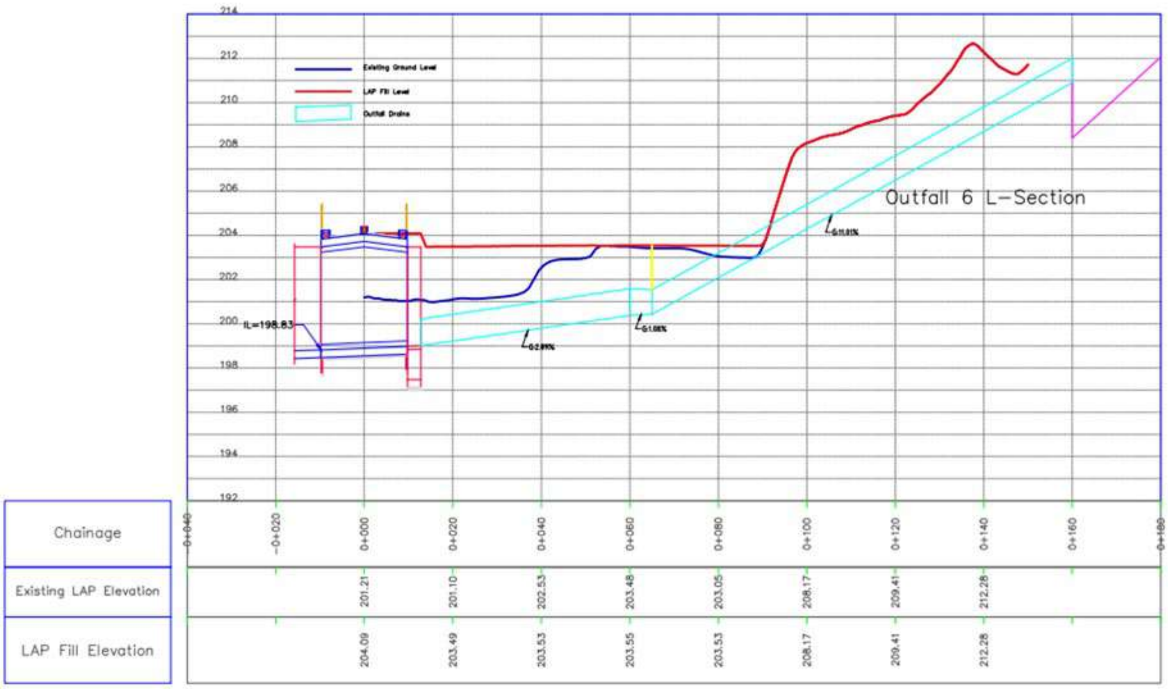

Figure 2-6: Longitudinal profile at 1-2m away from longitudinal section of Outfall 6.

As per PCR project team, the remaining height of the drain would be achieved by having stone pitching on embankment slope on their project scope. If they are to do stone pitching as per project scope it will require additional area on both side of outfall as shown in the Figure 2-7 below & the additional area require as follow;

a) At the gentle slope section of Outfall 1 & Outfall 2 it requires an additional 2.2m strip on average on the both sides of the drain to accommodate stone pitching.

b) At the gentle slope section of Outfall 3, Outfall 4, Outfall 5, Outfall 6 & Outfall 7 it requires additional 6m in average to accommodate stone pitching.

c) Similarly, on the steeper slope section of Outfall 3, Outfall 5, Outfall 6 & Outfall 7 it requires 4m strips on average to accommodate stone pitching.

The main issues are that we don't have area/space on both sides of Outfall 1, outfall 2, Outfall 4, Outfall 6 & Outfall 7. In case of Outfall 3 & Outfall 5 we have a drain buffer but it isn't enough to accommodate stone pitching. Therefore, it is recommended to have an RCC cantilever wall along these outfalls as it is robust & resist active earth pressure well as shown in the **Figure 2-8**.

While PCR team were commenting & suggesting that they could lift the invert level of Outfall such that the top of Trapezoidal will be at the same level of the LAP filling level but with that it will require additional filling in the LAP to solve the drainage issues. Therefore, we recommend not to raise the invert level of Outfall drains.

Moreover, we have informed & consulted with PCR team on the design invert level of the Outfall drains & they have agreed to make the necessary changes required in the invert level of the outfall drains. They did the necessary changes & have provided the final design invert level of the outfall drains & the LAP filling analysis to resolve the drainage issues was done according to it.

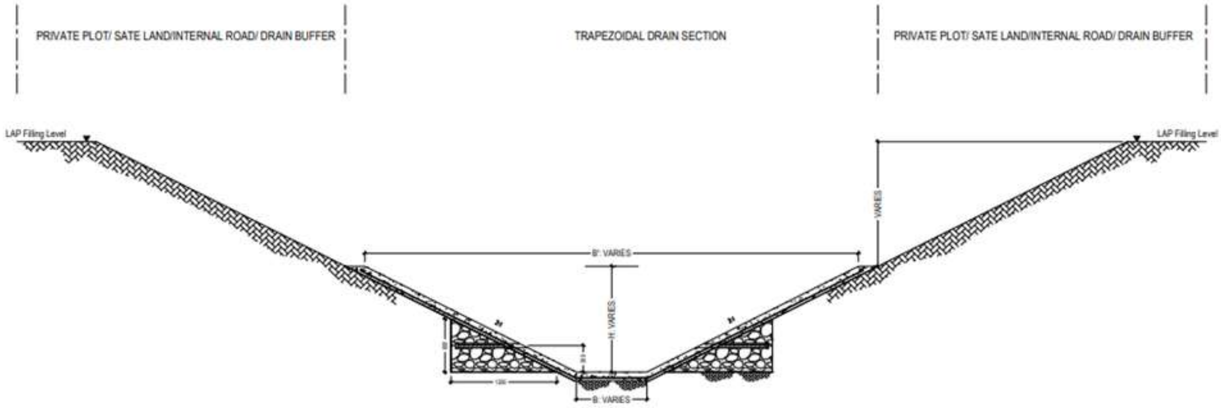

Figure 2-7: Typical Section of Trapezoidal Drain section showing extent of Embankment.

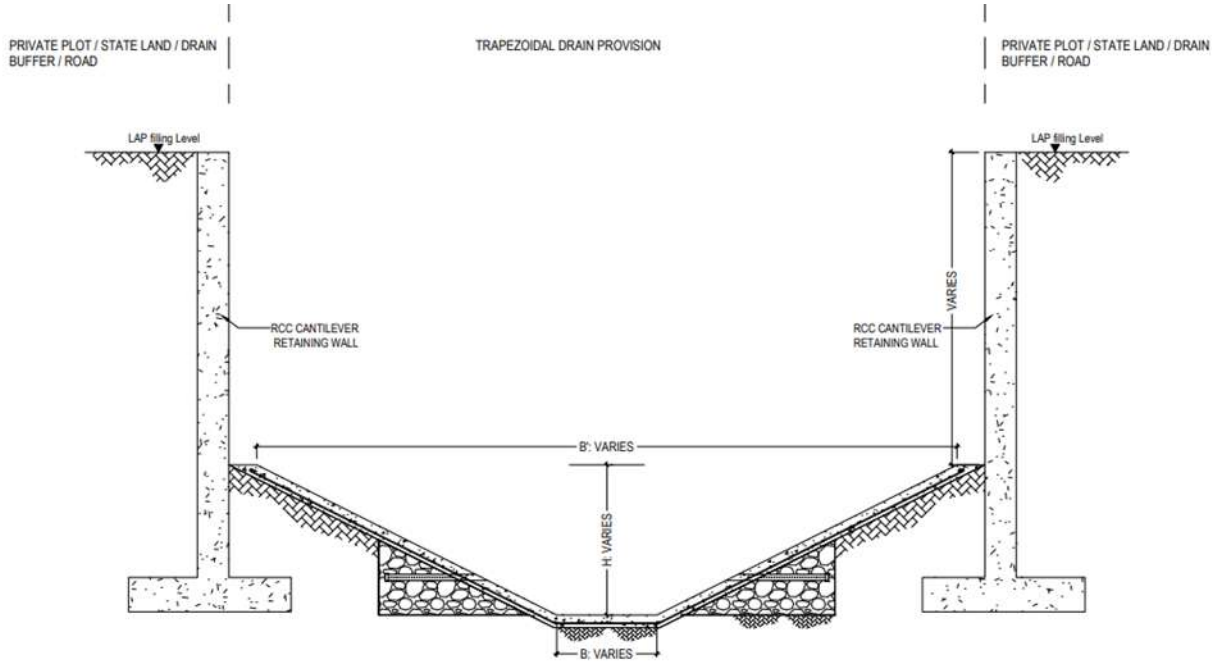

Figure 2-8: Typical Trapezoidal drain with recommended RCC Retaining wall on both sides.

2.2.2 Crossing

The PCR four-lane Highway project is under construction and currently they are constructing major Culverts & bridges along the PCR road alignment. They are almost done with construction of embankment of road and will begin the construction of Outfalls on east side of PCR, which fall in Ammochhu LAP, in the next phase beginning soon after this monsoon. There are eight outfalls and twenty-two number of crossings (culverts) along these outfalls within the LAP.

It is not possible to construct the crossings over the outfall in the LAP together with PCR outfalls drain since the Ammochhu LAP Urban Design & detail Infrastructure Design project is still in design stage.

Hence, provision should be kept at crossing while constructing PCR Outfalls drain, such that when these major crossings (culverts) are constructed in future, it should not affect the adjacent part of outfall drain. Whereas, the outfall, if constructed as per the present design, will incur wastage of resources in-terms of dismantling the drain already constructed while constructing the culverts at the crossing along the outfall. Therefore, it is recommended to review the existing outfall design and instead have an open channel U-shape cross section outfall drain like outfall in the PTPD project.

The following are the key findings for the review and recommended change in design for the outfall;

a) We can use the similar Outfall Drain design as Box Culvert crossing in the LAP. A thicker bottom slab and side wall for the section may be adopted and slab may be casted over these major crossings at a later date when Thromde administration have budget and decide to lay internal road.
b) The span of major crossing will decrease thereby reducing the significant cost of major crossing (Culvert/Bridge). It will be easier to construct and will not require specially designed connector structure to connect culvert & Outfall drain both upstream & downstream as it is the case in the present design, wherein firstly, the already constructed outfall will have to be dismantled at the outfall crossings and then the culverts will have to be constructed along with a special connector structure in order to connect the culvert with the trapezoidal shaped outfall.
c) A similar structural detail of PTDP Outfall could be adopted for PCR Outfall provided the outfall drain section should be able to accommodate the design flow from its catchment.
d) U-shape Outfall drains are robust, durable and require comparatively lesser top width.

2.2.3 Sediment Trap

The detailed design of Sediment trap was done previously by National Hydrologist under guidance of International Hydrologist Specialist under Detailed Design and Procurement Assistance for Phuentsholing-Chamkuna Road project. It is not recommended to review & revise the Sediment trap design as we don't have the required technical capacity to review & revise the design. However, having check dams may be also explored instead of sediment traps.

While coming to the practicality of implementation of the design sediment trap, we also did site visit & verification on 2nd April, 2022. All the concern points pointed out by the PCR team during the last meeting were pertinent & valid. The following were pertinent concern point pointed by PCR team;

i. The purpose & functionality of Line drain in Sediment trap of Outfall 2, Outfall 6 & Outfall 7 was being questioned as topography of area where sediment trap is supposed to be constructed for these outfalls are relatively steep. Even if they construct Sediment trap for these outfalls the purpose & functionality of Sediment trap won't be fulfilled because velocity of flow in Line drain will be high & will probably scour line drain itself & Culvert's abutment immediate to it.

ii. The team leader recommends Check-dam in place of Sediment trap in Outfall 2, Outfall 6 & Outfall 7 as Check-dam can perform the purpose & functionality of Sediment trap more effectively. Moreover, check-dam is easier & cheaper to construct.

iii. The Sediment Trap should be cleaned as soon as it gets full every monsoon season but all the Sediment traps fall beyond the LAP jurisdiction & there is no access road if the authority responsible has deployed the excavator to clean debris from Sediment Trap.

2.2.4 PROVISION ON DEBRIS CLEANING

As per PCR design documents, it has been mentioned that Outfall drain shall be cleaned periodically to avoid overflow into the LAP area especially during monsoon season. While carrying out the cleaning of debris from Outfall drain deployment of machine like wheeled-excavator having longer boom shall be required.

In the current design of the trapezoidal section, the bottom width of the drain varies from 0.4m to 2.0m & because of it we won't be able to accommodate machines inside the trapezoidal drain to clean debris. Moreover, from a structural point of view, 100mm RCC with nominal reinforcement won't be able to withstand the load of the machine-like Excavator. Hence, we have to provide provision along the Outfall to clean the debris in the outfall.

We can use internal service roads as provision along Outfalls to clean the debris especially in case of Outfall 1, Outfall 2, Outfall 3, Outfall 5 & Outfall 7. While in case of Outfall 4 & Outfall 6 it is not possible to provide provision because we don't have parallel internal service & area available along the outfall.

Other than providing provision for debris cleaning along the Outfall we can reduce the debris flow in the outfall by adopting bio-engineering techniques like slope stabilization in the upstream catchment of the outfall.

3 STORMWATER DRAINAGE

3.1 Introduction

Safe and reliable storm water drainage system is an enduring ambition of a society. Unmanaged and unplanned stormwater drainage is disastrous and ugly. Almost all urban centers in Bhutan have urban storm water drainages either along the roads or on their own. Moreover, Urban flooding in Bhutan is a common phenomenon, even in the capital city of Thimphu which is supposed to have better designs, budget and manpower. The matter is worse in the southern belt of the country such as Phuentsholing where the monsoon rainfall intensity is high.

Urban drainage includes two types of fluids viz waste water and storm water. The Storm water runoff is a major problem in Phuentsholing.

3.1.1 Existing Drainage in the LAP

Presently, LAP is sparely developed with few permanent buildings[5] & currently the land uses in the LAP are mostly warehouses which will have to go in the future due to contradicting land-use. Most of the permanent buildings are NHDCL[6] colony and few private buildings.

The existing drains in the LAP are as follows;

e) Plinth drains in the NHDCL Building Colony, Private Residential buildings and Warehouses.

f) Open drains at various location of the proposed Outfalls & few nearby residential buildings.

Most of the existing drains fall in the plots as shown in Figure 3-1 and the overall conditions is not satisfactory. There are blockages in drain due to sediments and trash and almost all the drains have missing links thereby flooding low-lying area which are within western & central avenues during monsoon season.

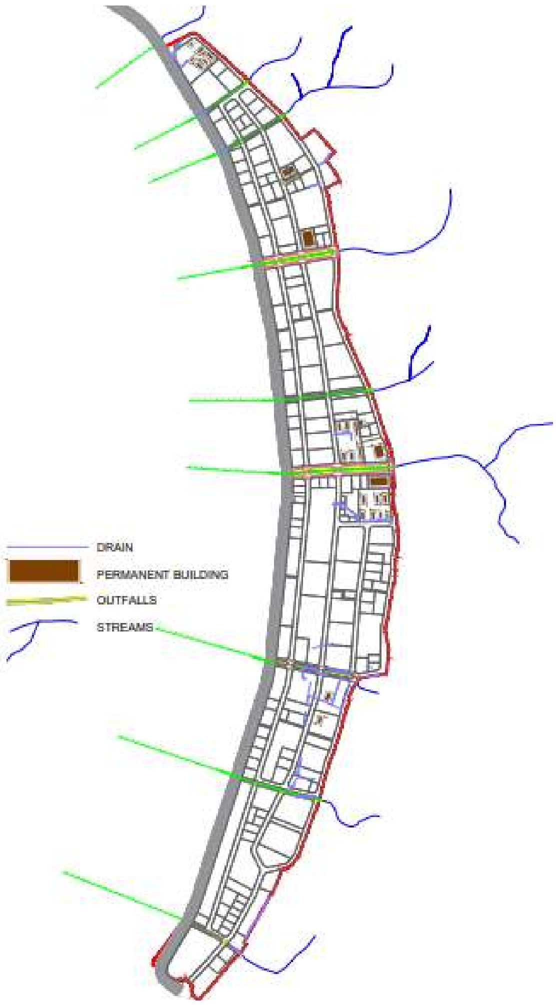

Figure 3-1: Existing drains in the Lap.

[5]Permanent Buildings are the building built within their respective plot boundary.

[6] National Housing Development Corporation Limited.

Figure 3-2 show existing drain cloated with sediment at the end of Outfall 1.

Figure 3-2: Existing Drain at end of Outfall 1.

Figure 3-3 show existing drains at end of Outfall 2.

Figure 3-3: Existing drain at end of Outfall 2.

Figure 3-4 show existing drain at end of outfall 2a.

Figure 3-4: Existing Drain at end of Outfall 2a.

Figure 3-5 show existing drains nearby NHDCL Building Colony.

Figure 3-5: Existing Drain nearby NDHCL Colony.

Figure 3-6 show existing drain along Outfall 3.

Figure 3-6: Existing Drain along Outfall 3.

All the existing drain along the proposed PCR outfalls will be replaced with RCC trapezoidal drain & U-shaped Covered drain which will be executed by PCR project. While others existing drains fall within the plots & has missing links and most of them are poorly managed. Hence, proper analysis & design of these Stormwater drainage will be carried out as per the scope of the project.

3.1.2 Existing Topography of the Site

The topography survey was done in November, 2021 as per the scope of project. The existing ground topography at site as follows & Figure 3-7 represent low-lying areas in the LAP;

i. The existing ground from Outfall 1 till Outfall 2a is relatively flat. However, within Western, Central & Eastern avenues, some major low-lying areas can be seen.

ii. The existing ground from Outfall 2a till Outfall 7 is relatively steep toward hill-side within Central & Eastern avenues, while existing ground is relatively flat and gentle between PCR Highway and Central avenues. However, within western & central avenues are major low-lying are identified.

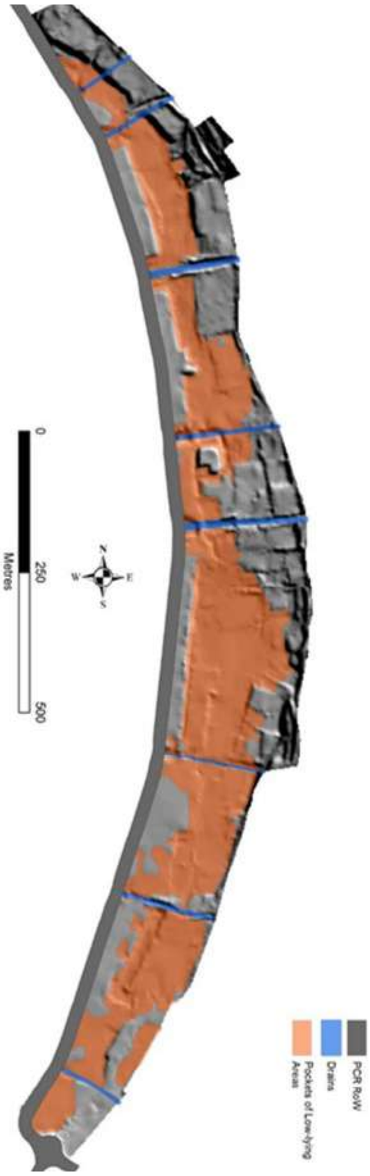

Figure 3-7: Low-lying area in the LAP.

iii. Few low-lying areas are also seen within Central avenues and parallel internal service road adjacent to it in-between Outfall 2a & Outfall 3. Similarly, few low-lying areas also identified within Central & Eastern avenues within Outfall 4 & Outfall 5.

Due to these major low-lying areas in the LAP, it was inevitable to solve Stormwater drainage in existing ground topography. Figure 3-8 represent Existing Topography in the LAP and Figure 3-9 represent Existing ground Topography showing invert level of PCR outfalls.

Other than Existing topography of the site, reviews of the on-going PCR project was important & necessary to integrate the LAP stormwater drainage. For the reviews, the consultants collected the necessary data likes Culvert/ Bridges drawings, Detailed Geo-technical Investigation, Hydrology and Drainage Report, Material Report and PCR Outfalls drawings from PCR project Official.

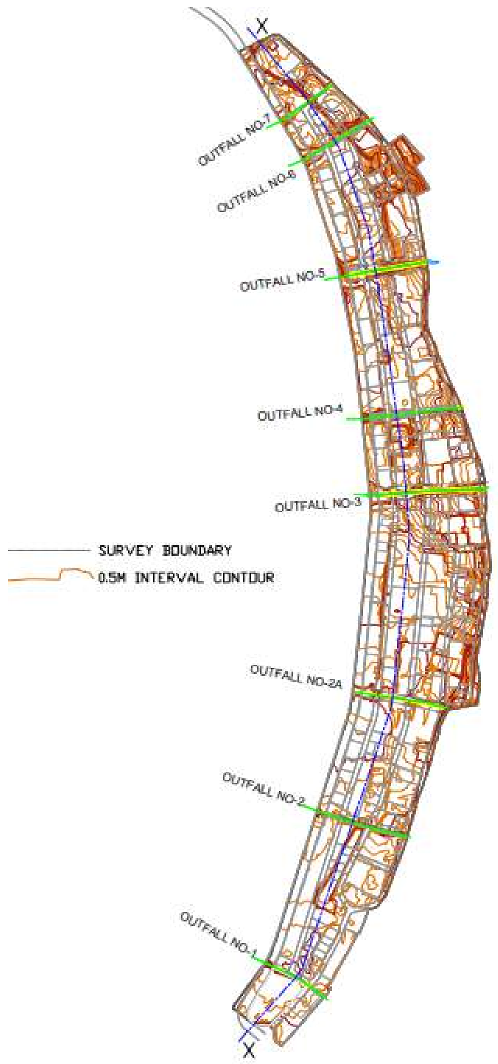

Figure 3-8: Existing Topography of the LAP.

Figure 3-9: Existing Topography Section X-X showing invert level of PCR Outfalls.

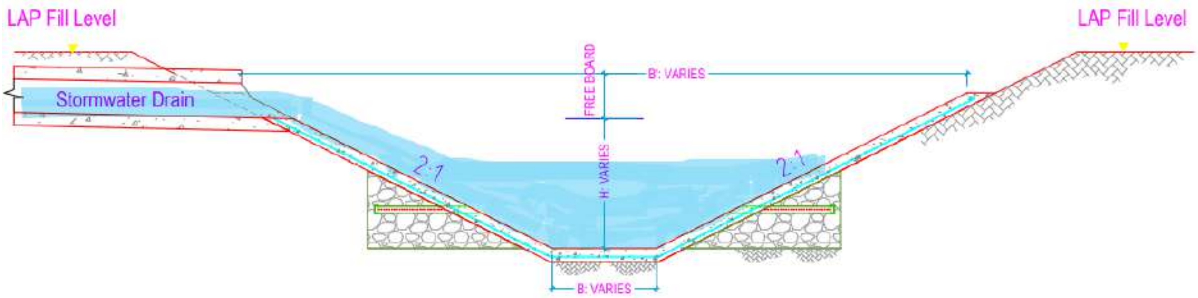

Figure 3-10: Typical Outlet connection detail at Outfall.

Eight of the twelve outfalls of the PCR project fall under Thromde Boundary on the east side of Phuentsholing Chamkuna Road (PCR). The slope of the drainage channel is steeper for the sections in the LAP, while the slope is lesser for the sections that are under the finished level of PTDP fill up level.

As per preliminary assessment of the Outfalls, the steeper section of drainage channel of the Outfalls was not posing difficulty in integrating LAP's internal stormwater drainage into the respective Outfalls as it was just the matter of deciding where to divert & discharge the run-off by taking design invert level of Outfalls into consideration.

While in the gentle slope section of channel of Outfall, following observations are made;

a. The design invert levels of Outfall 1 & Outfall 2a lies below the existing ground but no significant (approximately 2m) difference between the invert level of outfalls & the existing ground.

b. The design invert levels of Outfall 2a lies at same level & follow existing ground profile.

c. The invert levels of Outfall 3, Outfall 4, Outfall 5, Outfall 6 & Outfall 7 lies below the existing ground level but the difference between existing ground levels and invert levels of Outfalls are approximately 3m.

However, to integrate LAP internal Stormwater drainage into the Outfall the Consultant suggested to place the outlets of internal stormwater drainage within freeboard of Outfall drain as shown in Figure 3-10 to prevent back-flow, debris flow & sedimentation into the internal stormwater drains. Taking into account the Outfall's overall height and patches of low-lying areas in the LAP it was inevitable to integrate the LAP internal stormwater drainage into the Outfalls without the LAP filling.

Hence during the concept stage, the detailed analysis of three different options of filling were done & presented. The detailed on the filling analysis to resolve the Stormwater drainage in the LAP already discuss on LAP Filling Analysis chapter and for detail refer that chapter.

In the LAP filling analysis for Option 1 & Option 2, the Stormwater drainage network analysis was done considering that only storm water drainage will come in the RoW (Right of way) of Internal roads. Now the final analysis of internal Stormwater drainage network in the LAP will done keeping in consideration that others utilities will also come along with it in RoW of internal roads. Moreover, the analysis will be done on the proposed LAP fill level.

3.2 Detailed Methodology for Stormwater Management

The detailed methodology on the Stormwater management in the LAP is as shown in Figure 3-11.

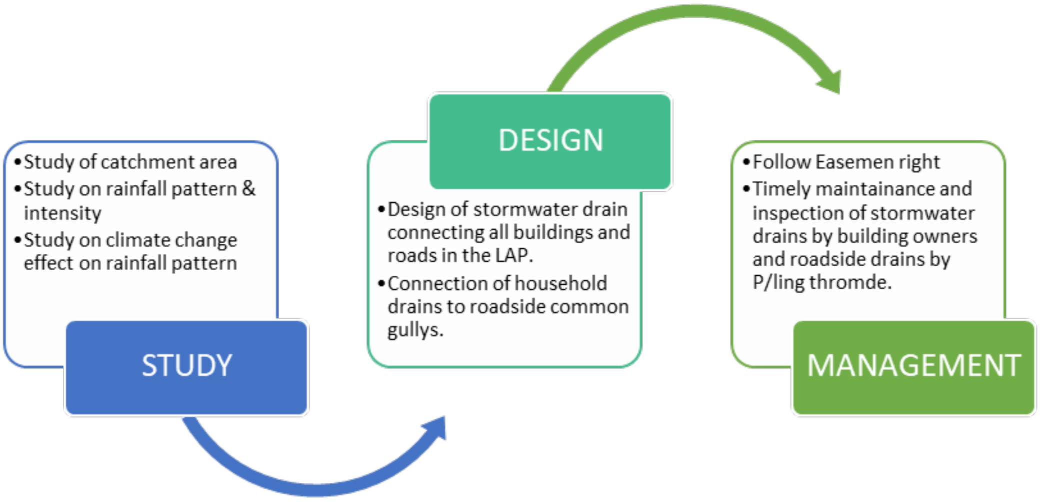

Figure 3-11: Methodology for Stormwater Management.

3.3 Study of Rainfall Pattern & Intensity, Catchment and Climate Change Effect on Rainfall Pattern

3.3.1 Rainfall Intensity and Pattern

Rainfall data from the Phuentsholing Meteorological station were collected and analyzed for this study. Data are available from the year 2008-2018. The details on the rainfall station and the duration of missing data are listed in Table 3-1.

Table 3-1: Details on rainfall station and missing data.

Data frequency	Daily	Missing data duration[7]	
Data duration	2008 – 2018	1st Aug – 24th Sept 2012	8th Jan 2016
Altitude	220 masl	1st Nov -31st Dec 2012	28th Feb – 29th Feb 2016
Latitude	26:86	31st Oct 2013	31st March 2016
Longitude	89:39	31st Aug 2013	1st – 2nd June 2016
Source	NCHM	1st Feb 2014–8th Aug 2015	28th May 2017

[7] The rainfall values are given -99.9 on the highlighted dates in the table below; these values are considered null while plotting the graphs.

a) DAILY RAINFALL PATTERN FROM 2008-2018

Figure 3-12 shows the daily rainfall pattern from the year 2008 – 2018. During this period, 11th August, 2017 received the highest rainfall, a rainfall of 285 mm in a day and in the year 2018 received the highest annual total annual rainfall of 6146 mm.

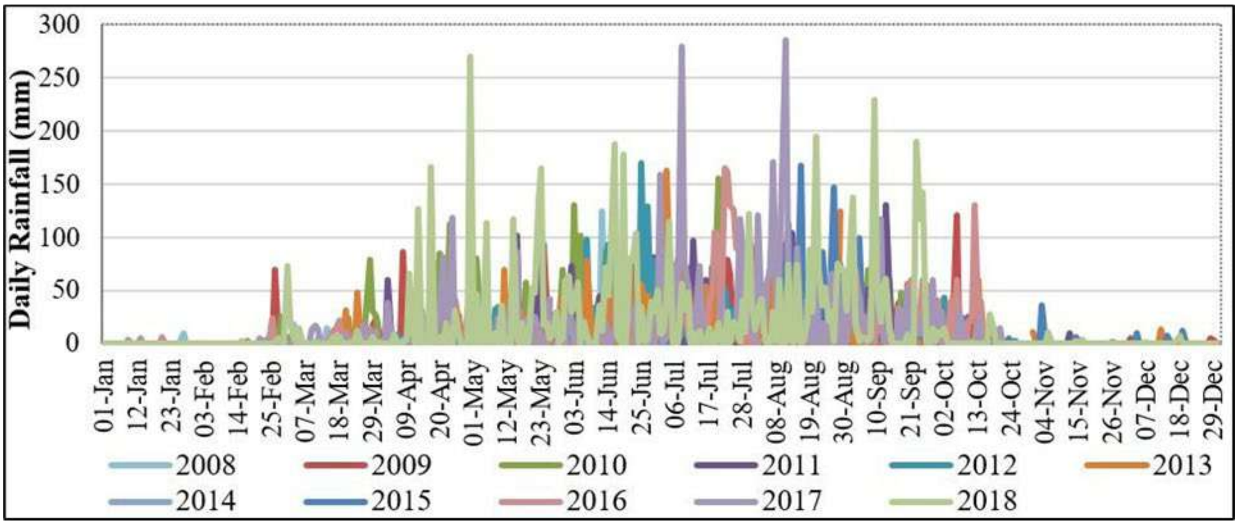

Figure 3-12: Daily rainfall pattern from 2008 – 2018

b) MONTHLY TOTAL RAINFALL

The months of June, July, August and September receives around 70% of the annual rainfall, while the minimum is in the months of January, February, November and December. The details are shown in the Figure 3-13.

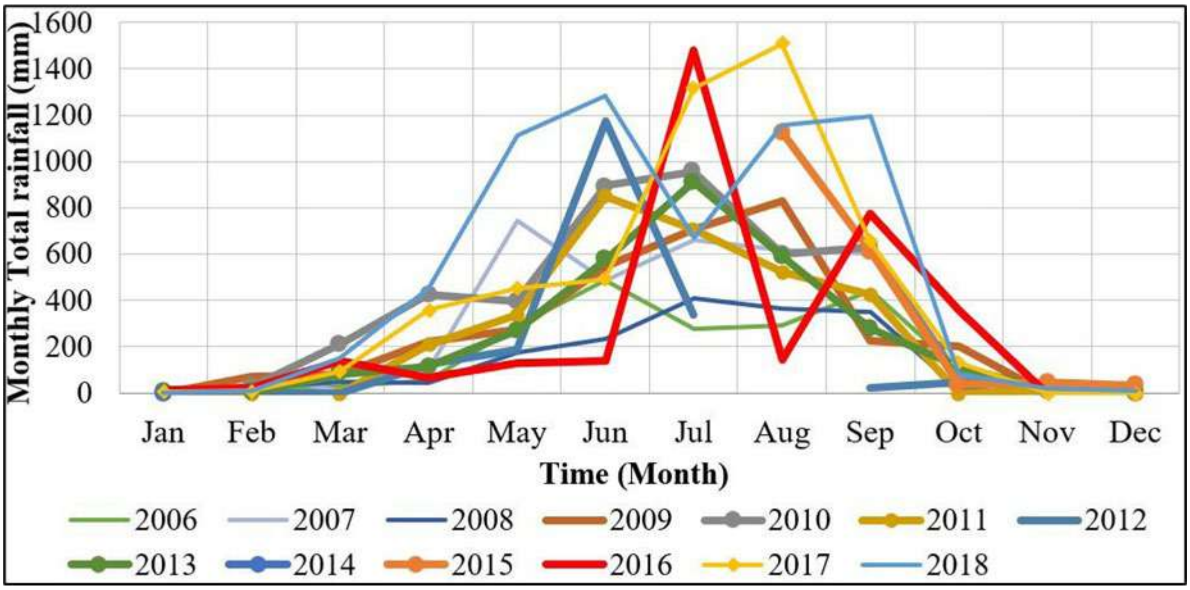

Figure 3-13: Monthly total rainfall from 2006 – 2018

3.3.2 Study of Catchment Area

A Digital Elevation Model (DEM) is a representation of continuous elevation values over a topographic surface; it will be used for terrain processing, watershed and drainage delineation by pour point using Arc Map 10.5 software, also known as Geographic Information System (GIS) for this study.

ArcGIS can delineate the total area flowing into a given outlet, also called a pour point based on a digital elevation model. In this analysis, the pour point will be the stream gage. ArcGIS can compute from the DEM the direction of flow down slope and how many cells flow into each cell (called flow accumulations).

The land cover of the whole catchment and parameters such as basin slope, drainage area and drainage length of each small basin has been studied in detail for the design stormwater drains. The highest rainfall intensity recorded at the study area was taken to design the storm water drainage. The climate change +effect in rainfall pattern and its intensity was also considered while calculating the quantity of storm water runoff at pour point of each basin.

3.3.3 Climate Change Assessment

The hydrological assessment of the CC Vulnerability Assessment project[8] carried out a statistical analysis of the hydrology for the Southern Belt of Bhutan. This analysis yields the following rainfall depth values at different durations and return periods as shown in Table 3-2.

Table 3-2: Design rainfall intensity at different durations and return periods under climate change conditions in 2050.

DESIGN RAINFALL INTENSITY for 2050 under CC conditions						
ARI, T years	2-year	5-year	10-year	20-year	50-year	100-year
Design Rainfall, by Gumbel	240	274	299	321	351	373
EV I, mm/day						
Duration, D hours	Rainfall for T-year return period,			mm		
1-hour	66.0	75.6	82.8	88.8	97.2	103.2
2-hour	91.2	104.4	114.0	122.4	134.4	142.8
3-hour	110.4	126.0	138.0	148.8	162.0	171.6
6-hour	152.4	174.0	189.6	204.0	223.2	236.4
12-hour	204.0	232.8	253.2	272.4	297.6	316.8
24-hour	240.0	273.6	298.8	321.6	350.4	373.2

[8] Enabling Climate Change Responses in Asia and the Pacific-Building Resilience to Disaster and Climate Change Impacts (Subproject 2), Climate Change Vulnerability Assessment and Adaptations, June 2013.

The International Centre for Integrated Mountain Development (ICIMOD) has made an assessment on the Climate Change impacts in Bhutan. This assessment expects for 2050 an increase in temperature being 2.5 °± 0.4° C. The hydrological study of the earlier mentioned project also mentions that the extreme rainfall should be adjusted accordingly with the temperature rise. A percentage adjustment of 8 % increase per 1°C should be applied in case of a 50 years return period.

These findings from the past study are used to determine the design storm water drainage system considering the impact of climate change on rainfall pattern and temperature. Based on Table 6 above, the 50 and 100 years return period rainfall intensity for Phuentsholing is 116.64mm and 123.84 mm for 1-hour duration respectively. To take into consideration of the climate change impact 20 % increment are necessary.

3.4 Analysis

The objective of the current work as per scope of project is to analyze the stormwater drainage and to recommended the stormwater management options for the LAP.

Overall analysis method adopted is shown schematically in Figure 3-14. The activities carried to achieve the analysis are as indicated below.

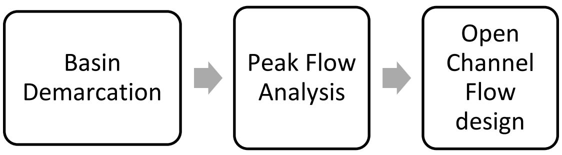

Figure 3-14: Flow Diagram for Analysis.

a) Basin delineation, land use and characterization of physical parameter.

b) Peak flow analysis using rational method.

c) Preparation and development of the proposed design of the stormwater drains.

3.4.1 BASIN

The steps for watershed delineation in ArcGIS are shown below.

Figure 3-15: Steps for watershed delineation.

The LAP is relatively flat, the stormwater drainage will run along the internal service road underneath the footpath. Hence, watershed analysis wasn't necessary for the internal stormwater drainage.

While toward hill-side beyond the LAP boundary, the existing natural topography is steeper. It was crucial to determine where run-off discharge from these areas will flow.

Although PCR Outfall drains will cater to run-off discharge from LAP & its catchment. The run-off discharge from adjacent hill-side area will flow into the eastern avenues/eastern internal service roads. Hence, to determine watershed that would be contributing the run-off discharge ArcGIS was used.

We used the spatial analysis hydrology tool in GIS to delineate the watershed and flow accumulation network from the ALOS Mosaic. Using the DEM as the starting point for the analysis, several steps were needed to convert the elevation data into flow direction and accumulation data, and finally into streams and watersheds as depicted in the Figure 2-15. The specific tools needed for the surface runoff analysis are found in the Hydrology tools within the Spatial Analyst toolbox extension.

Figure 3-16 represent the watershed delineation.

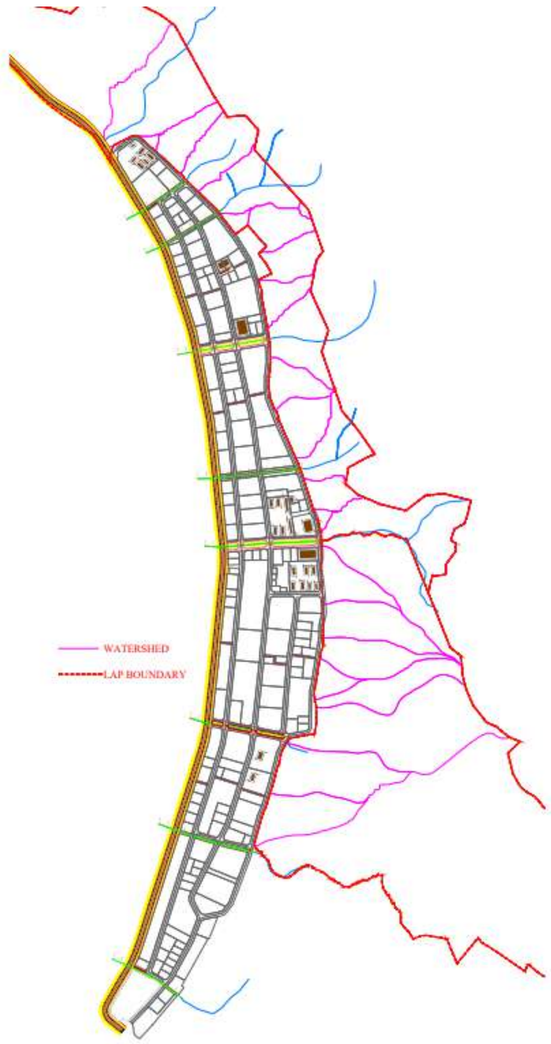

Figure 3-16: Watershed Delineation.

3.4.2 PEAK FLOW

Majority of Urban storm drainage systems are designed based on the Rational Method. As much as 90% cases across the globe, despite having several limitations, uses Rational Method. This empirical formula relates the Peak discharge or runoff to drainage area, the rainfall intensity and runoff coefficient. The equation is;

$$Q = 0.028PAI$$

Where;

Q=Design peak discharge, m^3/s.

P=Coefficient of run-off for catchment characteristics.

A=Area of catchment in hectares.

I=Rainfall Intensity in cm/hr. for the design return period and for duration equal to 'Time of Concentration' of watershed.

The coefficient of runoff (P) is the portion of precipitation that makes its way to the drain. Its value depends on a large number of factors such as permeability of the surface, type of ground cover, shape and size of catchment area, the topography, the geology, initial state of wetness and duration of storm.

The run-off coefficient used in determination of peak flow is given in Table 3-3 & Table 3-4.

The rainfall intensity, "I" in mm/hr. relates to the design return period and for duration equal to the "time of concentration" of the watershed. In order to design the design storm intensity for known duration and return period, some type of intensity-duration-frequency (IDF) data for the location of interest is necessary. The Rational Method is applicable to rural catchments up to 25 km^2 but limited to 1 km^2 in case of urban catchments due to complexity of land use.

The "I" value is considered from the for the 50 years and 100 years with 20% increment. The rainfall intensities are 116.64 and 123.84 mm per hour respectively.

Table 3-3: Values of Coefficient of Run-off.

SL. No	Description of Surface	Coefficient of Run-off(p)
1.0	Watertight pavement surface (Concrete or bitumen), steep bare rock	0.90
2.0	Green area (Loamy)	0.30
3.0	Green area (Sandy)	0.20
4.0	Unpaved area along roads	0.30
5.0	Lawns and Parks	0.15
6.0	Flat built-up area with 60 percent area impervious	0.55
7.0	Moderately steep built-up area with about 70 percent area impervious	0.80

Table 3-4: Coefficient of Run-off for various surfaces.

SL. No	Description of Surface	Coefficient of Run-off(p)
1.0	Mostly densely built-up area	0.7-0.90
2.0	For adjoining area to adjacent to built-up area	0.5-0.70
3.0	Residential areas	0.25-0.50
4.0	Sub-Urban areas with few-building	0.10-0.25

The Tc = time of concentration – time required for water to flow from the most remote point of the area once the soil has become saturated and minor depressions filled. It is assumed that when the duration of the rainstorm equals the time of concentration, all parts of the watershed are contributing simultaneously to the discharge at the outlet.

Out of many methods used to estimate Tc, one of the simplest and most widely used is the Kirpich equation. The equation is;

$$T_C = 0.0195\frac{L^{0.77}}{S^{0.385}}$$

Where;

L=The distance from the most remote point to the outlet in m.

S=Slope of the Catchment Area.

Based on the rational method of analysis, the total peak flow generated from the watershed will be known which will be used in hydraulic design of drains.

3.4.3 Hydraulic Design of Drain

The Stormwater drains section are usually design as open channel. Owing to its simplicity and acceptable degree of accuracy in a variety of practical application, Manning's formula is valid for turbulent flow which is the most widely used for uniform flow for designing stormwater pipe conduits and channels.

The capacity of the drain is normally designed using Manning's formula. The Manning formula is given below;

$$Q = 1/n\, AR^{2/3}S^{1/2}$$

$$V = 1/n\, R^{2/3}S^{1/2}$$

Where;

Q= Discharge in m^3/s.

V= Mean Velocity of flow in m/s.

n=Manning's roughness coefficient.

R=Hydraulic radius in m, which is area of flow cross section divided by the wetted perimeter.

S=Eenergy slope of the channel, which is roughly taken as slope of drain bed.

A= Area of the flow cross-section in m^2.

The Manning's roughness coefficient for the design of storm was referred from Table 7.1 of IRC SP:42-2014.

a) Minimum Velocity

While deciding the drain sections, it is not enough that they are sufficient to carry the required discharge. To ensure self-cleaning of the drain, a minimum velocity of 1.5m per second may be desirable. However, this may require the installation of concrete drains.

In the LAP the minimum velocity of 1.5 m/sec was maintained in the major secondary drains which caters to run-off discharge from plots & road pavement.

b) Minimum Freeboard

The minimum freeboard in the drainage shall be as per Table 3-5. The minimum freeboard specified in the table was used in design of drain sections.

Table 3-5: Minimum freeboard.

Drain Size	Freeboard
Beyond 300 mm bed width	10cm
Beyond 300mm & up to 900mm bed width	15cm
Beyond 900 mm & up to 1500 mm bed width	30cm

c) Channel Shapes

The usual channel shapes are;

i. Parabolic.

ii. Trapezoidal.

iii. Rectangular.

iv. Triangular or V shaped.

The parabolic profile is considered to be the best for hydraulic flow but its actual construction and maintenance is difficult. The V shaped drain is not very popular in urban areas as its desilting is difficult. The trapezoidal and rectangular sections are easy to construct and are considered most suitable.

In urban area all drains passing through built-up area and near to bus stand, crossing etc. should preferably be covered so that the urban drains are not used as dustbins.

In the LAP, the stormwater drains are proposed along the internal service roads. The RoW of internal service road is 10m, which consist of 6.6m carriageway, 1.9m & 1.5m footpaths on sides. Due to limited space within footpath which will have to cater to other utility services, it was impractical to adopt trapezoidal section of drain. Hence rectangular section storm water drain was proposed in the LAP.

d) Economical Section

As far as possible, for obtaining economical sections, the relation between bed width and depth shall be as follows;

i. Rectangular, b=2d.

ii. Trapezoidal, b=0.82d (1:1 side slope) & b=1.24d (1/2:1 side slope).

3.5 Design Concept

Various design options and materials are available for design of drains and associated structures. We are selecting the most economical and technically robust designs and materials for the LAP. The suggested designs and materials are in following sections.

3.5.1 Bottom Shape

An overall recommendation for all interventions for hydraulic structures either for repair or new construction is to have 'U' shape drains and the intersections that are less than 90 degree or at least with some curves on the sides or round edges.

These structures will have lower friction and lead to lower sedimentation, scouring and lower chances of overflowing during intense rainfall. U shape drains will be easier to clean & maintain.

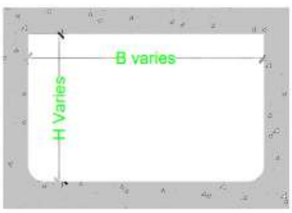

Figure 3-17: 'U' shape Drain cross-section.

3.5.2 Drain Sections

For the economic reasons, the perimeter of channel cross-section should be minimized. Theoretically, a semi-circular shape provides the maximum hydraulic capacity for the minimum channel perimeter. A trapezoidal section which is a pragmatic approximation to the semi-circular shape is often adopted because it is easier and cheaper to construct, and can accommodate a wider range of flows than the simple rectangular channel.

In the LAP, there are limited space allocated for footpath in RoW of internal service roads. Due to limited space underneath footpath which will cater to other utility services, the option of having rectangular drain with covers was considered in the view of easy constructions, maintenance and depth of the invert. However, at road crossings RCC Hume pipe will be provided.

3.5.3 Material of Drain

There are several materials used for the construction of drain. Some of the most common materials used are as follows;

a. Random Rubble masonry (RRM).

b. Reinforced Cement Concrete (RCC).

c. Plain Cement Concrete (PCC).

d. Pipe system.

For the LAP's Stormwater drainage, the Reinforced Cement Concrete material was considered for the drain due to the following reasons;

i. RCC drains can meet the needs of a variety of loading conditions. It is strong, ever-lasting and causes few issues. The structural strength of

RCC drains offers stability over time that may not be found in other drain materials.

ii. The RCC drains are incredibly durable. It won't buckle or wear and are resistant to corrosions.

iii. Hydraulic performance is determined by the structure's ability to allow safe and regular flow of water. Reinforced Cement Concrete will never lose its original shape unless it's cracked and broken. This allows for improved hydraulic efficiency as the resistance to water's flow is minimal and unchanging.

3.6 Proposed Drainage

3.6.1 THE PROPOSED DRAINAGE NETWORK

The proposed Stormwater drainage are placed underneath footpath along road roads. All the avenue roads have RoW of 10m of which 3.4m is allocated to footpath in case of Central & Eastern Avenue and 3m in case of Western Avenue.

Due to this it was not possible to place Service Utility Duct and Strom-Water drain together underneath same footpath. So, the Strom-water drainage was place strategically along the avenue roads as stated below;

- The major stormwater drainage was placed on right-side of RoW of Eastern Avenue as it will be convenient to intercept the run-off from hill-side of back-side Dhamdara and Chamkuna area.
- Similarly, the major drainage along central avenue is place on right side of RoW of road as the plots between Central & eastern Avenue are gently sloping toward Central Avenue and moreover the central avenue road lies significantly below eastern below.
- In case of Western Avenue, the stormwater drainage was placed underneath footpath on both side of RoW due to following reason;
  i. PCR four-lane Highway lies 600mm above LAP fill level and
  ii. Left-side footpath of Central Avenue was used for the Service Utility Duct.

Wherever it was not practical to discharge run-off into the Outfall along that particular Avenue Road, the proposed footpath in-between plots were used to divert into adjacent drain below. All these situations are explained in detail below;

- In case of Stormwater drainage along Central Avenue in-between the Outfall 5 & Outfall 6, the drainage has to be diverted to drainage on Western Avenue due to the following reason as stated below;
  i. The lowest elevation on the proposed LAP filling Level was in the middle & it was not possible to discharge run-off to either Outfall 5 & Outfall 6.
  ii. While taking the drainage outlet to Outfall, the invert level of drainage outlet

coming almost at same as invert level of outfall 6.

- In case of stormwater drainage along Central Avenue in between Outfall 4 & outfall 5, the drainage has to diverted to the adjacent drainage on Western Avenue as the existing lowest level was found at chainage of +1630m of road R1. It was not possible to divert the discharge to either Outfall 4 & outfall 5.
- In case of stormwater drainage along Eastern Avenue in-between Outfall 2a & Outfall 3, the drainage was diverted to adjacent drainage on Central Avenue as the sag point was observed at chainage +1150m of Road R1. It was not practical to discharge run-off either to Outfall 3 & Oufall2a.

In the road R4, while maintaining the desired gradient in road the LAP filling at the design road R4 well was necessary to enable better property connections which are on the right side of road R4. LAP filling should be in bench to solve drainage there.

The proposed stormwater drainage network is shown in Figure 3-18. In this proposed drainage network, the following nomenclature has been considered;

- D: Major Secondary Drain.
- RD: Small Roadside Drain.

The major drain D type will collect run-off discharge from both road pavement and adjacent plots or land. While RD type will collect run-off discharge from a part of road pavement & footpath and run along the road with the Service Utility Duct.

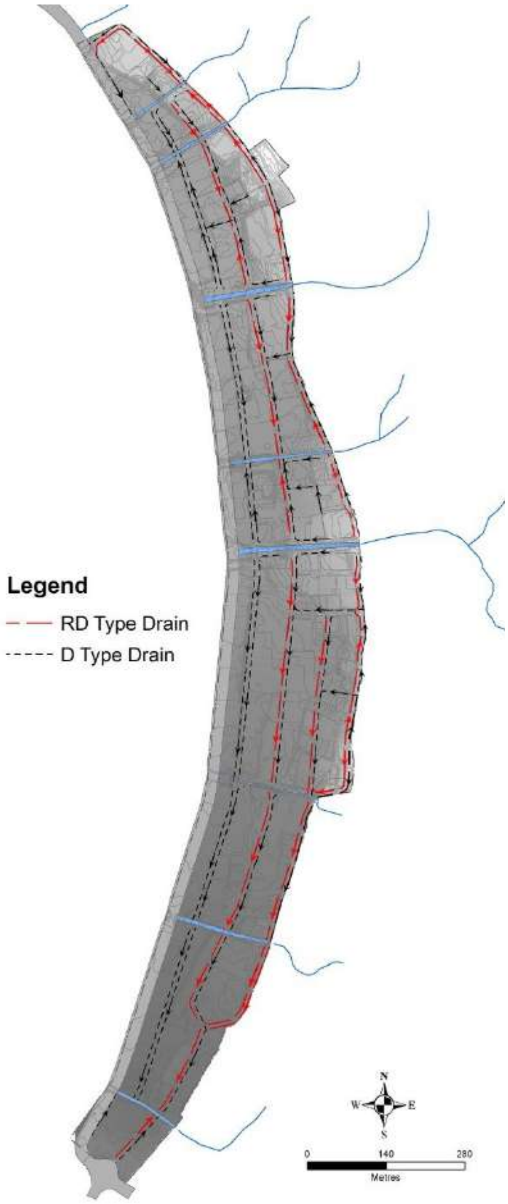

Figure 3-18: Proposed Stormwater drainage network.

3.6.2 PROPOSED DRAINS

The proposed RD type drain the cross section is shown in the Figure 3-19.The width of RD type drain is 350mm and height is 275mm. The details of RD type drains are given in Annexure A.

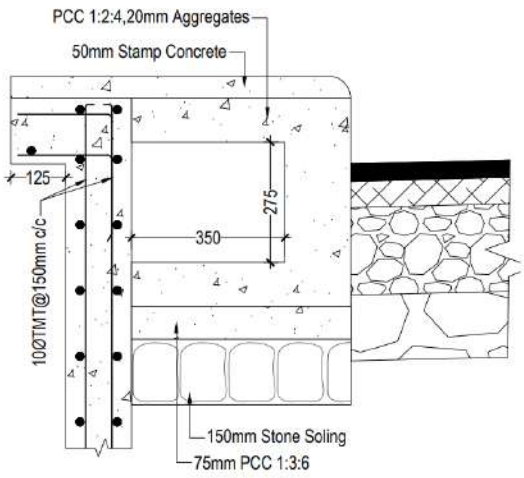

Figure 3-19:Typical RD type Drain along road.

The major D type drain cross section is shown in the Figure 3-20. The width of D type drains ranges from 400mm to 1100mm and height from 300mm to 850mm. The details of D type drains are given in Annexure B & structural details should be referred from structural drawings.

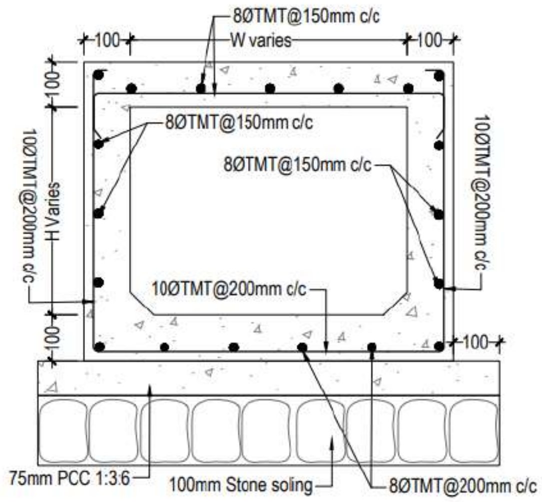

Figure 3-20: Typical D type drain section.

The longitudinal profiles of drains are given in the stormwater technical drawings. While at road crossings the stormwater drain shall be replaced with equivalent size of NP3 Hume pipe which can accommodate peak discharge of that particular drain. The sizes of NP3 Hume pipes are given Annexure C.

3.6.3 MANHOLE

The manholes shall be constructed at every chainage of 5m interval except for those chainages which fall on the road pavement. If chainage falls right next to footpath, then for that manhole should be consider on footpath. The spacing of manholes at every 5m interval is considered from inspection & maintenance point of view considering the climatic condition.

The typical section of RD type drain's manhole along Eastern & Western Avenue is shown in the Figure 3-21 below.

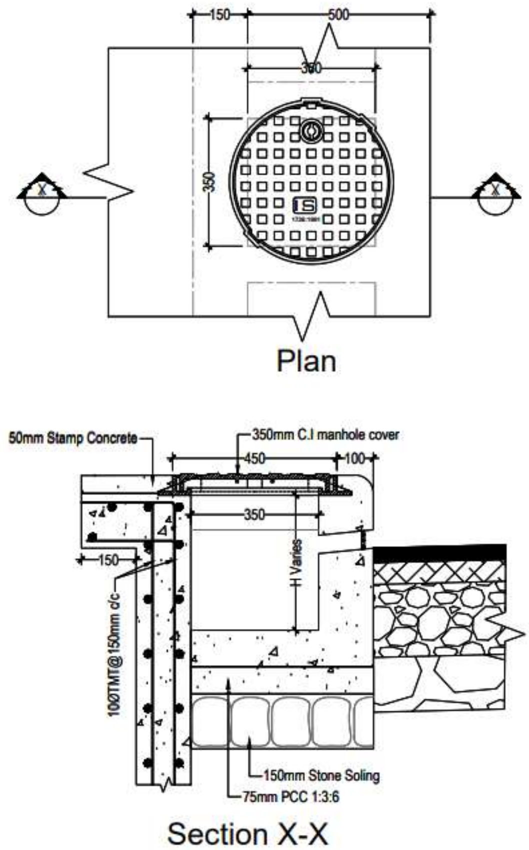

Figure 3-21: Typical RD type Drain's Manhole.

For the Central Avenue in Cobble Street, drain RD type drain's manhole is shown in the Figure 3-22 below.

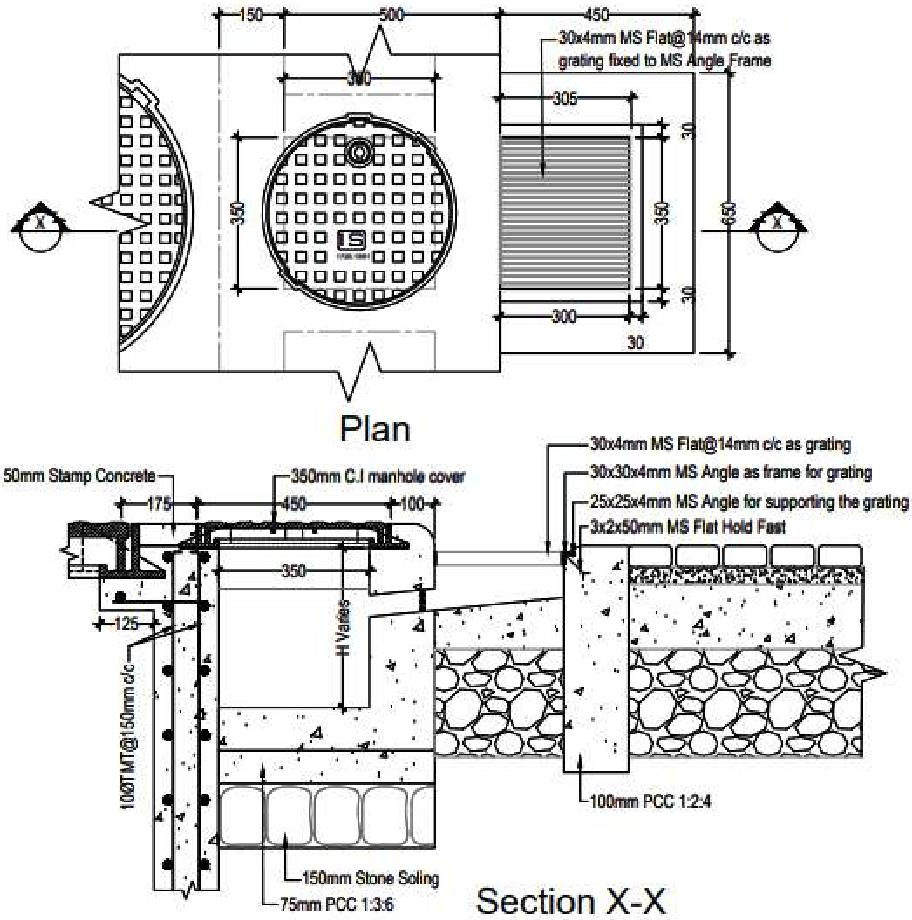

Figure 3-22: Typical RD type drain's manhole in Cobble Street.

The Manhole of Major D type drain are of two types. For the drain width raging from 300mm to 850mm, the size of Manhole will be 1.2mx1.150m and for width ranging from 950mm to 1100mm the size of the Manhole will be 1.2mx1.4m.

The height of Manhole varies as per the design road profile and the height of Manhole will be calculated as difference between the footpath level and the design invert level of drain. The footpath level shall be taken as the design level of road minus the cross fall plus the height of footpath above road.

The height of Manhole varies from 0.4m to 2.2m and accordingly the reinforcement details in shear wall differ. Refer the stormwater technical drawings for all these details.

In case of Manhole's height greater than 1.5m, the ladder made up of 20mm TMT bars at regular interval of 150mm embedded in shear wall shall be provided for easy access for inspection and maintenance during the monsoon seasons.

Figure 3-23, Figure 3-24 & Figure 3-25 shows typical D type stormwater drain's Manhole.

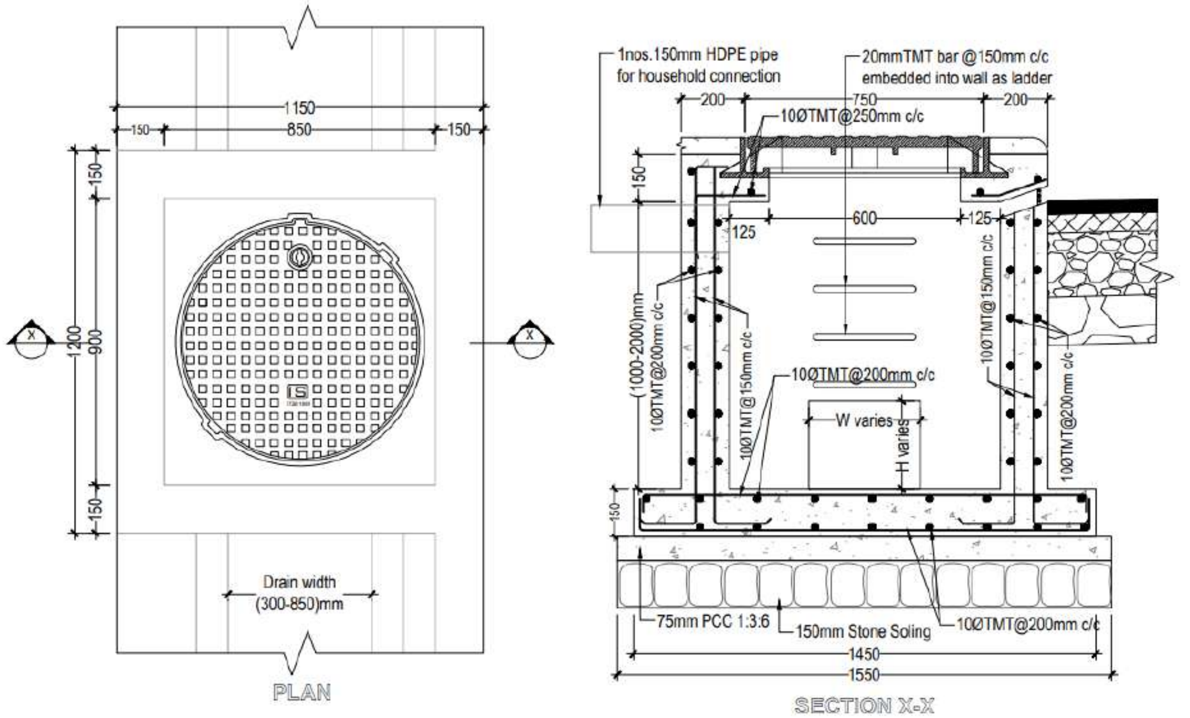

Figure 3-23: Typical D type Drain's Manhole details.

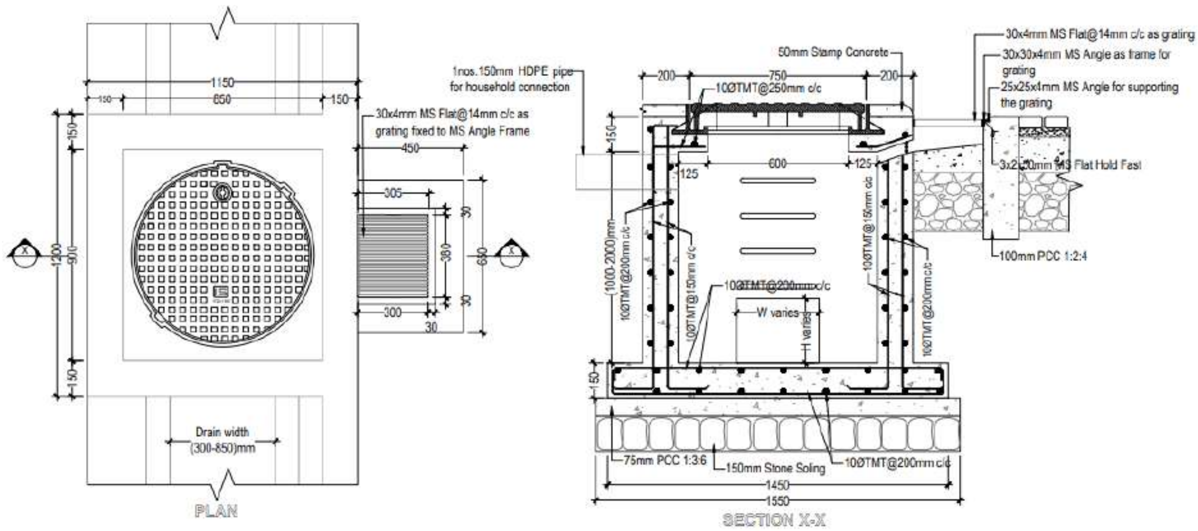

Figure 3-24: Typical D Type Drain Manhole Detail.

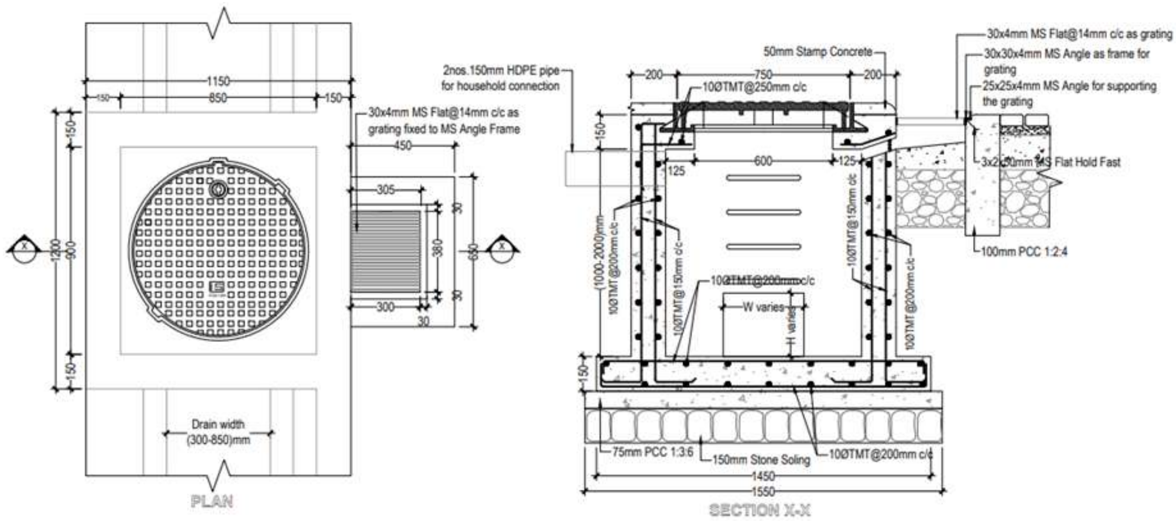

Figure 3-25: Typical Major D Type Drain's Manhole detail in Cobble Street.

3.6.4 HOUSEHOLD DRAIN CONNECTIONS

Run-off discharge from building in the plot shall be drain-off to the nearest Manhole. It will be the responsibility of the plot owner to construct the plinth drain from their plot till the nearest Manhole to drain-off run-off discharge from their respective plot into the Manhole.

In the Manhole, a 160mm HDPE pipe has been kept as provision for household connections. This provision shall be kept at every third manhole (every 15m interval) or the nearest manhole in the plot.

3.7 Stormwater Implementation plan

The implementation of stormwater structures cannot happen overnight nor it is recommended, as the development of the LAP is not yet started.

Almost all the proposed stormwater drains are along roads underneath footpath. Hence, the stormwater drainage package should be implemented together with roads & Service Utility Duct package as and when Thromde have the budget.

There are two drains namely D1 & D32 which runs along plot boundary of two or more plots. These drains are specially placed there to serve those private plots and should be constructed as an when building is constructed in any of those plots.

3.8 Stormwater Management operations

There are few measures important for Stormwater management to ensure proper functioning and longevity of the structures.

3.8.1 MONTHLY INSPECTION FOR DAMAGED OR CLOGGED CATCH PIT GRATES

Drains and catch pit grates shall be inspected and cleared of debris to maintain inlet capacity. Use of laying stormwater drains for laying water supply pipes should be checked and avoided.

3.8.2 ANNUAL CLEANING OF MANHOLES

Drain and its manhole structures shall be inspected and sediment should be cleaned annually or as and when required to maintain adequate functionality of the stormwater conveyance system. All sediments shall be properly handled and disposed of in accordance with environment guidelines and regulations.

3.8.3 Regular Monitoring by designated officer

It is the Thromde engineer/office's responsibility to ensure that drains are operating properly and the issues in the drainage should be resolve on time. For this he/she should depute pre-identified sweepers and caretakers for each area of the LAP. Similarly, the officer/engineer should submit the monthly status report of the cleanliness and functionality of all the infrastructure under his/her jurisdiction including drains to the Thromde management. If anything is wrong or not functioning, he/she should be made answerable.

3.8.4 Compliance Monitoring

Thromde Environment Officer or MoWHS Environment Unit and NEC have the authority to conduct compliance monitoring to ensure that requirements that have been specified in the environment terms are being met.

3.8.5 Budget Provision

For all of the above to happen as planned, it is also necessary for Thromde management to provide enough budget for operation and maintenance on a timely manner. Gelephu Thromde has recruited private entity to clean and maintain the drains and roads in the LAPs. Such examples can be replicated.

3.9 Bill of Quantities and Estimate

The total cost of implementing the Ammochhu LAP's stormwater drainage is Nu.102,271,375.62. The detailed Bill of quantities and cost estimate of Civil works are provided in excel to the Thromde as project deliberates.

The summary of the breakdown of the cost is given in the Table 3-6.

Table 3-6: Summary of the Cost.

Items	Cost (Nu.)
Earthwork	7,391,797.34
PCC works	6,736,481.42
Stone Works	2,800,574.92
RCC works	62,810,109.78
Other works	21,191,326.87
OHS Cost	1,346,098.72
Grand Total	**102,271,375.62**

4 WATER SUPPLY SYSTEM

4.1 Introduction

Access to clean drinking water is vital for a healthy and thriving modern city. The importance of clean water and sanitation are so important that it has been adopted as one of the seventeen Sustainable Development Goals established by the United Nations General Assembly in 2015. Furthermore, the United Nations has deemed access to clean drinking water and sanitation a basic human right. Designing and implementing sound water supply system goes a long way in providing a continuous and safe drinking water.

Presently, Ammochhu LAP is sparsely developed with only a few permanent structures. The residents in the LAP are currently supplied drinking water from a reservoir located within the LAP via temporary water pipes. The reservoir is filled with treated water sourced from boreholes via a booster station. Since the urban design of the LAP such as roadways has only been planned and not implemented, construction of permanent distribution network has not been carried out. Therefore, there is a need to design a sound and robust distribution network for the LAP as the area is projected to develop rapidly in the near future.

The main roadways for the LAP which has been designed and planned would serve as the main path along which the water distribution pipes will run to form a robust network to supply water to the residents. The LAP is fairly flat with an elevation difference of only 18m from the highest to the lowest point. In addition to the current reservoir, a second reservoir is available to supply additional water to the LAP area. This second reservoir currently supplies water to the temporary shelter in Phuentsholing Township Development Project (PTDP) area. The reservoir would be available to supply water to the LAP once PTDP implements its own water supply network.

Phuentsholing Thromde has drafted a Water and Wastewater Master Plan, hereon referred to as the Master Plan, which details the overview of the water supply and wastewater removal system for the entire Thromde. An overview of the water supply system for Ammochhu LAP is provided in the Master Plan. The Master Plan details the water source, the treatment process, the location of reservoirs, and a skeletal distribution network for the LAP. However, a detailed analysis of the demand for the LAP and the required pipe sizes are left to be worked out once the urban design was finalized.

Therefore, for this project, a detailed analysis of the present and future demand of the LAP was be carried out. Based on the projected demand, the required pipe sizes were designed. A robust water supply network was designed to meet a 24x7 demand with adequate pressure.

4.2 Design Process

The proposed water supply system in the Master Plan was reviewed to get an understanding of the plan and goals for the LAP. In addition, the Ammochhu LAP report was also reviewed to study the overarching urban design goals and the projected carrying capacity. For the engineering design of the supply network, the following approach was taken;

1. The demand from each plot was calculated based on the precinct designation. Water demand is dependent on land use and will vary considerably based on whether it is residential, commercial or industrial. Based on the biggest structure allowed in each area, the projected water demand was calculated.

2. A model of the water distribution network was created in WaterCAD. The demand load calculated in step 1 was applied to the system to create a hydraulic model. Ground elevation data was taken from the topographical survey which was carried out by White Cypress Associate team as part of this infrastructure design project. In addition, the location and elevation of the main reservoir which currently supplied water to the LAP was sourced from the topographical survey.

3. The location and elevation of the second reservoir, which currently supplies water to the temporary shelters in PTDP area, was sourced from Thromde.

4. A steady state analysis of the distribution network with the two reservoirs connected to the distribution was carried out in WaterCAD. In addition to the steady state, an extended period simulation (EPS) of the hydraulic model with peak demand was carried out over a 24-hour period. The EPS provides added information on the working and robustness of the system over time which is not available under a steady state analysis.

5. The minimum pipe sizes which would meet the requirements of pressure and energy loss in the system was chosen from the EPS analysis and presented.

6. A detailed engineering drawing and the BoQs was prepared for the water supply network presented as part of the final deliverables.

4.3 Design Criteria

The designed water supply system shall supply water to its residents 24x7 without interruption, except during maintenance process. System appurtenances shall be strategically located so that maintenance work can be carried out with minimum disruption to the entire network. The water supply system shall provide adequate supply to meet the carrying capacity of the LAP. The designed network will also have adequate capacity to meet the fire need for emergency use. The design criteria for the Thromde water supply were discussed in detail in the Master Plan. However, the salient design criteria which are important to achieve the above conditions are discussed below.

4.3.1 Design Period and Population

New water supply systems are designed to meet the demand of growing communities for an economically justifiable number of years in the future. The choice of relevant design period is generally based on the useful life of component structures and equipment, taking into account the wear and tear. Another factor is the anticipated rate of population growth and water use by the community and its industries.

For Ammochhu, it is expected to see a rapid growth in the population due to its close proximity to the core town and the development of Phuentsholing Township on the adjacent side of the LAP. The useful life cycle period of distribution systems, including conduits and reservoirs are between 25 and 50 years. Given the expected development in the LAP, it is judicious to design a supply system which can cater to a fully developed LAP.

For the design of water supply system, it is customary to estimate the population that will be served by the supply system. Population data from government census and projection of the future population based on the current populations are usually employed to determine the design population. For the LAP, the carrying capacity as determined in the "Review of Ammochhu Local Area Plan, 2019" was adopted as design population. The carrying capacity represents the maximum number of residents which can be accommodated in the area without environmental degradation. The carrying capacity as calculated in the report is 22,458 people.

4.3.2 Water Consumption

Water supply systems are designed to meet population needs for a reasonable number of years in the future. For Ammochhu, the network will be designed to have capacity to supply water to a fully developed LAP which has reached it carrying capacity of 22,458 residents. To calculate the total water volume requirement, it is important to ascertain the volume of water required for an average person. This rate of consumption is normally expressed as the mean annual use in liters per capita daily (Lpcd). The demand for the Thromde will be adopted from the Master Plan and is reproduced here for reference.

Table 4-1: Water Demand for different category.

Category	Water Demand
Domestic	200 Lpcd
Commercial	15% of Domestic
Hotel	180 L/bed
Institutional	45 Lpcd without boarding
Industrial (drinking purpose)	35 Lpcd
Non-revenue water	20% of the above
Fire Demand	10%

Ammochhu LAP is mostly residential with a mix of commercial area and few green spaces. Therefore, the relevant demand from the Table 4-1 was used during the modeling building process. For a fully developed Ammochhu LAP with 22,458 residents and a water demand of 200 Lpcd, the total volume of water required is 4.49 MLD. The distribution network was sized to have adequate capacity to convey the 4.49 MLD to different areas in the network.

4.3.3 Variation of Patterns of Water Demand

The consumption and demand of water varies over the course of 24 hours in a day. For example, the demand for water increases as people wake up to prepare food and get ready for work. Similarly, domestic demand increases during the afternoon as people prepare lunch. Therefore, there is an increased demand during certain time of the day and a reduced demand at other times. The water supply system should be able to handle the increase in demand without compromising on the quantity as well as pressure at the point of use.

In steady state analysis, the increase in demand for water is modeled by choosing a peak factor which multiplies the average water demand to simulate the increase in water demand during certain time of the day. The peak factor simulates the highest hour demand during the course of the life cycle of the supply system and includes factors due to variability from daily, weekly and monthly demand.

In an extended period, simulation analysis, where the model is simulated over a course of 24 – 72 hours to reflect real work water demand, it is important to incorporate the variability in water demand on the system. The variability of water demand over the course of 24 hours for Thromde residents were studied and presented in the Master plan. The demand pattern is reproduced here for reference and has been modeled in the EPS analysis for the LAP water supply system.

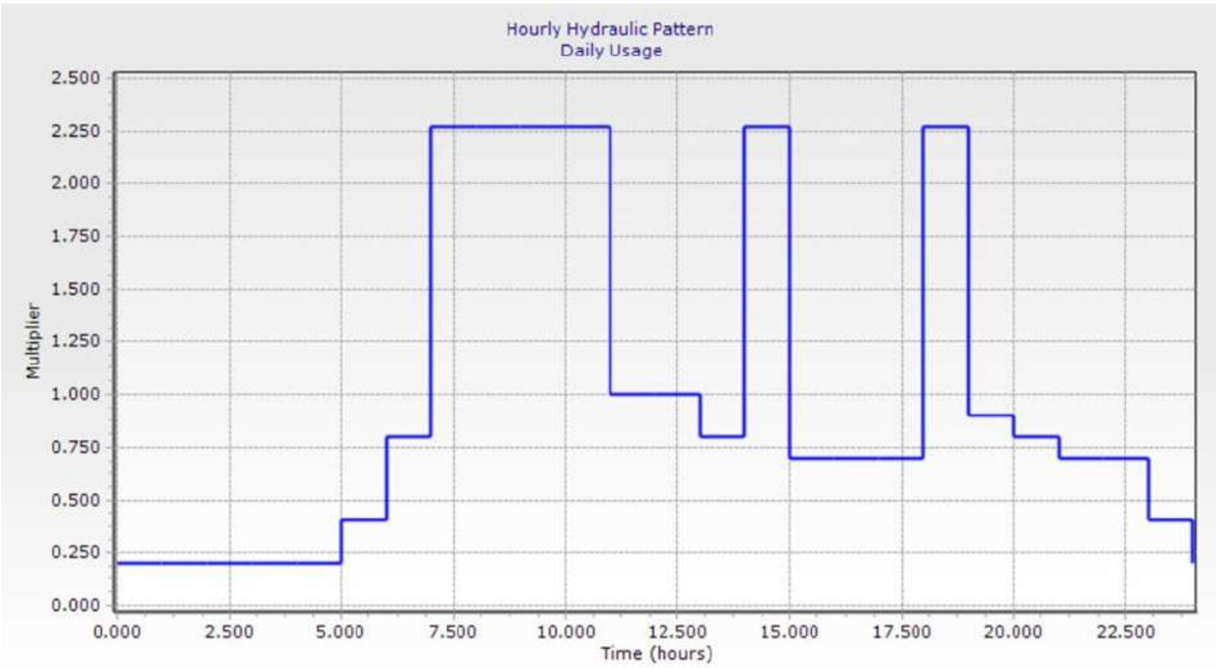

Figure 4-1: Daily Water Demand pattern for Phuentsholing Thromde.

4.3.4 Residual Head

An important design condition is residual head at the junction where the consumer taps into the distribution network. The residual head, expressed in meters of water, denoted the height to which the water would climb if a vertical pipe were to be connected at that point in the network. The residual head, also known as pressure head, can be used to easily understand if the water supply system would be able to deliver water to a certain height. For the LAP, the predominant precinct designation is UV-1, where the permissible structure is G+5. Therefore, with a stand floor height of 3 m, the average height of the highest floor in the LAP is 15m. A minimum residual head of 17m is adequate to deliver water to residents living on the 5th floor of such apartments.

4.3.5 Flow Velocity

The network was optimized with the aim to limit the flow velocity between 0.5 m/s and 3 m/s. A constant high velocity in pipe can lead to scouring and reduced life expectancy of the pipe. A high velocity also leads to excessive energy loss in the system which could otherwise be used to provide pressure at the point of use. Therefore, low velocities are preferred. However, achieving low velocities requires the use of large diameter pipe which increases the cost of construction. Hence, optimization of pipe sizes is important to achieve the most economical network.

4.3.6 Valves

Valves are installed to regulate water flow and pressure in the distribution network. Valves can be used to stop and start flow, control the flow rate, divert the flow, prevent backflow, and control and relieve pressure. Although valves have a lot of benefits, the costliness of them prevents them from being used extensively. The location and type of valves to isolation network and minimize supply disruption for maintenance are proposed and included in the network.

4.3.7 Hydraulics of Flow in pressurized Circular pipe

The hydraulic analysis will be carried out using Hazen-Williams's formula for friction loss in the system. The formula is widely used and recommended for design of distribution network. The formula includes coefficient C, which accounts for pipeline hydraulic friction characteristics. The roughness coefficient varies with pipe material, size and age. For new pipes, the proposed coefficient for ductile iron (DI) and galvanized iron (GI) pipes is be taken as 140 and 120 respectively.

$$h_f = 10.675 \times \left(\frac{Q}{C}\right)^{1.85} \times \frac{L_e}{D^{4.87}}$$

Where;

h_f= Head loss due to friction.

L_e= Equivalent pipe length in m.

Q= Flow rate in m^3/s.

C= Hazen-Williams's friction coefficient.

D= Internal diameter of pipe in m.

4.4 Proposed Water Supply System

The water supply system consists of water source, water treatment, storage and distribution networks. The water source and treatment for the LAP are detailed in the Master Plan. The primary water source is ground water drawn from bore wells located near YDF buildings. A disinfection treatment process is applied to the drawn water to render it safe for consumption. Based on the analysis of the water quality, a sole disinfection treatment process was deemed satisfactory to meet the requirements of safe potable water. The treated water stored in reservoirs from which is it pumped to various water storage tanks to be distributed via gravity flow to residents.

There are currently two operational water reservoirs, a reservoir capacity of 250 m^3 within the LAP area located at an elevation of 222.4m and another reservoir capacity of 300 m^3 outside the LAP area at an elevation of 216m. The reservoir outside the LAP currently supplies water to the temporary shelters in PTDP area. However, once PTDP installs permanent water supply, the reservoir will be used to supply water to the LAP.

4.4.1 Water Distribution Network

A model of the distribution network for the LAP was created and modeled in WaterCAD software. The model was assigned load based on demand from permissible structures which would be allowed in the area closest to each junction. Since the LAP is sparsely developed, the network loads were assigned based on demand from land development in the future. This meant that the worst-case scenario with the highest demand from a fully developed LAP is simulated in the analysis.

The network was optimized with the smallest sized pipes which would meet the demand with adequate pressure. The elevation different between the highest and lowest point in the LAP is 18 m, therefore the area was not required to be broken into pressure zones by using break-pressure tanks or pressure reducing valves.

A steady state analysis with the peak flow was conducted to design the network to meet the highest demand. The peak demand simulates the worst-case scenario in the foreseeable future in the life of the system. The hypothesis tested here is that if the network works in the most adverse case, it will work with no problem in the rest of the scenarios. In other words, if the pipes are sized to transport a discharge of 50 L/s of water, it will also be able to discharge 5 L/s. An EPS analysis was also performed to check the system for low-pressure and high-pressure areas during the course of 24 hours. The required reservoir capacity was also calculated from the EPS analysis.

4.4.2 Distribution Network Details

A gridiron network composed of two different pipe sizes was designed for the LAP. The modeling results showed that a maximum pipe size of 150mm were required in sections where there were high flows. In most sections, 100mm pipe were ideal as it provided a good compromise between high head-loss which occurs in smaller diameter pipes due to high flow rates in comparison to the increased in cost from adopting bigger diameter pipes. Galvanized Iron (GI) pipes were modeled for all distribution network pipes. A schematic diagram of the water supply distribution network is shown in Figure 4-2. Table 4-2 summarizes the total length of pipe required to implement the distribution network.

Table 4-2: Summary of Pipes and its total length.

Pipe Diameter	Length (m)
100 mm	3950
150 mm	2170

The distribution network was analyzed for residual pressure at the connection point (nodes in the model) at off peak (time of least demand) and peak demand hour (time of maximum demand) through a course of 24 hours. The off-peak time occurs at 1 AM while the peak demand hour occurs at 1 PM. A distribution network which meets the residual pressure requirement at both the peak and off-peak demand hour will be able to provide adequate pressure throughout 24 hours and the network is designed to meet this requirement. The residual pressure head at consumer connection points during off-peak demand hour and peak demand hour are given in Figure 4-3. For more details on the pressure, flow and velocity in the pipes during peak and off-peak hour, refer to Annexure D of this report.

The Master Plan states that a minimum of 17 m of residual head be available at all connection points in the distribution system. However, due to the topographical constraints as well as the location of reservoirs which are fixed, a few connection points have residual pressure below 17m during the peak time. Those locations nevertheless, have residual pressure in the range of 13 m-17 m as seen in the Figure 4-3 which is more than adequate to supply water up to 5th floor in a residential building. A booster station could have been incorporated to increase the pressure in the system, but this solution was not pursed as it would only add cost, add complexity and lead to increase in water loss due to increased pressure in the system.

The status quo for water supply system in Phuentsholing is to supply water with adequate pressure to supply the ground level of all residents. In areas where there are multi-storied structures, the proprietor of the building stores the water in a storage tank and pump it to another storage tank located on the top of the building. The water is then distributed down from the tank to the residents in the building.

In areas where the minimum residual pressure is insufficient to deliver water to the level where the residents live, especially if a G+8 or G+9 story buildings are allowed in the area, it should be the responsibility of the owner to provide storage and pumping facility to distribute to its residents. Increasing the residuals pressure in the system to accommodate a few high-rise buildings would not only increase the cost of operation for Thromde, it would also increase water loss due to high pressure. In addition, the residents living on lower floors of a building would experience very high pressure making it unpleasant to use. It is also the engineering standard that any structures which are higher than 6 stories have their own pumping facility to provide adequate pressure to the residents. The municipality responsibility is to provide water at reasonable pressure to all residents.

4.4.3 Reservoir Capacity

Two reservoirs supply water to the network. The reservoir located at an elevation of 223m which currently supplies water to the LAP is named reservoir 1 and the reservoir located at an elevation of 216m which currently supplies water to the temporary shelter in PTDP area is named reservoir 2.

The network draws most of the water during off-peak hour from reservoir 1 as it is located at a higher elevation. This is due to the fact that the pressure head at the connection points in the network is higher than the elevation head of the reservoir 2. To prevent flow of water from the network into reservoir 2 during off-peak hours, a check valve is installed in the pipe connecting the reservoir to the distribution network. Without the check valves, water will be drawn from reservoir 1 and will flow into reservoir 2 during off-peak demand hour. However, during peak demand, water is drawn from both reservoirs. The total volumetric demand for the LAP is 4,305 m3. Of the total, 3099 m3 is supplied by reservoir 1 and 1206 m3 is supplied by reservoir 2.

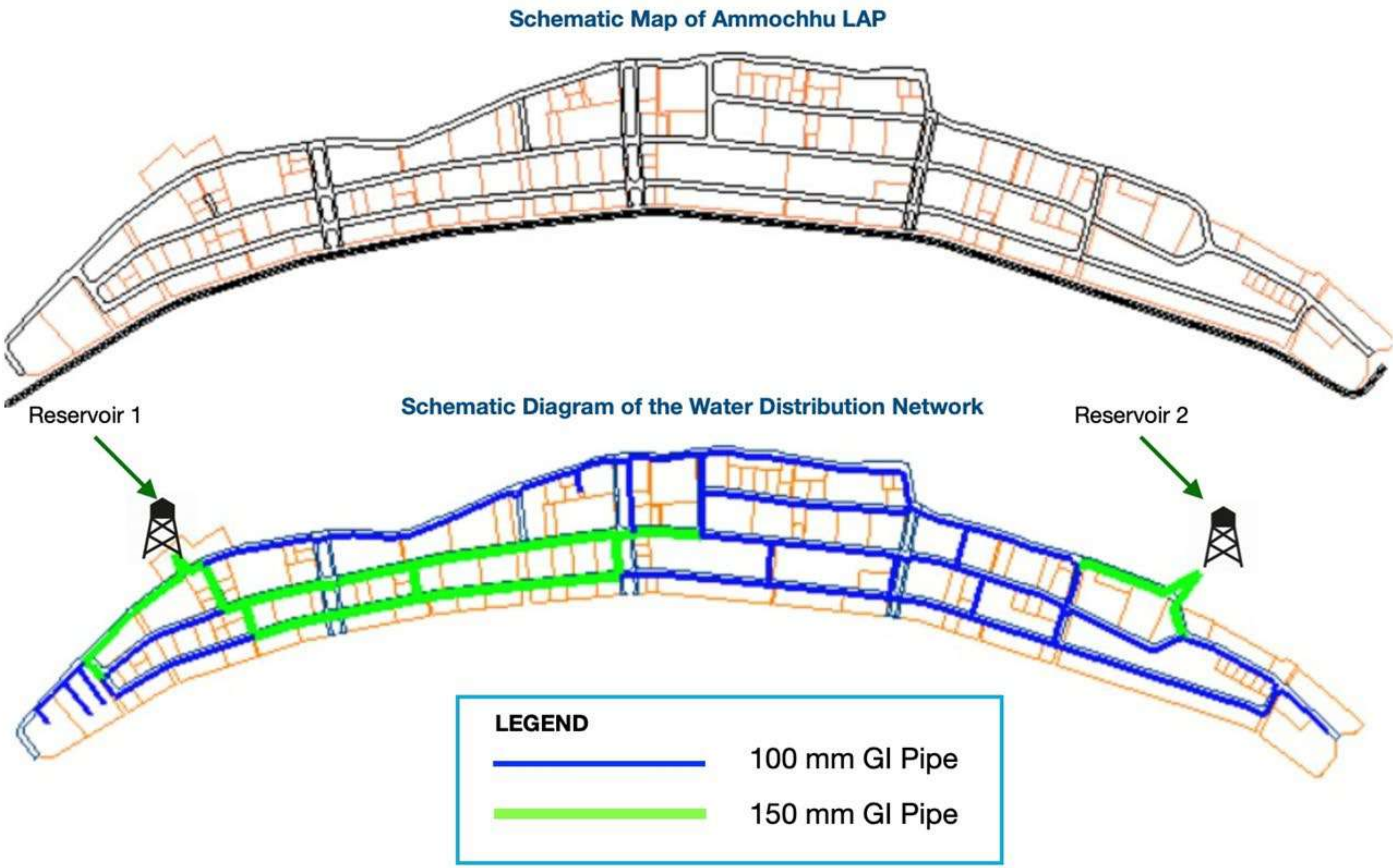

Figure 4-2: A Schematic Diagram of the Water Distribution Network showing Pipe Sizes and Reservoir Location

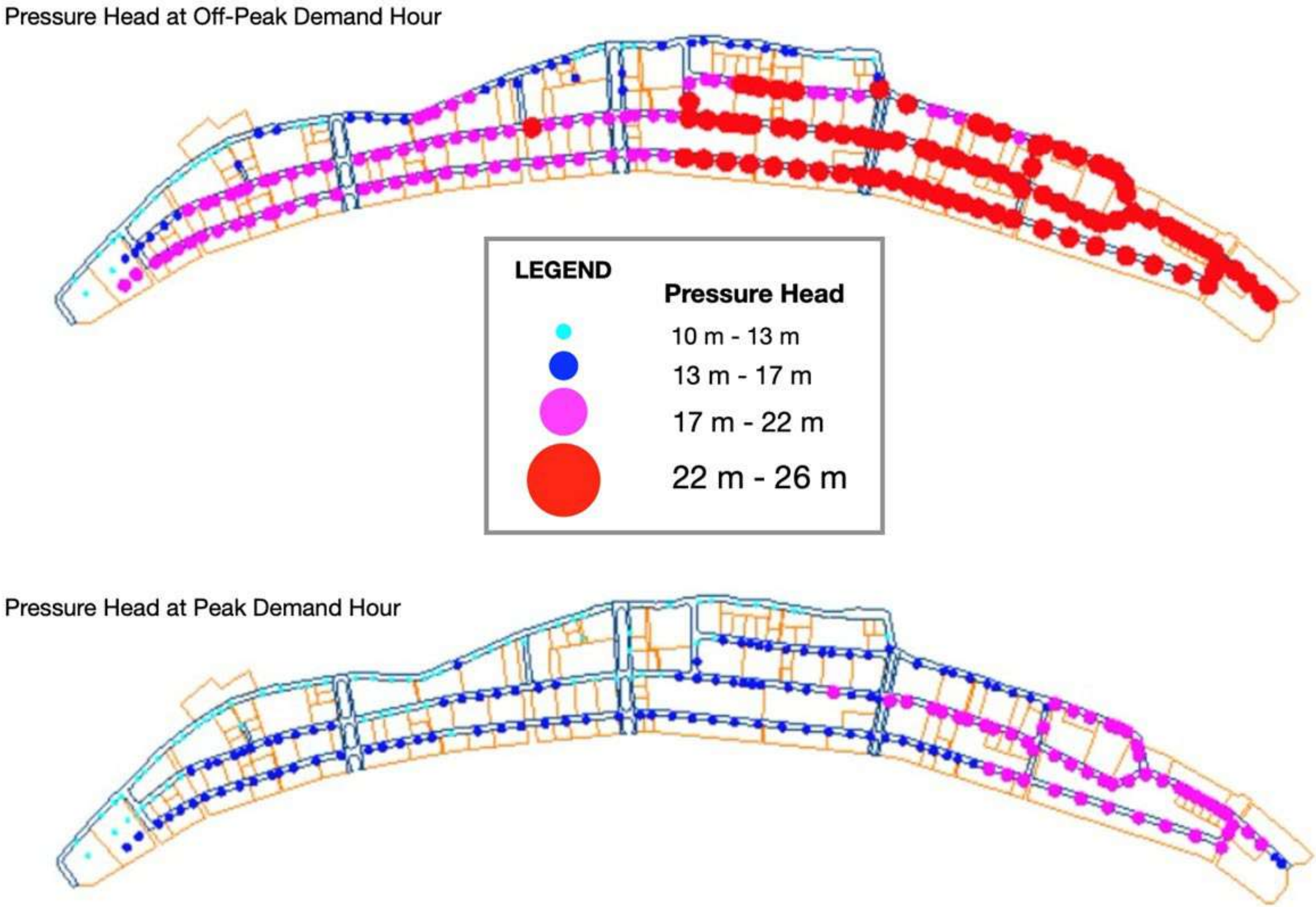

Figure 4-3: Pressure Head at Consumer Connection Points at Peak Demand Hour.

To size a reservoir, it is important to ascertain total daily demand. The demand on each reservoir over a course of 24 hours was extracted from the EPS analysis. Based on the total daily demand on reservoir 1, a minimum pumping rate of 150 m^3/h was required. Based on the demand and supply analysis, a minimum equalization volume of 364 m^3 was required. A standard practice is to keep 10% of the daily demand for fire and emergency reserve. For reservoir 1, the reserve is 310 m^3. Therefore, the requirement volume for reservoir 1 is 674 m^3.

For reservoir 2, the daily demand is 177m^3, with 121m^3 required as reserve. Therefore, the minimum reservoir capacity for reservoir 2 is 298m^3. Table 4-3 details the required.

The analysis of the reservoir capacity shows that no additional capacity needs to be added for reservoir 2. The current capacity would be adequate they meet the demand of a fully developed Ammochhu LAP. However, reservoir 1 needs to be augmented with an additional capacity of 425m^3 to meet the demand of a fully developed LAP.

Table 4-3: Required Reservoir capacity[9].

Volume in m^3	Reservoir 1	Reservoir 2
Total Daily Demand	3099	1206
Equalization Volume	364	177
Fire and Emergency Reserve	310	121
Reservoir Capacity Required	674	299

Since the LAP is sparely developed, reservoir 1 can easily meet the current demand. Additional capacity will need to be added only when the LAP reaches 50% of it carrying capacity. That is to say, when the population of Ammochhu reaches more than 11,000 people. Given that the current population is less than 1000, it is very unlikely that an additional reservoir will have to be built in the next 5 to 10 years. However, once additional capacity is required, additional capacity can be augmented by a RCC reservoir tank next to the LAP reservoir location. A section view drawing of the RCC reservoir is given in Figure 4-4[10] . Detailed drawing and BOQs are provided as part of the final submission.

[9] Refer to Appendix on Hydraulic Modeling Results for reservoir capacity calculation results

[10] For detailed engineering drawing, check CAD drawing STR-02 provided with this report

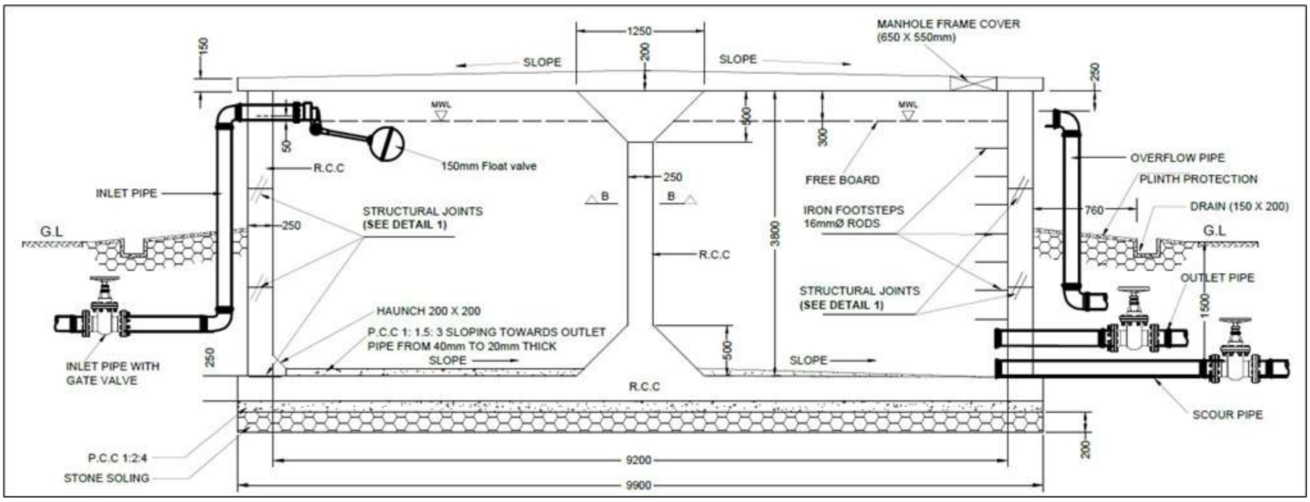

Figure 4-4: Section view of RCC Reservoir Tank.

4.4.4 Valves

For isolation of pipes during maintenance, sluice valves are provided at strategic intersection of pipes. One-way valves are provided on the pipe connecting reservoir 2 to the network so that water doesn't flow from reservoir 1 to reservoir 2 during off peak demand hour. Scour valve is provided at the lowest point in the network system for flushing the system for cleaning and maintenance purposes. For the location of the valves, check key plan CAD drawing CU-WS-1 provided with this report.

4.4.5 Fire Hydrant

Fire hydrants are connection points within the water distribution network where the fire fighters can tap into the network to quickly put out fire. Fire hydrants are strategically located throughout the LAP to provide quick access to directly tap the network as well as fill fire trucks for those near-by areas not reachable directly via the hydrants.

The fire hydrants should be located at least 0.6m away from the curb to allow for adequate space for installation as well as protection from vehicular accidents. Four different types of fire hydrants are commonly used in water distribution networks- dry-barrel hydrants, wet-barrel hydrants, warm-climate hydrants, and flush hydrants. For Ammochhu LAP, a dry-barrel hydrant is proposed as there is no flow of water from a broken hydrant. A typical section of a dry-barrel hydrants is shown in Figure 4-5. The locations of the fire hydrants are detailed in the Master layout of the water distribution network drawing.

Figure 4-5: A Typical dry-barrel hydrant and its section.

4.4.6 Consumer Connection

Consumer connection will be carried out by tapping the water main. Each property should be connected with a 20mm diameter GI pipe connection as shown in Figure 4-6. This will ensure that the property will have access to the designed minimum residual head of 17m. Sharing of a single consumer connection can lead to excess demand on the connection leading to reduction in pressure and flow.

The typical consumer connection shown in Figure 4-6 should be implemented during the construction of the water supply network. In order to minimize the need to dismantle the road for future connection, a provision for water connection for every plot should be provided during the implementation of the network.

Consumer connection can be provided as and when a connection is required, however, it is recommended that connections for areas which are under development to be provisioned during the implementation phase. The water main runs inside the utility duct in all areas except along the western avenue. There is no utility duct along the western avenue. On either side of the western avenue are storm water drains. The water main long the western avenue is laid next to the storm drain underneath the footpath. A cover depth of 0.8m is proposed to protect the pipe from damage. The minimum trench width and cover requirements for different pipe sizes is given in the CAD file provided with this report.

To connect property on the opposite side of the water main where the lateral connection has to cross the road to reach the property, a 50mm HDPE pipe laid under the road to convey the 25mm diameter lateral is proposed. The consumer connection pipe can then be conveyed through this 50mm pipe when a connection is required on the opposite side. Four lateral pipes of 20mm diameter pipes can be conveyed inside the 50mm diameter HDPE pipe. However, if the site engineering foresees a need to convey more than 4 lateral pipes in the future, a large HDPE can be installed during the construction.

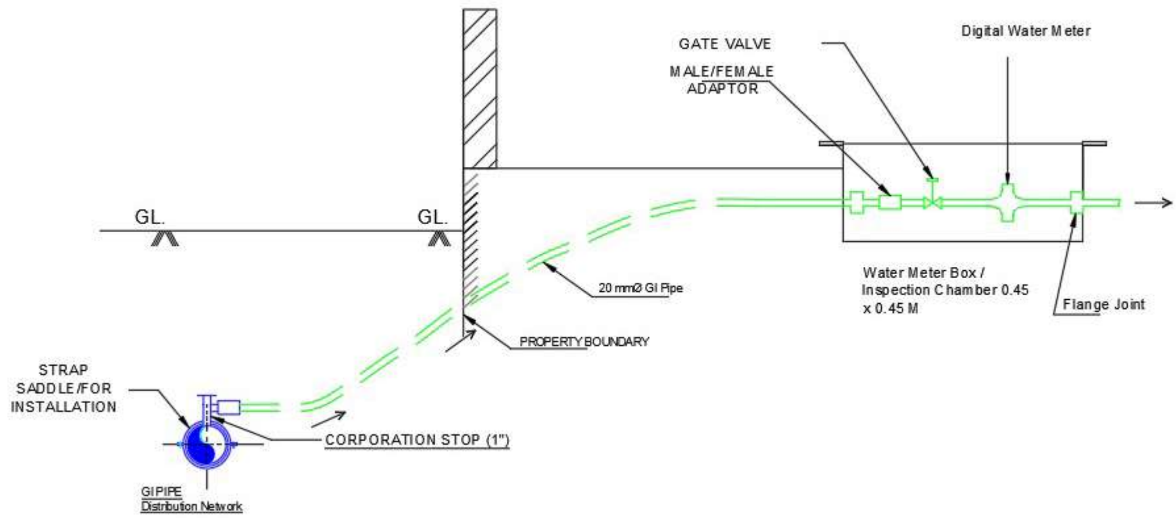

Figure 4-6: Typical Consumer Connection.

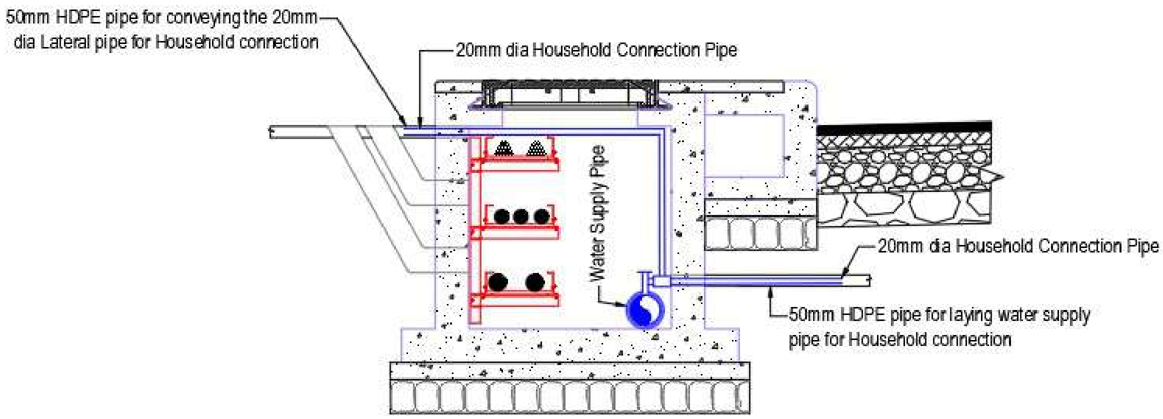

Figure 4-7: Cross-section of the Utility Duct with water main and lateral pipe for consumer connection.

4.4.7 Connection with PTDP

The Phuentsholing Township Development Project lies next to Ammochhu LAP separated by the PCR road. The project has developed a separate water supply system for the area complete with treatment, storage and distribution. Due to the proximity to the LAP, there is an opportunity to improve the robustness of the water supply system by interconnecting the two system to provide water during emergencies/shortages. A direct interconnection between the two system at the distribution network is not possible as the two systems were designed separately and have different operating characteristics. However, a connection between the two storage reservoirs is possible to supply and receive water.

During the design of the PTDP's network, a location for a possible LAP connection was identified. This location has been used as the taping point to convey water to the LAP reservoir. The layout of the pipe line is given in Figure 4-8. PN10 HDPE pipes were used in the implementation of PTDP's network and the connection to PTDP's network is via a PN10 HDPE pipe of 315mm in diameter. A PN10 HDPE pipe of 225mm diameter is recommended for connecting the LAP reservoir with PTDP's network. This pipe line which is 460m in length can be used to convey water from PTDP to the LAP reservoir and vice versa depending on the water requirements in the two areas.

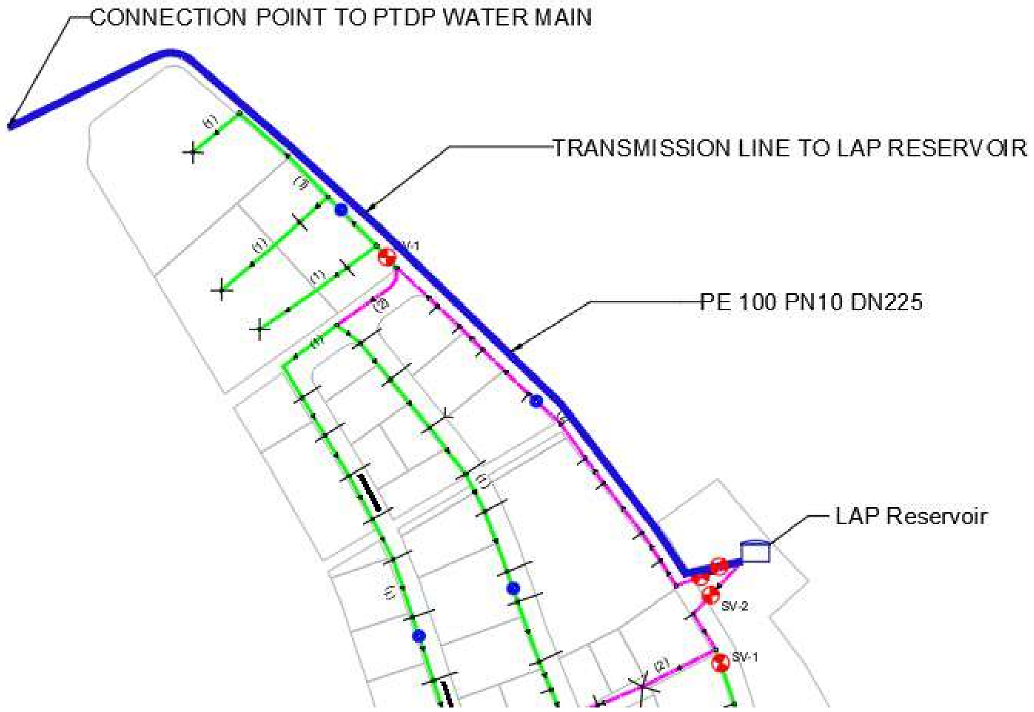

Figure 4-8: Pipe Line Connection between PTDP and LAP Reservoir.

4.5 Bill of Quantities and Estimate

The total cost of implementing the Ammochhu LAP water supply network is Nu.27,856,943.93. The detailed Bill of quantities and cost estimate has been submitted in excel format as project deliberates.

5 SEWER SYSTEM

5.1 Introduction

Sewer systems convey domestic, commercial, and industrial wastewater (and in many cases storm water and groundwater) from its sources to a location where it can be treated and discharged to a receiving water body. Wastewater is defined as "the spent or used water of a community or industry which contains dissolved and suspended matter." (American Society of Civil Engineers. 1982).

Sewer systems are designed to collect and transport wastewater from domestic, commercial, and industrial sources. However, inadvertent or illegal connections frequently result in entry of additional flows into the system. Infiltration is water that enters the system from the ground through defective pipes, pipe joints, connections, or manhole walls. Inflow is water discharged into the sewer from sources such as building and foundation drains, drains from wet or swampy areas, manhole covers, cross connections, catch basins, or surface runoff. Collectively, these flows are referred to as infiltration/inflow or I/I and will form the primary source of wastewater to be conveyed by the sewer network.

At present, there are no sewer system in place in Ammochhu LAP. The LAP is fairly undeveloped with only a few permanent buildings (which has been constructed recently), and temporary structures which are used as warehouses and workshops. Currently, the wastewater generated from the residential houses are conveyed to septic tanks and the grey water from kitchen and bathroom released into road side and storm drains.

For this assignment, a sewer network will be designed which will be in line with the Urban Development Plan of Ammochhu LAP. The network will consist of network of pipes, connecting manholes or access chambers and conveying the wastewater to the main sewer pipe along the PCR road.

5.2 Design Process

The Water and Wastewater Master Plan, hereon referred to as the Master Plan, details an overview of the drinking water distribution system and sanitary sewer system for Phuentsholing Thromde. The sewer system part of the Master Plan was reviewed thoroughly to understand the overall sewer system of the Thromde as well as get a grasp of the objectives moving forward. Although the Master Plan details the sewer system for the core town in details, such details for Ammochhu LAP are lacking.

To implement a sound sanitary sewer network, a detailed analysis needs to be carried out to correctly size the sanitary sewer pipes so that it has adequate capacity to handle the growing needs of the residents in the LAP. For this project, the sanitary sewer network for Ammochhu LAP was modeled and analyzed using SewerCAD, an industry leading software used throughout the world for modeling and analyzing sewer networks.

The design process included the following processes;

1. Wastewater generation was calculated based on the water demand of the different land use in

the LAP. 80% of the water supplied via the water supply network was projected to be removed as wastewater.

2. The hydrologic data for rainfall and storm event for Phuentsholing will be analyzed as input for the Infiltration and Inflow (I/I) into the sewer network for wet weather calculation.

3. For sewers where the flows are not normally under pressure and mostly flow partially full, Manning's formula was used.

4. Manholes will be located at junction of pipes and change in pipe sizes. Ground elevation of the manholes was determined from the topographical survey which was updated to reflect the fill level in the LAP area. Invert elevation was calculated based on the slope requirement as well as volume and flow capacity.

5. Two models of the sanitary sewer network were created in SewerCAD. A detailed analysis of the chosen model was carried out and presented in this report.

6. Finer details of the network were worked out and detailed in a CAD file provided with this report. Location of pipes, their sizing, manhole location and invert elevation were detailed.

7. A detailed BoQ was prepared and submitted. In addition, the modeling data from SewerCAD is submitted as part of the final deliverables.

5.3 Design Criteria

5.3.1 Design Period and Flows

The length of time up to which the capacity of a sewer will be adequate is referred to as the design period. In fixing a design period, consideration must be given for the useful life of structures and equipment employed, taking into account obsolescence as well as wear and tear.

Similarly, design flow is largely a function of the population served, population density, water consumption. Lateral and sub main sewers are usually designed for peak flows of the population at saturation density as set forth in the Master Plan. Trunk sewers, interceptors, and outfalls are difficult and uneconomical to be enlarged or duplicated and hence are designed for longer design periods.

Accurate estimates of sewer system flow rates are the foundation of a sound hydraulic sewer models. Flows in wastewater sewer systems are generally divided into two categories. Wastewater (also called sanitary or dry weather) flows are the intentional discharges into the sewer system. They may originate from residential, commercial, institutional, or industrial sources. Sanitary sewer systems also collect infiltration and inflow (I/I), which principally originate as precipitation.

The major contributor of dry weather flow into the sewer system in the LAP is from residential buildings. For this project, CPHEEO recommendation of removing 80% of the water supplied via the distribution system will be taken as the loading for the sewer system. The daily per capita demand of water for Phuentsholing Thromde, as per the Master Plan, is 200 LPCD. Therefore, each resident in the LAP contributes 160 LPCD of wastewater which should be removed via the sewer network.

In Ammochhu LAP, the majority of the plot falls under Urban Village-1 (UV-1) designation. For such plots, the maximum permissible floor height is 6 as stated in Ammochhu Development Control Regulations. As per Population and Housing Census of Bhutan (PHCB) 2017, the mean household size of Phuentsholing Thromde is 4.4. Given that each floor of most residential building houses two apartment, the total residential dwelling is taken as 12. Therefore, the wastewater generated from each residential building is taken to be 8450 LPD [11](liters per day).

For the design of a new system, gravity sections usually convey peak flows without surcharge or with some excess capacity. In practice, peak hourly flows at the end of the design life of a sewer are used to simulate a worst-case scenario and provide a conservative design for the new system. For this project, a peak value of 2.5 was adopted from the CPHEEO manual. Table 5-1 adopted from CPHEEO manual specifies the peak value recommended based on the population served by the sewer system.

Table 5-1: Criteria for peak Factor (adopted from CPHEEO manual 2013).

Contributary Population	Peak Factor
Up to 20000	3.0
20,000 to 50,000	2.5
50,000 to 75,000	2.0- 2.25
More than 75,000	2.0

Since there are only a few permanent structures which has been built on the LAP, the sewer modeling was carried out based on a fully developed LAP. For all plots which are currently undeveloped, it was assumed that a G+5 story residential building would be built and the subsequent wastewater generated was used in sizing the sewer pipes.

In addition to the design flow, allowances for increase in flow due to infiltration from groundwater into the sewer lines through pipe joints was made. In accordance with CPHEEO manual, the infiltration flow will be calculated based on Table 5-2 and limited to a maximum of 10% of the design sewage flow.

Table 5-2: Groundwater Infiltration into Sewer pipes (CPHEEO 2013).

	Minimum	Maximum
Liters/ha/day	5,000	50,000
Liters/km/day	500	5,000
Liters/day/ manhole	250	500

[11] This assumes the worst-case scenario as the ground level apartments in residential buildings are mostly used for commercial purpose rather than being used for residential dwelling. Therefore, higher conservative value of 8450 LPD is taken for modeling purpose

5.3.2 Sewer Pipe material and Sizes

High density polypropylene (HDPE) pipes were used in modeling the flow of wastewater in the sewer pipes. HDPE pipe was chosen as it was recommended and approved by Thromde in the Master Plan to be used as sewer pipe. The Master Plan list the advantages HDPE provides over other pipe materials available in the market. For more information, please refer to the Master Plan.

The modeling was carried out using sizes that were available in the market. CPHEEO manual recommends a minimum of 150mm size diameter for towns having present / base year population of less than 1 lakh. Since the projected LAP population is less than 1 lakh, we adopt a minimum of 150mm pipe size for the sewer network.

5.3.3 Manholes

Manholes are opening in the sewer lines which provides access to maintenance personal for inspection, maintenance and conduct system upgrades. The openings are usually fitted with removable covers made from materials which are able to withstand traffic loads. Manholes maybe constructed from precast concrete, places insitu or constructed from other materials such as bricks, HDPE, PVC or RCC materials.

The construction of insitu manhole is recommended when prefabricated structures are not readily available. The insitu construction have the added advantage of combing different materials such as bricks with RCC which provides excellent quality control and workmanship during construction. However, when precast structures manufactured from RCC or HDPE/PVC are readily available in the market, they have the advantage of faster construction allowing minimum road closure in already developed areas. Therefore, it is recommended to use precast structure if the Thromde has easy access to it.

The maximum distance of 30m between sewer lines will be maintained where maintenance crew cannot physically enter the pipes for cleaning and maintenance. Where the sewer lines are large enough for maintenance crew to enter for inspection and maintenance, the manholes will be placed at every change of alignment, gradient and diameter change, head of sewer and branches, and junction of two or more sewers.

5.3.4 Hydraulics of Flow in Circular Sewers pipe

The analysis of fluid flow is closed conduit is based on the flow characteristics of the sewage under consideration. When the velocity of the fluid is same throughout the depth of pipe over time, the flow is termed steady flow. For design purpose, the flow in sewer is assumed to be steady even though the flow through the sewer will vary over the course of the day with peak flow mostly occurring during the morning.

The sewer lines were designed such that deposition of particles and solids present in the sewage will not occur. However, such velocities might be impractical to maintain throughout the day in practice. Therefore, steps will be taken so that the scouring velocities will be achieved at least once a day during peak flow to encourage self-cleansing of sewer lines. If this is not achieved, the sewer lines can steadily get deposited with silts causing blockage.

Empirical formulas are usually employed in calculating the velocities of flow in close conduits. For sewers where the flows are not normally under pressure and mostly flow partially full, Manning's formula is used. Manning's Formula for Gravity Flow is given below;

$$V = \left[\frac{1}{n}\right] \times [R^{\frac{2}{3}} \times S^{\frac{1}{2}}]$$

For Circular Conduits

$$V = \left(\frac{1}{n}\right)(3.968 \times 10^{-3}) D^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$Q = \left(\frac{1}{n}\right)(3.118 \times 10^{-6}) D^{2.67} S^{\frac{1}{2}}$$

Where;

Q= Discharge in l/s.

S =slope of hydraulic gradient.

D= Internal diameter of pipe in mm.

R= Hydraulic radius in m.

V= Velocity in m/s.

n= Manning's coefficient of roughness.

Manning formula given above is used as the formula of choice in SewerCAD for modeling and analyzing flow in the sewer network.

5.3.5 Minimum Velocity for preventing Sedimentation

To ensure that deposition of suspended solids does not take place, self-cleansing velocities using Shield's formula is considered in the design of sewers.

$$V = \frac{1}{n}\left(R^{\frac{1}{6}} \sqrt{K_s (S_s - 1) d_p}\right)$$

Where;

n= manning's roughness coefficient.

R= Hydraulic mean Radius in m.

K_s= Dimensionless constant with a value of about 0.004 to start motion of granular particles and about 0.80 for adequate self-cleansing of sewers.

S_s= Specific gravity of particle.

d_p= Particle size in mm.

The above formula indicates that velocity required to transport material in sewers is mainly dependent on the particle size and specific gravity and slightly dependent on conduit shape and depth of flow. The specific gravity of grit is usually in the range of 2.4 to 2.65. Gravity sewers shall be designed for the velocities as in Table 5-3.

Table 5-3: Design Velocities to be ensured in gravity (CPHEEO).

No	Criteria	Value
1	Minimum velocity at initial peak flow	0.6 m/s
2	Minimum velocity at ultimate peak flow	0.8 m/s
3	Maximum velocity	3.0 m/s

5.3.6 Pipe Slope and Cover Depth

The pipe slope is the invert (lowest point in the cross-section) drop per unit length of the pipe. It is typically expressed as a unit less value (i.e., length/length), but units may be provided ((e.g., m/m or mm/mm). Often it is expressed as a percent which has been adopted and presented in this report.

All sewers were designed to give mean velocities, when flowing full, of not less than 0.6 m/s (which is the minimum velocity discussed in section 4.3.5). This is achieved by assigning a minimum slope given in Table 5-4 and checking the design results after the analysis. However, where possible, slopes greater than the minimum recommended were chosen as it is desirable for construction, to control sewer gases and to maintain self-cleansing velocities at all rates of flow within the design limits.

Table 5-4: Minimum Slope of Sewer pipe.

Pipe Size	Minimum Slope in m/100m	Minimum slope in %
150	0.0167	1.67
200	0.0067	0.67
250	0.0057	0.57
300	0.0044	0.44
350	0.0040	0.40

The cover depth is the distance from the soil surface to the top of the outside surface of the pipe. The minimum cover of 1.0 m is kept for all sewer pipes. As the service area is relatively flat, the required pipe slope is greater than the slope of the ground surface. Therefore, as the gravity sewer proceeds downstream, it is forced deeper increasing the cover depth in most of the service areas.

5.4 Proposed Sanitary Sewer Design

The proposed sanitary sewer network was designed to handle a fully developed LAP. The modeling and analysis were carried out in SewerCAD, a hydraulic modeling software which is commonly used to design and size sewer networks of varying sizes. In generating the model, actual data collected from the design site were used when available. For data which were not available, recommended values from CPHEEO manual and reputable engineering documents were used.

Ground elevation data were extracted from the recently conducted topographical survey by the White Cypress Associate team. The updated topographical map with the new fill level was used in to accurately model the ground elevation where LAP filling was proposed. Sanitary Loading data were derived from data sourced from the Master Plan and the "Review of Amo Chhu Local Area Plan 2019" report. PTDP Sewer Network details were sourced from the PTDP project officials.

Two network options[12] were presented for evaluation and consideration during the concept phase of the project. In the first option, a network comprised of subnetworks which connected to PTDP's primary sewer trunk line along the PCR road was proposed. In the second option, the subnetworks connected to a primary trunk line within the LAP which conveyed the wastewater to the single manhole in the PTDP's primary sewer trunk line.

[12] Refer to the design concept report for more details.

During the presentation of the concept plan, there were consensus among the client and the consultant that option with subnetworks connecting to different point on the PTDP's primary sewer line would be preferable. It was decided that the design be discussed with PTDP's team before proceeding with the detailed design analysis. To that end, an online meeting between the client, PTDP's team and the consultant was arranged. During the meeting, however, it was brought to the attention of the client and the consultant that the implementation of the PTDP's sewer line has already been carried out and the increase in capacity of the primary sewer line that would be required to accommodate the sewer loading from the subnetworks would not be possible. Since option one was not feasible, the client and the consultant agreed to proceed with design option two.

5.4.1 Sanitary Sewer System Design

Ammochhu LAP is bordered by Dhamdara on the east and PTDP on the west with the RCR road running between the border. The LAP is relatively flat with the ground sloping gently from north to south. The service area slopes from an elevation high of 214m in the north to 194m in the south.

The sanitary sewer network was designed to collect the wastewater via gravity flow. The wastewater is collected via laterals through property connections and conveyed to the nearest manhole. The manholes are connected via sewer pipes with the sizes gradually increasing as it flows south collecting more wastewater. A primary sewer line was designed to run along the Western Avenue Road which acts as both a collector of wastewater from the laterals (which connects to properties lying on either side of the road) as well as secondary sewer pipes (which collects wastewater from properties located on higher elevation). The primary sewer line drains the wastewater collected from the LAP to a designated manhole on the main PTDP's primary sewer line.

In addition to wastewater generated from Ammochhu LAP, the network was also designed to provide capacity to handle wastewater from backside Dhamdara. Manhole (Label - MH25-2-2) receives the wastewater from backside Dhamdara while manhole (MH15-2-2) receives wastewater from adjacent areas in Dhamdara. For schematic of the network, refer to Figure 5-1.

The main sewer trunk line increases in size as it collects wastewater from the adjacent properties via the laterals as well as the secondary sewer pipes transporting wastewater from the secondary road. Detailed analysis carried out using SewerCAD shows that a range of pipe sizes from 160mm to 315mm were required to provide adequate capacity to convey the peak flow. A 450mm RCC Hume pipe were proposed under the drain outfalls. Detailed analysis of the sewer network including hydraulic results are presented in the Model Results section.

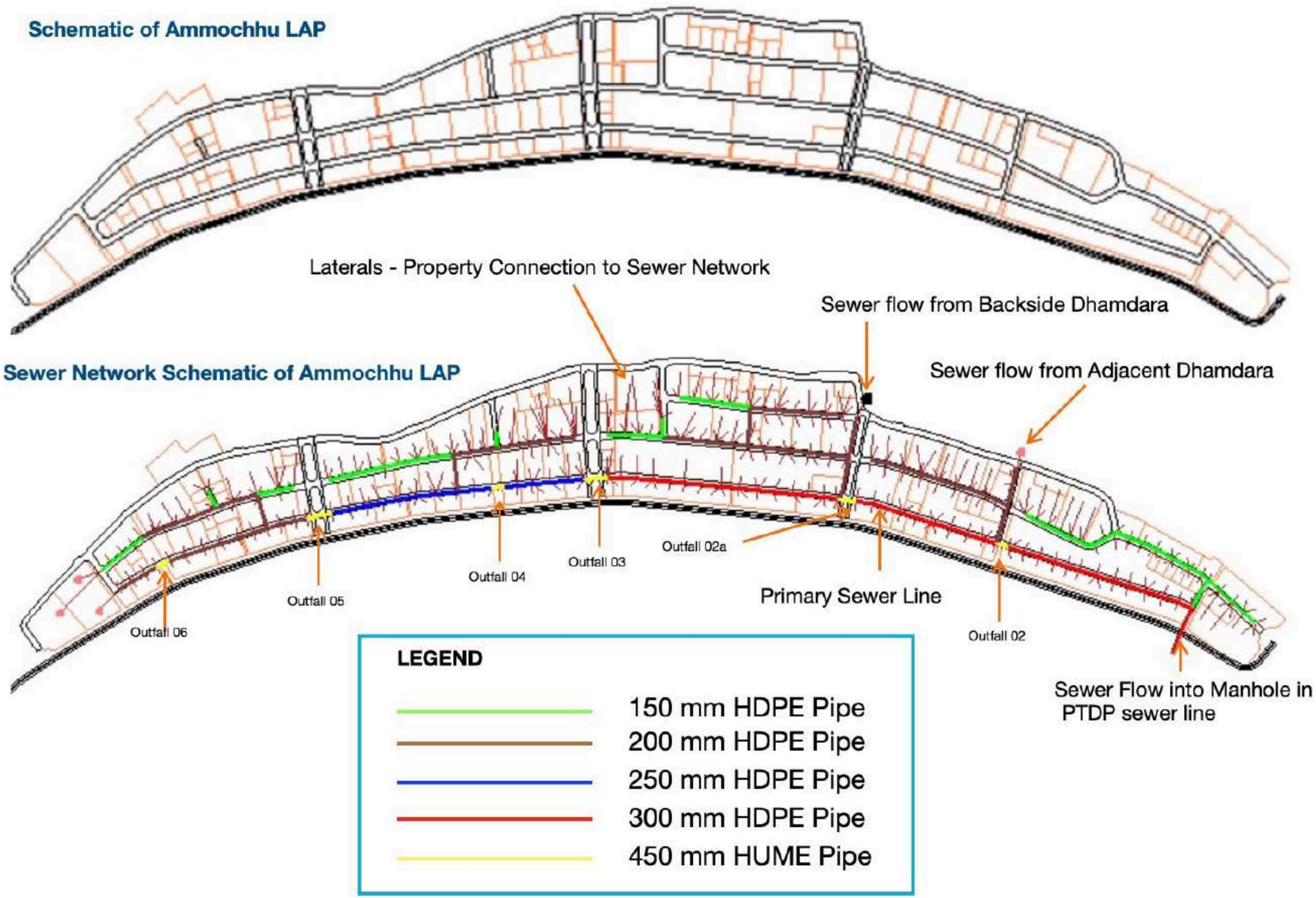

Figure 5-1: Network Option 2 with Dedicated Main Sewer Trunk Along Western Avenue

5.4.2 SewerCAD Modelling Details

The SewerCAD model was constructed using topographical data from a recent survey of the LAP carried out by the White Cypress Associate's team. The survey data had a control interval of 0.25m and covered the whole service area. Loading data were derived from the water and wastewater master plan. Additional information such as the location and invert elevation of the receiving manhole in PTDP's sewer network were provided by PTDP's official.

a) Manholes/Junctions

The manholes were modeled to collect wastewater from both laterals from properties connections as well as inflows from other sewer pipes. The location and size of manholes were based on the design recommendation as discussed in the design section of this report.

b) Sewer Pipes/Conduits

Circular HDPE pipe of standard diameter were used for modeling the network. The diameters were chosen to match the standard sizes which are readily available in the market. A manning's n value of 0.01 which is typical for HDPE pipes was used in the hydraulic analysis. The pipes were also modeled to allow groundwater infiltration of 10% of the total design flow.

c) Property Connections

Property connections were provided to every plot in the LAP area. The number of property connections provided were based on the size of the plot. Service connections were also modeled for green spaces to provide with added capacity for any future development including public washrooms.

d) Lateral/Property Connections

The lateral connections were modeled using the 100mm HDPE pipe and represents the connection of each building to the nearest manhole. The model was optimized to provide adequate capacity at each of those manholes which would be receiving wastewater from the household as well as secondary sewer pipe which might connect to it.

e) Simulation Period

The model was analyzed for a period of 24 hours using a synthetic sanitary flow pattern. The synthetic flow pattern was constructed to include a peak flow of 2.5 times the average flow. CHPEEO's manual recommendation for peak flow for a design population of 20,000 to 50,000 is 2.5. Using this design value, a synthetic flow pattern was constructed to simulate the flow during an average day as shown in Figure 5-2. The peak flow occurs at 8 AM in the morning as people wake up and use water to shower, cook and clean. Higher flows are also simulated at noon and evening as people prepare lunch and dinner. At other times, the flows are kept below the average to simulate low flow in the network. The network was optimized to handle the volume at the peak flow which would represent the worst-case scenario for the network.

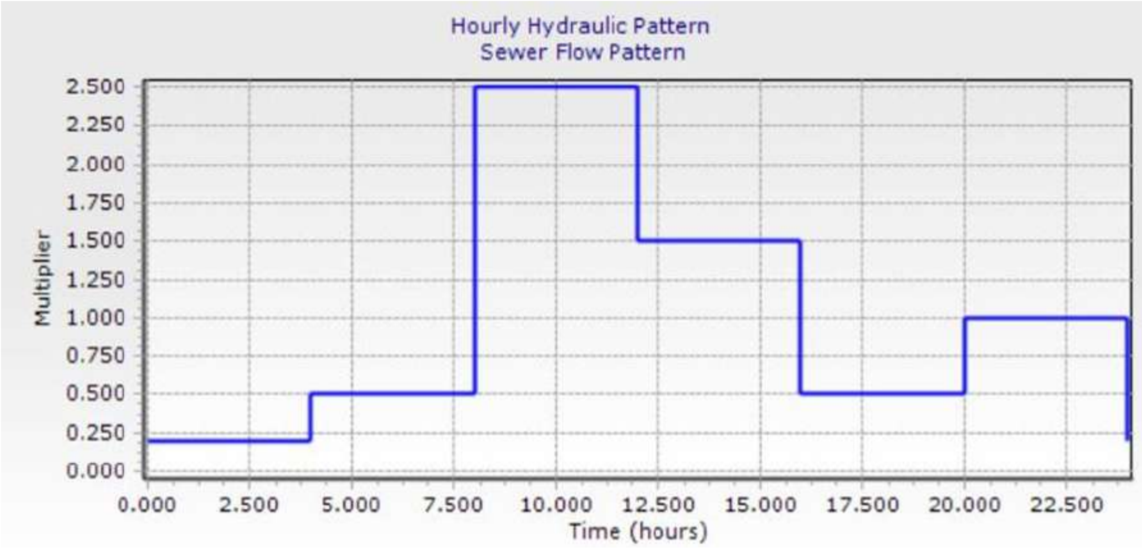

Figure 5-2: Sanitary Sewer Flow pattern over 24 hours.

5.4.3 MODEL RESULTS

The model was put through rigorous analysis to optimize the pipe sizes which would provide adequate capacity while not over sizing the network to keep the cost at a minimum. The software can run an analysis in two modes - a steady state mode and an extended period simulation mode. In steady state, the model is analyzed for a single point in time. Such analysis is useful during the construction of the model. However, it lacks the capability to predict how the model would respond at different flow at different time of the day. For this, an extended period simulation analysis is required. In an extended period, simulation, the model is analyzed at different times with the flow specified. The result from an extended period simulation can be used to optimize the network to provide adequate capacity while keeping the cost at a minimum.

a) SEWER NETWORK

An extended period simulation was carried out for Ammochhu sewer network. The result of the process is the network shown in Figure 4-1[13] . The figure shows the network of pipes, their sizes, the location of manholes. A detailed CAD file with the precise location, pipe sizes, pipe length, manhole location, invert depth, Rim elevation was provided as part of the final report.

b) SUMMARY OF SEWER PIPE REQUIREMENT

The table shows the summary of pipe sizes along with the quantity that is required for the network. A minimum pipe size of 160mm was used. However, the use of 160mm pipes were limited as the slope that was required to implement lead to very

[13] For a detailed drawing of the network, please check the engineering drawing provided with this report.

deep manholes, which would cancel any cost saving by using smaller sized pipe due to increase in excavation cost. Therefore, the 225 mm pipe sizes were chosen in most cases to minimize the depth of manhole. A shallow depth manhole would make maintenance easy as well as reduce the cost of construction. The summary of the total length of pipe required for different diameter is given in Table 5-5. The detail on pipe details can be found in Annexure E of this report.

Table 5-5: Summary of Quantity of pipe requirement.

Pipe Diameter (mm)	Pipe Length (m)
160	1131
225	1864
250	490
315	1074
450[14]	175

c) Hydraulics Profile

A hydraulic profile shows the depth of water flowing in the pipes and manholes. It shows how full the pipes are at different points along the pipe and if there is flooding at the manholes. An under-capacity network would show flooding at manhole and would represent over flowing at the specified manhole. The network for Ammochhu was optimized so that the pipe would flow full at peak flow but would not overflow at the manholes. The hydraulic profile for the main sewer trunk line is shown in Figure 5-3 and Figure 5-4. Figure 5-3 shows the hydraulic profile during peak flow. As can be seen from the figure, the pipes in the downstream is full during the peak flow, however, the water doesn't rise to the top of the manhole. Figure 5-4 shows the hydraulic profile during off peak flow and the displays the minimum flow in the pipe at those times.

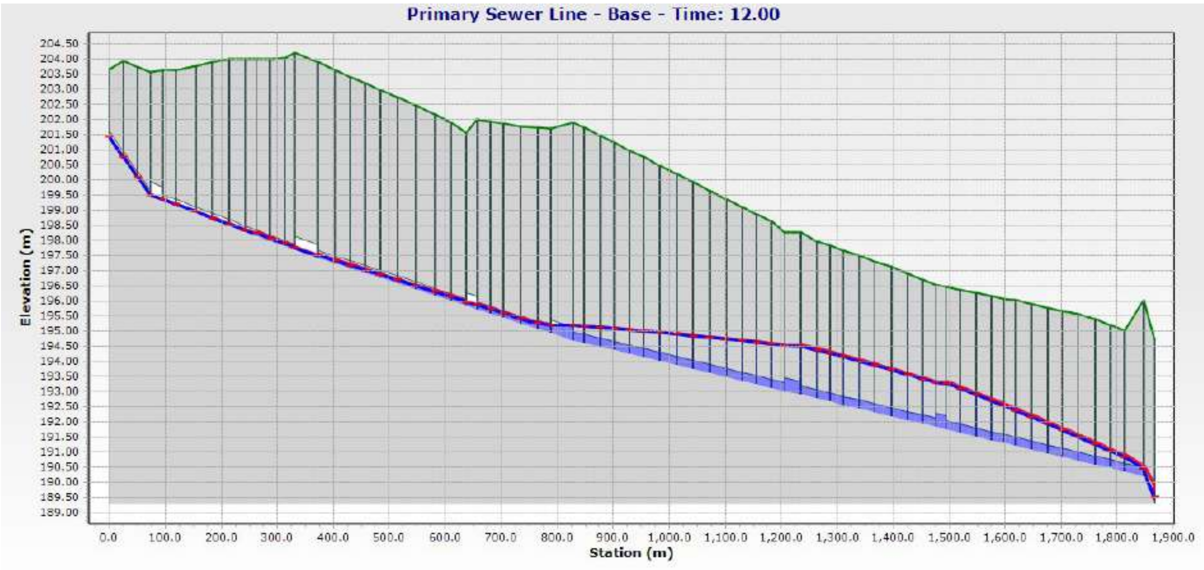

Figure 5-3: Hydraulic profile during peak flow.

[14] RCC Hume pipe

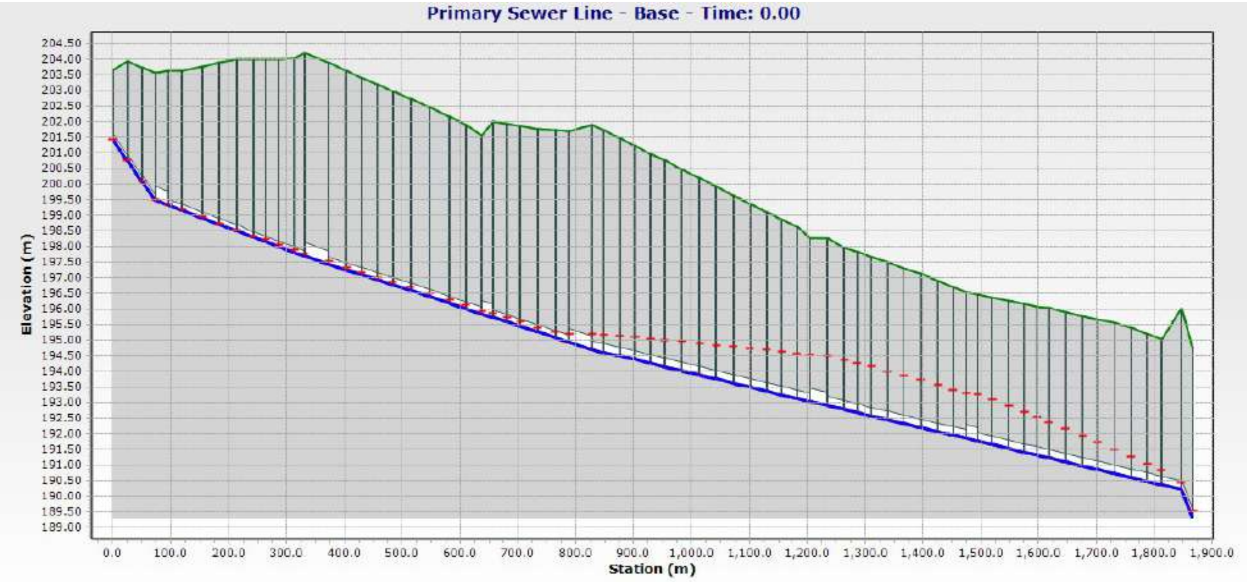

Figure 5-4: Hydraulic profile during Off peak demand

5.4.4 Sewer Pipe at Drain Outfall

Any designed dip in a gravity sewer is referred to as an inverted siphon or depressed sewer (refer Figure 5-5). This occurs when the sewer must pass under structures such as other pipes, highways, or subways; a river; or across a valley. The sewer line is below the hydraulic grade line and is always filled with sewage and under pressure. An inverted siphon is modeled hydraulically by accounting for entrance losses, pressure flow through the siphon, exit losses, and a transition to open-channel flow.

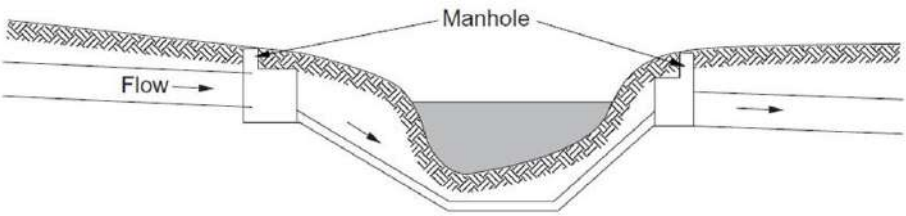

Figure 5-5: Sewer inverted Siphon crossing under a stream

To avoid blockage of the sewer pipe by grit and other material collecting at the low point within the siphon, the fluid velocity in any of the inverted siphon barrels must be greater than 1 m/s at least once each day. However, this depends upon the slope of the outlet side of the inverted siphon and the fluid's ability to move the grit up and out of the inverted siphon. In addition, it is common to place manholes at both ends of an inverted siphon to allow easy access for cleaning the siphon.

For Ammochhu LAP, a design proposal with two depressed sewers were proposed at outfall 06 and outfall 05 as the invert elevation of the manholes on opposite side of the outfall drains were higher than drain invert elevation. At all other outfalls, the sewer pipe runs below the bottom of the outfall drains, so no depressed sewers were required.

However, during the presentation of the design, there were concerns on the reliability and blockage in the depressed sewers. Therefore, an alternate design which doesn't require depressed sewer at any of the outfall was considered and updated for final implementation.

The main sewer trunk line along the western avenue was dropped by 1.4m allowing the sewer pipe at outfall 06 and outfall 05 to fall below the drain invert elevation. A uniform increase in the invert elevation of all the manholes along the western avenue meant that it retains the same hydraulic grade line and provides the same capacity as the previous design. The invert elevation of the last manhole which connects to PTDP's sewer manhole was checked to make sure that it was higher and avoid requiring a pumping station. After uniformly increasing the invert of all the manholes along the western avenue, there was still a 0.9m elevation different between the invert elevation of the last manhole in Ammochhu LAP and the receiving manhole in PTDP network, providing sufficient slope to convey sewage from Ammochhu LAP to PTDP's network without the need for pumping.

A 450mm diameter Hume pipe under all outfalls is proposed. Since the outfall drains are proposed to be constructed before the sewer network, it is proposed that the Hume pipes under the outfall drains be laid during the construction of the outfall drains. The details of the Hume pipe, such as the depth and size along the outfall drains are provided.

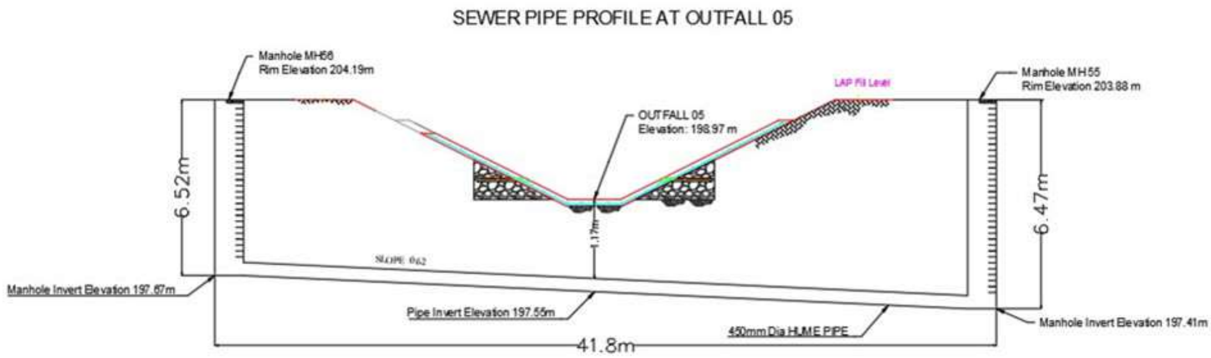

Figure 5-6: Sewer pipe under Outfall 5

5.4.5 DESIGN FLOW VS FLOW OVER TIME

In the design of the sewer networks, a worst-case scenario was used for the calculation of flows. In this scenario, it was assumed that all area in the LAP which can be developed would be built with structures allowed by their precinct designation. Based on the median family size of Phuentsholing and the number of units available in each building, the number of residents and thus, the flow was calculated. Such a projection means that more than 21,000 people would reside in Ammochhu LAP alone which is highly unlikely given the past trends. For comparison, the population of Chhukha Dzongkhag in 2017 was 27,652. However, designing a network which would be able to handle the worst case would be in a position to handle the surge in flows during storm events. Therefore, the networks were sized to cater to the worst-case scenario and provide adequate capacity at all times as the area develops in the future.

However, the total flow from Ammochhu LAP and backside Dhamdara into PTDP's sewer line would only be a fraction of the worst-case flow. The worst-case flow would only happen in the future when the LAP is fully developed. Therefore, to project the realistic wastewater flow volume from Ammochhu LAP and backside Dhamdara over time, it is more practical to use population data and the projected growth rate.

Therefore, combining past population growth trends with plausible future rate of development in the LAP, a population projection at various time period can be calculated. As noted in the Ammochhu LAP report, the population of the LAP was 518 in 2017. Taking the growth rate of Phuentsholing at 2.5%, the population is expected to be 915 by 2039. However, such a projection would undercount the future population as the area is likely to develop at a much higher pace once the infrastructure such as roads, water supply and sewer networks are in place. One case in point is the recent development of 106 units by National Housing Development Corporation Limited (NHDCL) in the LAP. The population is projected to increase by 477 (calculated based on the median family size in Phuentsholing) after the completion of the units. This implies a doubling of population in the last five years due to the construction of the residential colony. In addition, with the development activities of the PTDP project, a rapid rise in population growth is also expected.

An estimation of the current population of the LAP based on the permanent structures in the LAP is about 1085. An estimate based on the 2017 data plus the additional residents due to NHDCL colony adds to 1063. Therefore, the higher value of 1085 was taken as base 2022 population. A conservative growth rate of 6.0% (which is more than twice the growth rate of Phuentsholing) was taken to calculate future population. A higher growth rate is taken due to the fact that Ammochhu LAP will likely develop much quicker based on the location as well as the topography of the area.

Similar assumptions were made for backside Dhamdara and an estimate of the current population was determined to be 350. A slightly smaller growth rate of 5% (which is twice the average growth rate) was taken due to the fact that the area is a more representative of the overall topography of Phuentsholing. Therefore, a lower rate of growth would be predicted compared to Ammochhu LAP. Using these estimates of the current population and a conservative growth rate, the following flow volumes were calculated as given in Table 5-6

Table 5-6: Projected flow volume from Ammochhu LAP and backside Dhamdara.

Year (Years from 2022)	Ammochhu LAP (projected Population)	Ammochhu LAP Flow (MLD)	Backside Dhamdara (Projected Population	Backside Dhamdara Flow (MLD)	Total Flow (MLD)
2022 (0 year)	1085	0.17	350	0.06	0.23
2027 (5 years)	1451	0.23	446	0.07	0.30
2032 (10 year)	1943	0.31	570	0.09	0.40
2042 (20 years)	3479	0.56	928	0.15	0.71
2052 (30 years)	6231	1.00	1512	0.24	1.24

The worst-case flow used in the sizing of the network for Ammochhu was 3.25 MLD and the flow from backside Dhamdara was 0.25 MLD for a total of 3.5 MLD. However, as discussed above, it is highly unlikely that the LAP would generate this worst-case flow in the next 30 years.

The more realistic figure given in Table 3-7 is likely to be the true volume which would be generated from the LAP and backside Dhamdara. Therefore, if the sewer networks were to be built today, the flow into PTDP sewer line would be about 0.23 MLD. A steady rise in the flow volume is expected with a high of 1.24 MLD by 2052. The wastewater from the LAP flows into manhole labelled A1-11 in PTDP's main sewer line along PCR road.

5.4.6 Sewer Flow from the LAP

During discussion with PTDP's officials for the connection of flow from Ammochhu LAP into PTDP's sewer line, it was mentioned that the arrangement to accept the flow was a temporary measure. During the design of PTDP's sewer network, only 0.6 MLD capacity was kept for Ammochhu LAP. Therefore, if the volume from Ammochhu LAP was to exceed the 0.6 MLD, PTDP's sewer network would no longer be able to accept the extra flow.

The flow from the Ammochhu LAP and backside Dhamdara would exceed the 0.6 MLD limit in the next 10 to 20 years. Therefore, once the limit is reached, it is proposed that Thromde look into conveying the sewer flow from the Ammochhu LAP and Backside Dhamdara to the core town STP via a new separate sewer trunk line. The flow in the sewer network would increase gradually as the area develops, therefore, Thromde should plan accordingly in consultation with PTDP as and when the flow reaches the limit set by PTDP.

5.4.7 Lateral/property Connection

The connection to property is provided via inspection chamber located near the property boundary on either side of the sewer line. The inspection chamber connects to the manhole via a 150mm diameter HDPE pipe. The inspection chambers are 1.2m in depth to allow for sufficient slope to collect wastewater from the residents. The location of inspection chambers and the property served are detailed in the CAD drawing provided as part of the report.

In areas where the inspection chambers are not located close to a property, easement rights would need to be implemented to install sewer pipe to connect the property to the chamber along property boundary. Such plots are annotated in the master plan of the sewer network. However, if it is not feasible to connect the network to the via the property boundary, the status quo of using septic tanks for wastewater collection is recommended. The detailed design of inspection chamber is provided as part of the final report.

5.5 Bill of Quantities and Estimate

The total cost of implementing the Ammochhu LAP sewer network is Nu.43,364,675. The summary of the breakdown of cost is given in Table 5-7.

Table 5-7: Summary of Cost.

Items	Cost (Nu)
Pipes Works	27,443,702
Manholes	15,920,974
Inspection Chamber	4,867,314
Grand Total	**43,364,675**

6 SERVICE UTILITY DUCT

6.1 Introduction

Settlements are aligned along the road. Unplanned, uncoordinated and lack of standards to design and lay the utility and service infrastructure have resulted in repeated digging of the roads, not only damaging structure but also affecting public safety. Moreover, with demand of urban residents for prompt and efficient delivery of service, it has become necessary to design and built integrated system where all utility and service line can be laid.

The advantages of such integrated duct are as follows;

- Convenient laying of pipes and cables by various agencies without having to dig the roads every time new connection has to be made.
- Easy repairs and laying of new pipes and cables in the future.
- Reduction in the recurring cost, and
- Proper coordination amongst the different utility services providers.

6.2 Existing Electricity & Telephone line

There are six existing substations in the LAP, out of which one privately owned and other fives belongs to BPC. Currently the BPC electricity distribution overhead lines run along Eastern avenues.

Similarly, the telephone distribution overhead lines run along the eastern avenues together with BPC electricity.

The Figure 6-1 shown the existing BPC substations in the LAP. Out of these six substations, two substation PHE30T23 & BO30H119 fall in the private plot and in center of proposed road respectively.

Figure 6-1: Existing BPC substations in the LAP.

For these two substations, consultation was done regarding the relocation with BPC Official. Initially the consultant identified nearby state land and provided probable land for relocation and submitted to BPC.

After detailed study and investigation, BPC has provided the new locations of these two substations which is given in the proposed Service Utility Duct network.

Figure 6-2 & Figure 6-3 shows existing BPC substation which need to be relocated.

Figure 6-2: Existing Substation (BO30H119) BPC near NHDCL colony which fall in the center of proposed road.

Figure 6-3: Existing Substation (PHE20T23) which fall in middle of private plot.

6.3 Design Consideration

For Integrated Duct design, due consideration on Electrical safety rules & regulations are must so that authorized/competent person can perform installation, operation, maintenance & inspection safely. Moreover, it is must for it to perform its function safely, effectively & efficiently and for public safety.

As proposed Integrated Duct accommodate high voltage underground power cables. Underground cables though eliminate the electric field altogether as it is screened out by the sheath around the cable, but they still produce magnetic field. So underground cable should be typically installed at its minimum burial depth.

As per Bhutan Electricity Authority Safety Code 2008, minimum depth of Burial of 33kV underground cable shall be 1.0m below ground. Similarly, as per Indian Standard Code of Practice for Installation & Maintenance of Power cables up to and including 33kV rating (IS: 1255-1983), the desired minimum depth of laying from ground surface to the top of the cables are as follows;

- High Voltage cables, 3.3kV too 11kV rating: 0.9m
- High Voltage cables, 22kV to 33kV rating: 1.05m
- Low Voltage and control cables: 0.7m
- Cables at road crossings: 1.0m.

The desired clearance from power cable to communication cable shall be 0.3m and power cable to gas/water main shall be 0.3m.

The space provided for cable racks has to be sufficient. They are generally fixed to the wall or supported by standing columns or structures enabling easy installation or replacement of cables.

The vertical distance between two racks should be minimum 0.3m and the clearance between first cable and the wall should be 25mm. The width of rack should be not exceeding 0.75m in order to facilitates installation of cables.

In the case of “Passable’ cable duct, the head room should be not be less than 2m and width sufficient to leave a free passage of that least 600m to 800mm either from one side or in the middle. Due consideration should be given for adequate ventilation, lighting and drainage.

When three single-core cables laid in one plane, the spacing between the cables should not be less than cable diameter.

The desired horizontal and vertical clearance for telecommunication optics cables is similar as for electricity power cables. To reduce interference of both electricity and telephone/cable-television/data cables, the telecommunication cables should be rack from separate wall. Its relative location to other service should be at a distance of 500mm.

Where electricity power cables and communications cables must be racked from same wall, the electricity power cables should be racked below the communication cables.

6.4 Proposed Service Utility Duct

The proposed Service Utility duct is reinforced concrete duct which run along the roads which will be used as footpath.

6.4.1 THE SERVICE UTILITY DUCT NETWORK

The proposed Service Utility Duct (see Figure 6-4) is placed on left-side of RoW of Eastern & central Avenues. While in case of Western avenues, the proposed Service Utility Duct isn’t placed either side of footpath in the RoW of road. It is paced under footpath provision in PCR four-lane Highway due to lack of space underneath footpath in Western Avenue.

The Service Utility Duct along Eastern, Central & PCR are interconnected by at various location by running Service Utility Duct parallel to Outfalls.

Along Outfall2a and access road toward northern end near NHDCL Colony, the provision is kept in the Service Utility Duct for running 33KV high tension underground cables for PTDP.

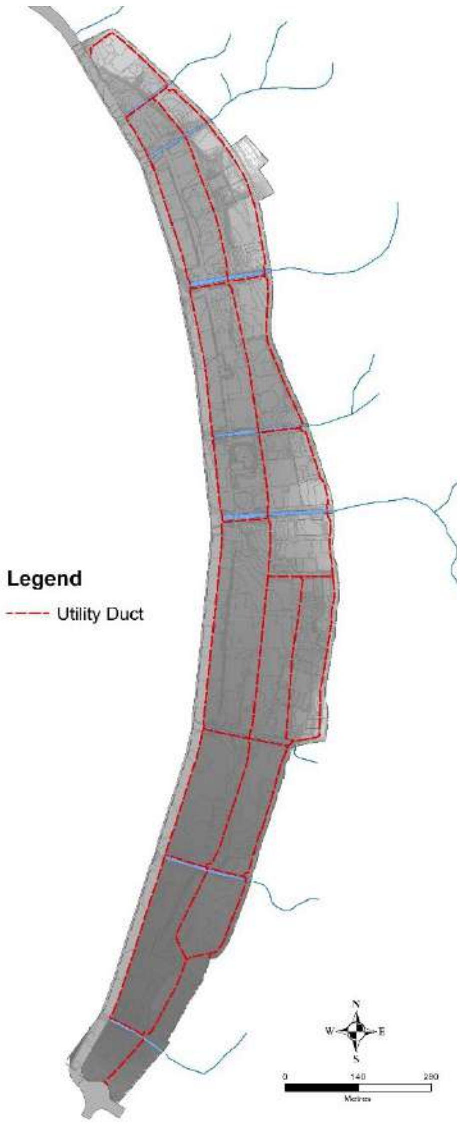

Figure 6-4: Proposed Service Utility Duct Network.

6.4.2 PROPOSED SERVICE UTILITY DUCT SECTION

The proposed Service Duct section is shown in the Figure 5-5 below. The overall width & Height of Service Utility Duct is 1200mm. The drain RD type always comes with Service Utility Duct & hence the drain RD type should be constructed together with Service Utility Duct.

Both drain RD type and Service Utility Duct follow the design road profile and during the construction the invert level of Service Utility Duct should be considered from the design road levels.

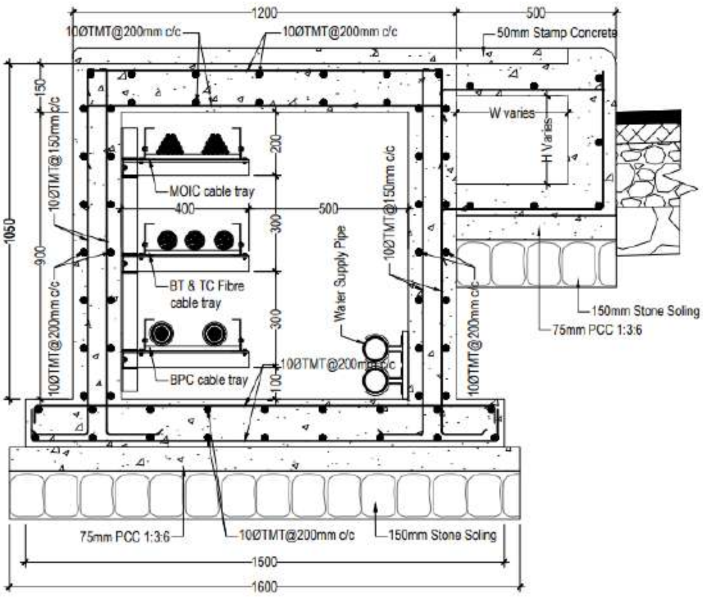

Figure 6-5: Typical Service Utility Duct Section.

6.4.3 MANHOLES

The Service Utility Duct shall be constructed at 15m interval. The Manhole spacing of 15m was considered as it will be easier & Convenient to lay underground cables, easy to detect fault & fix the fault in Underground cable and enable proper and timely inspection & maintenance the Utilities services.

At road crossing & Outfall Service Utility Duct shall be discontinued. The utilities services will be laid under road pavement due to which the overall of height of Manhole will be 1.5m in case of Utility Duct along flexible pavement and 1.35m in case of duct along Cobble Street.

Figure 6-6 & Figure 6-7 shows typical Service Utility Duct Plan & Section details.

6.4.4 PROPERTY CONNECTIONS

At every Manhole two number 90mm HDPE pipe outlets staggered 300mm center to center both horizontally & vertically has been provision for property connections.

While this provision shouldn't be provided for manhole at road & Outfall crossings. It shouldn't be provided for manholes on Service Utility Duct which run parallel to Outfalls.

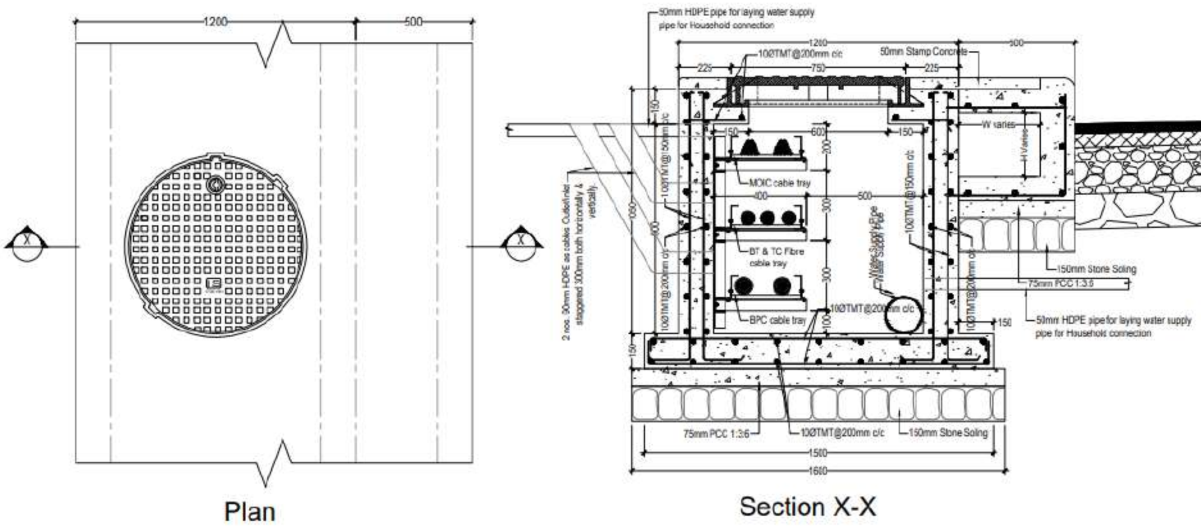

Figure 6-6: Typical Service Utility Duct Manhole Plan & Section.

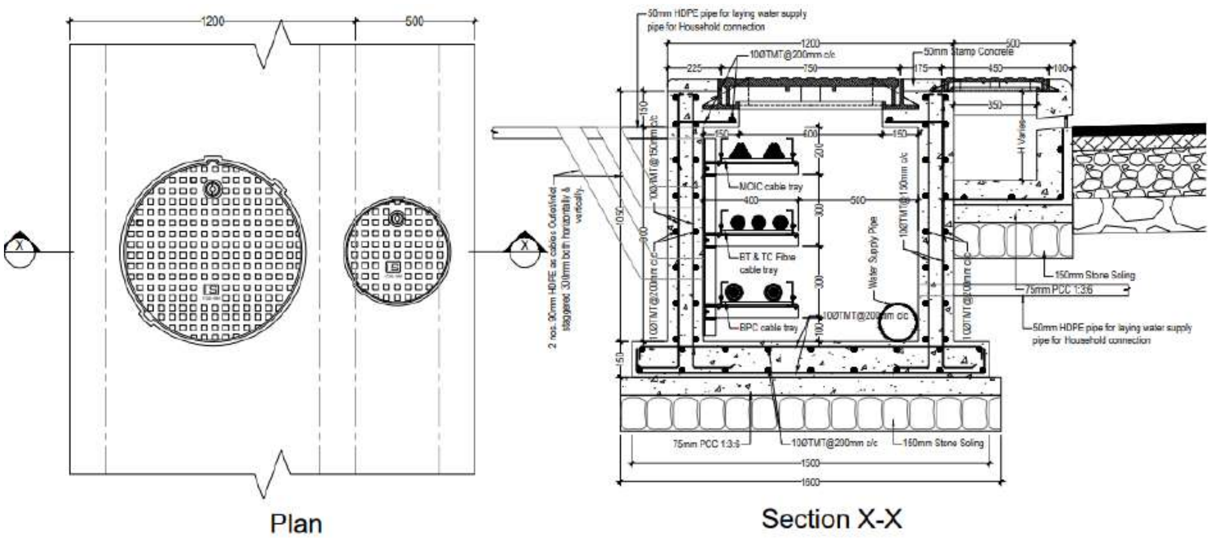

Figure 6-7: Typical Service Utility Duct Plan & section with RD type drain's manhole.

6.4.5 Proposed New Substation & Relocation of Existing Substation

The existing BPC substation BO30H119 was relocated into the state land below NHDCL colony and substation PHE30T23 was relocated to state land on left-side of outfall 6. While the new substation is proposed in the state land at southern end of the LAP.

The relocation of these two existing substation and new substation is provided Service Utility Duct network AutoCAD drawing. Refer it for details.

6.4.6 Proposed Ring Main Unit (RMU)

RMU (Ring Main Unit) cabinet or medium voltage cabinet is integrated electrical equipment to connect, measure, and integrate fixed type breaker with transformer protection function.

RMU cabinets are small in size, highly reliable, safe, easy to maintain, easy to replace, and expand. Depending on requirements, the RMU is available in different voltages suitable for both indoor and outdoor installations.

The RMU cabinet is connected by output and input cable compartments by isolating breakers and cables out to transformers using load breakers and protective fuse.

There are three RMU proposed for the LAP & its locations are given Service Utility Duct Network AutoCAD drawing. The RMU is proposed on the state land and space requirement is 3.75mx1.985m as per BPC standard.

6.4.7 Proposed Unitized Substation (USS)

Unitized Substation is a single substation factory build package having transformer, HT and LT controls.

The Unitized Substation (USS), is designed to provide safe, reliable, compact and economical electrical power distribution for industrial plants, commercial buildings, outdoor lighting and any other general purposes power distribution applications.

There are six Unitized Substation proposed for the LAP. Its locations are on state land & require 5.3mx4m space. The location of USS should be referred from the Service Utility Duct Network AutoCAD drawings.

6.4.8 Road Crossings and Outfall

At the road crossing the Service Utility Duct should be discontinued and the utilities should be laid inside 160mm HDPE pipe. The underground power cable should be placed 1m below the road pavement surface.

When stormwater drain and utility service HDPE pipe crisscrosses each other, the Utility Pipe should be placed below the stormwater drain.

While in case of Outfall crossings, the Service Utility Duct should be discontinued and all the utilities service cable should be taken out through 160mm HPDE provision in the manhole. Once the utilities service cables are out duct, it should be clamped to Outfall's culvert using MS Angles.

On the other side of Outfall's crossing, the utilities service cables should be taken into Service Duct through 160mm HDPE pipe provision as cables inlet.

6.4.9 BPC Power Source

For PTDP project, BPC will supply two 33KV line from Chamkuna and near Outfall 2a. Accordingly, the provision was kept by running Service Utility Duct along Outfall 2a & along Service internal road near Bridge BR01 in the LAP for running BPC supply line to PTDP area.

For the LAP, two sources are identified to supply 11KV line. One toward the northern end near NHDCL colony from Chamkuna and another near Outfall 2.

6.5 Bill of Quantities and Estimate

The Total cost of implementing the Ammochhu LAP's Service Utility Duct is Nu.129,793,009.53. The summary of breakdown of cost is given in the Table 6-1.

Table 6-1: Summary of Cost.

Items	Cost (Nu.)
Earthwork	6,164,574.39
PCC works	3,383,531.31
Stone Works	1,951,446.47
RCC works	79,912,508.99
Other works	36,848,480.82
OHS Cost	1,532,467.56
Grand Total	**129,793,009.53**

7 ROAD DESIGN

7.1 Road Details

The detailed design of roads covers roads under Table 7-1.

Table 7-1: list of roads.

List of Roads				
Road Name	RoW (m)	Length (m)	Carriage Way	No. of Lane
R-1	10.00	2,397.48	6.60	2
R-2	10.00	1,738.31	6.60	2
R-3	10.00	1,935.62	7.00	2
R-4	10.00	333.31	6.60	2
R-5	10.00	124.73	6.60	2
R-6	8.00	135.06	5.00	2
R-7	10.00	84.49	7.00	2
R-8	8.00	88.84	4.60	2
R-9	8.00	40.03	4.60	2
R-10	4.50	173.17	3.00	1
R-11	4.50	171.64	3.00	1
R-12	4.50	212.74	3.00	1
R-13	4.50	212.93	3.00	1
R-14	4.50	155.42	3.00	1
R-15	4.50	155.89	3.00	1
Total length		7,959.65		

7.1.1 Road Widths

The Table 7-2 below shows the minimum width requirement of various road elements as set by Urban Road Standard-2002.

Table 7-2: Minimum width requirement-road elements.

Road Classification and ROW	Max. no. of Lanes	Carriage-way (m)	Footpath, Drain, Shoulder & Median (m)	Minimum Widths
Primary road Minimum-15m Ideal-18m	 4 4	 12.00 13.20	 3.00 4.80	Footpath= 1.20m Drain = 0.30 m Shoulder=0.50m = 0.25 m
Secondary: Minimum-10m Ideal-12m	 2 2	 6.00 6.60	 4.00 5.40	
Access: Minimum-6m Ideal-8m	 1 1	 3.50 3.50	 2.50 4.50	

7.2 Ancillary Road Works

The design of road works includes ancillary items within the ROW, such as retaining structures, road side footpaths & road side drains.

Random Rubble Masonry wall is proposed as a retaining structure. The location of the retaining structures is identified. The walls to be constructed fully within the RoW to avoid encroaching into neighboring plots.

7.2.1 Roadside Footpath

Road side footpath provision is maintained as per the Urban Design.

7.2.2 Roadside V-drain

Road side V-drain is replaced with covered box drain. This enables to have more space for the road carriage way. The decision was taken, during the first draft meeting.

7.2.3 Retaining Wall

Random Rubble Masonry wall is proposed as a retaining structure. The location of the retaining structures is identified. The walls to be constructed fully within the RoW to avoid encroaching into neighboring plots.

7.3 Geometric Design

7.3.1 Road Features

- Road Right of Way - 10.00m, 8.00m, & 4.50m
- Carriage way width-7.00m, 6.60m, 5.00m, 4.60m & 3.00m.
- Pavement type - Flexible & cobble surface
- Design speed - 10-20km/h
- Pavement cross fall-2.50%
- Exceptional minimum curve radius - 10m
- General curve radius - 100m
- Maximum road gradient -9.50%

7.3.2 Use of Software

The design of vertical and horizontal alignment of all roads, and the calculation of earthworks quantities, has been undertaken using MX ROAD software.

7.3.3 Vertical Alignment

The vertical alignment is fixed closely with the existing ground profile to maintain the plot access and to avoid deep cutting and filling.

All the roads are designed within permissible limits as highlighted under Urban Road Standard-2002, as tabulated below.

The road R1 or the eastern avenue road is only the road existing on the ground. For this road, the vertical alignment is designed considering the control levels of the existing culverts and buildings.

The vertical alignment design has also incorporated the storm drainage levels requirement.

Table 7-3: Maximum gradients for different road classes as per URS-2002

Road Classification	Maximum Recommended	Desirable Maximum	Exceptional Maximum	Maximum Distance
Primary	9%	6.6%	10%	90 m
Secondary	9%	6.6%	11%	75 m
Access	9%	6.6%	12.5%	60 m

7.3.4 HORIZONTAL ALIGNMENT

The horizontal alignment is defined by the layout plan and RoW of roads. The horizontal alignment is designed with the concept of best fit of straights and curves along the ROW of each road.

7.4 Road Pavement

The design of road pavement is in line with the Urban Design except for Road R2 or the central avenue road, the entire road pavements are design for flexible pavement. The central avenue road is design with cobble road surface.

The selection of pavement thickness is tabulated below;

Table 7-4: Pavement thickness

Pavement Thickness (N=5 msa, CBR=10%)				
Reference	GSB (mm)	WMM (mm)	DBM (mm)	AC (mm)
IRC-37	150	250	40	25
DoR Pavement Chart	150	200	60	40
URS-2002	170	230		
Adopted	**150**	**200**	**60**	**40**

7.5 Earthworks

The volume of earthworks (cutting/filling) have been minimized by setting the vertical alignment of closer to natural ground. The DTM model is generated after proposing the LAP filling equivalent to PCR level.

The road is designed over the filled model of PCR levels. The earth work quantities for both cut and fill volume is work out using MX ROAD software.

7.6 Bill of Quantities and Estimate

The completed Bill of Quantities is prepared. The cost estimate is based on BSR 2022 rates with 15% cost index.

The total cost of implementing the Ammochhu LAP's Road network is Nu.181,099,207.25. The summary of the cost breakdown of cost is given in Table 7-5.

Table 7-5: Summary of Cost.

Sl. No	Particular	Amount (Nu.)
1	Road & Pavements	50,600,588.47
2	Bituminous Works	40,454,276.28
3	Cobble Road	22,502,791.31
4	Retaining Wall	67,541,551.20
	Total	**181,099,207.25**

8 STREET LIGHTING & CCTV

8.1 Proposed Street Lighting Network

The proposed street lighting in the LAP is shown in Figure 8-1.

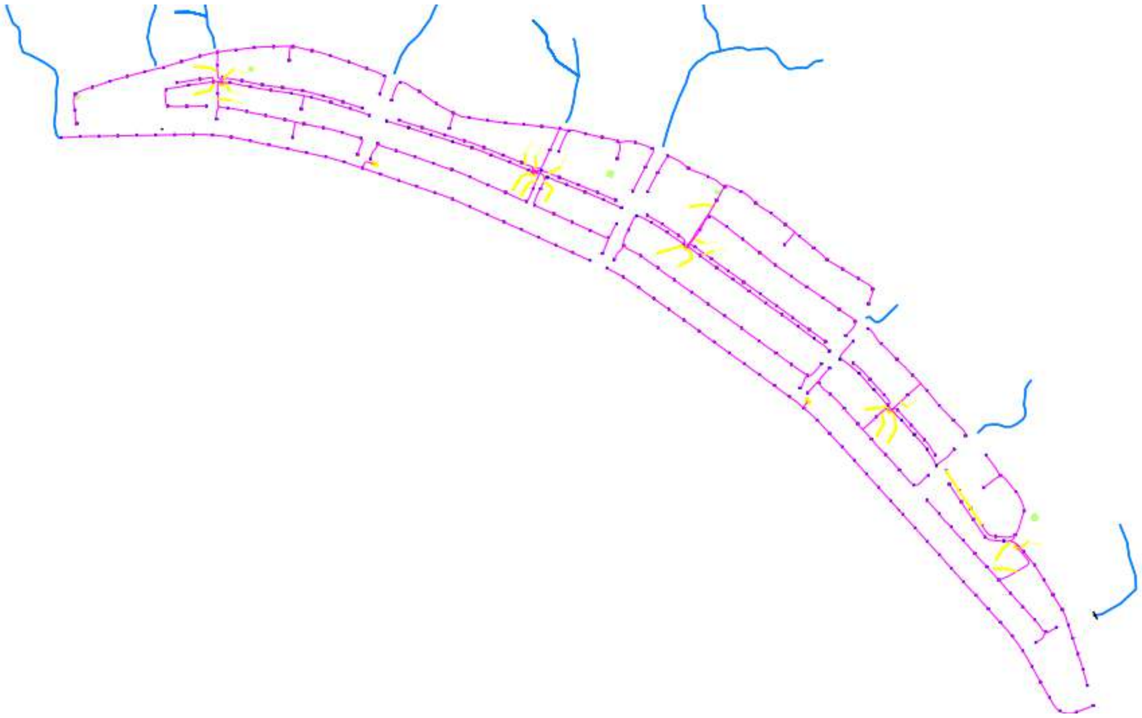

Figure 8-1: Proposed Street Lighting.

8.2 Street light specifications

8.2.1 STREET LIGHT POLE

The street lighting poles shall be manufactured from steel sheets and having 8-meter height galvanized steel shaft octagonal cross section and tapered from bottom to top, and made of one piece. The bottom and top diameter should not be less than 130mm and 70mm respectively. The thickness of the shaft wall will be 3mm minimum.

The poles shall be hot-dip galvanized, all component must be well protected against corrosion, and minimum thickness of zinc coatings is 85 µ m and min density 500 gm /m2 on both inside and outside surfaces.

The octagonal Poles shall have inbuilt junction box, 10A 2P MCB and door of approximate 500mm length at the elevation of 1.5m from the ground surface. The door shall be vandal resistance and shall be weather proof to ensure safety of inside connections. The door shall be flush with the exterior surface and shall have suitable locking arrangement. There shall also be suitable arrangement for the purpose of earthing.

The poles shall have a base plate welded to the lower part of the shaft from outside and inside to serve for fixing the shaft to concrete foundation by means of four bolts, thickness of the plate shall not be less than 12mm, and its diameters are 250mm with a center distance 180x180 mm between holes, the shapes of holes are oval.

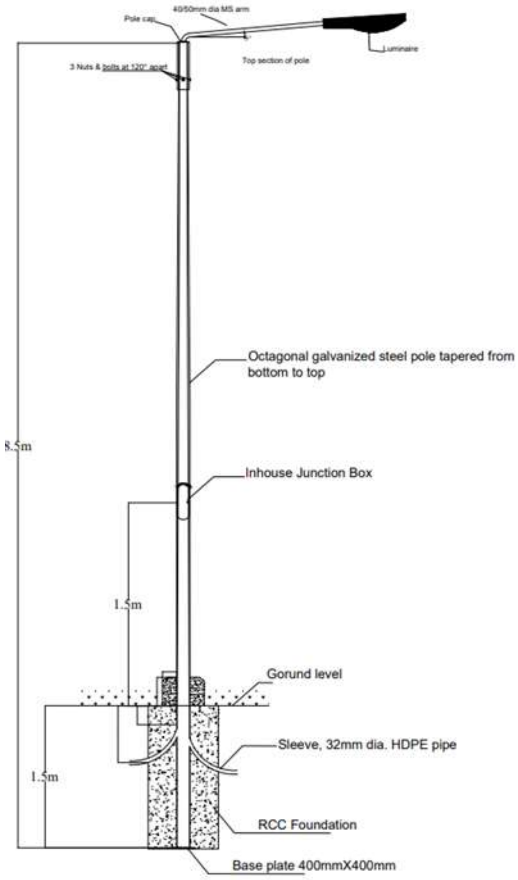

Figure 8-2: Typical Street Light Elevation.

a) Brackets

The Pole shall be fitted by single or double or triple brackets made of galvanized steel tube with arm outreach of 1.5m. The arm mounted on the top of the shaft and able to support single or double or triple lanterns of 60mm side entry, the arm shall hold in its position by headless screws. The angle of the arms should be design as per the road alignment and submit to the client for approval.

b) LED Street light Luminaries

- Input voltage: 230 V
- LED lamp efficacy: Min 115 lumens/watt.
- Power factor: >0.90
- Life expecting: 50,000 hours.
- No of hours usage /day: 10 to 12 hours/day.
- Rated watt: 90 W (cool white).
- LED type: High power LED (1 Watt).
- Colour Temperature: 4,500 to 7,000 K with test certificate.
- Ingress protection: IP 65/IP 66 as per IS/IEC60529-2001 with test certificate.
- Average lighting/ beam angle: 120 to 160 degrees.
- Luminary Casing: Pressure die cast Aluminum with toughened glass cover and water proof fixture.
- LED thermal management: LED shall be mounted on heat sink conductive aluminum with fins to dissipate the heat to ambient air.
- Warranty: 2 years.

8.2.2 Decorative Pole for Central Avenue Road

The 3.5m high, 76mm pole top dia. for post top lamp shall be aesthetically designed decorative hot dipped galvanized mild steel step pole with grey powder finished, Compatible for all Havells post top range, Window - Plasma cut & duly chamfered opening window of 300x110 mm for terminal connections & mounting accessories. Window cover is fitted with allen screws to the pole, inbuilt 4-ways terminal block, 6A 2P MCB, minimum 1-meter long 32mm diameter HDPE pipe for cable sleeve (in & out), complete with all accessories. Cat. No. or ordering code: LHMP02135099, Havells make.

a) POST TOP LAMP

- Input voltage: 230 V
- LED lamp efficacy: Min 115 lumens/watt.
- Life expecting: 50,000 hours.
- Rated watt 60 W (cool white).
- Colour Temperature: 4,500 to 7,000 K with test certificate.
- Ingress protection: IP 65 as per IS/IEC60529-2001 with test certificate.
- Luminary body: Pressure die cast Aluminum.
- Ordering No. LHEVAHP7PN6J060, Sapphire-IP65.
- Warranty: 2 Years

8.3 Proposed CCTV Network

The CCTV is proposed along all the internal service roads and in the median of PCR four-lane highway as shown in Figure 8-3 .

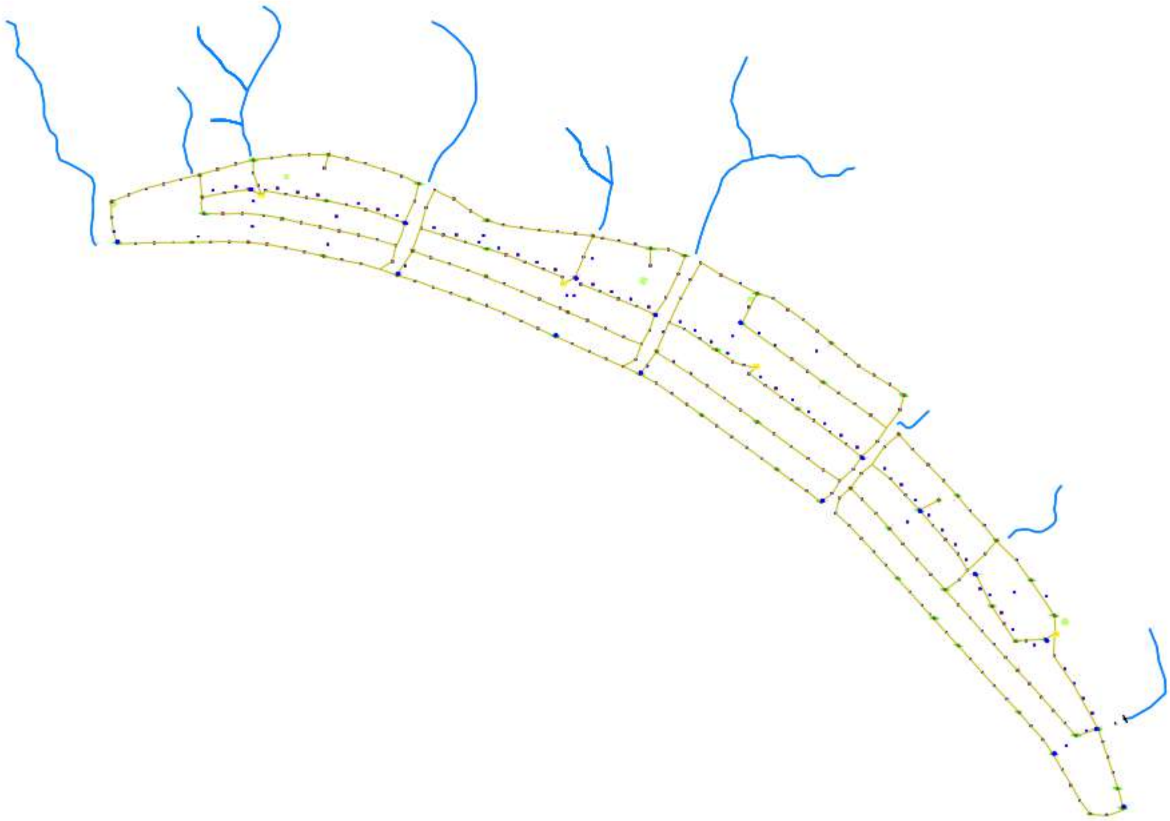

Figure 8-3: Proposed CCTV network

8.4 Technical Specifications

The Technical Specification of CCTV will be provided with the Bill of quantities.

8.5 Bill of Quantities & estimate

The total cost of implementing the Ammochhu LAP Street lighting & CCTV is Nu.52,199,762.74.

9 SOLID WASTE MANAGEMENT

9.1 Introduction

Waste is an issue at Ammochhu LAP area as there is no segregation of waste at source. With the change in consumption patterns and population growth, waste generation has increased over the years everywhere, including Phuentsholing. The increasing number of various kinds of waste generation from homes, offices and commercials establishments in the area puts additional pressure on the existing system.

9.2 Existing Scenario and Waste Issues

Currently, the LAP area does not have proper solid waste management scheme apart from mere establishments of various communal bins at various locations of the towns. At present, wastes are not collected properly and thus resulting in polluting the environment making the surroundings unhygienic and filthy. Moreover, nearby the Ammochhu LAP, currently solid wastes are disposed into drains resulting into blockages of drains and causing local floodings. Leachates of pollutants and oils from the nearby workshops and garages directly flows in the streams and river water nearby polluting the waterbodies. Therefore, it is essential to propose some form of SWM plan before the detailed SWM plan is proposed by the Thromde for the Ammochhu LAP.

9.3 Proposed Solid Waste Management Plan

Phuentsholing Thromde has plans to introduce segregation of dry and wet waste at source and increase frequency of waste collection trucks in Ammochhu LAPs during the preparation of LAP to address the waste issues which will become prominent after the construction and when the LAP becomes fully functional. Therefore, the following methods are suggested to tackle the issues on solid waste in a better way.

9.3.1 SEGREGATION OF WASTE AT SOURCE INTO THREE CATEGORIES

Segregation of waste is essential as the amount of waste being generated today is of various types due to change in consumption pattern and imports of various materials from neighboring countries. Waste at source should be segregated into at-least three categories i.e., dry, wet and hazardous waste. Waste, if segregated at source, improves collection efficiency and leads to better processing of waste. This system is now already set up in Thimphu Thromde as shown in Figure 9-1 below.

Figure 9-1: Three different categories of waste as per NECS in 2022.

A separate communal bin for different categories of waste as shown in Figure 9-2 should be kept at various locations. Waste from these bins should be monitored and timely collected by the waste collection trucks in order to avoid overflowing of waste from the bins like the current scenario occurring in various parts of the town. Since wet wastes from kitchen are found to be at larger quantity than any other waste in all places, decomposing of wet waste at source and using it at organic manure for garden and campus beautifications are recommended as it decreases the pressure on the landfill.

Figure 9-2: Separate bins for different categories of waste.

9.3.2 IMPROVING WASTE COLLECTION FREQUENCY AND REDESIGNING COLLECTION ROUTE.

The collection route will be redesigned significantly based on actual number of households in each zone. All three types of aforementioned waste should be collected separately by the Thromde waste collection trucks. The frequency of the waste collection trucks to the area has to be increased to at-least 3 times in a week, twice a week for wet waste collection and one day for dry and hazardous waste which can be collected together but segregated later.

To cater to the demand for increase in frequency of waste collection vehicles and coverage across the area, it will be necessary to procure additional waste collection vehicles to cater to all the households in the LAP. It is also necessary to maintain budget for fuel and maintenance to provide efficient service. It is also recommended that attention could be paid to explore the use of electric and other alternate fuel vehicles for waste management.

The Thromde management with the support of the Environment Officer should also educate the LAP residents on the concept of waste reduction, reuse and recycle (3Rs). In addition to that, the residents should then be taught how to segregate their waste into the 3 categories of waste using the nationally accepted norms. As of May 2021, the waste flagship project is being rolled out by the Officer of the Prime Minister and National Environment Commission Secretariat. The Thromde could seek support from the programme to ensure that the best practices and methods are used in Ammochhu LAP along with Phuentsholing LAPs. The latest classification of Hazardous waste would also be determined and updated by the programme which shall then be utilized in Phuentsholing as well.

9.3.3 DROP-OFF CENTERS IN THE COMMUNITY

Establishment of drop-off center within the community seems to be an alternative solution for proper solid waste management. A pilot project near Kelki Higher Secondary School in Thimphu has proven to be beneficial for Thimphu Thromde residents. Since 2022, nine waste drop-off centers and 25 waste collection facilities have been set up in Thimphu. Such practices could promote recycling of wastes and reduce waste going to the landfill. Figure 9-3 below shows a new waste drop off center in Thimphu.

Figure 9-3: Drop-off Centre in Thimphu.

In places where it is difficult to put up proper drop-off centers, it would also be ideal to set up waste collection facilities as shown in figure below.

Figure 9-4: A waste Collection facility in Thimphu[15].

Setting up waste drop-in centers at different locations with proper buy-back mechanism in the area would encourage people to segregate their waste and dispose them when waste collection facilities are not available due to breakdown of waste collection trucks or due to some other reasons. The drop-off centers are considered to be self-sustaining in the long run.

9.3.4 Collection System

According to National waste Inventory Survey of Bhutan 2019 (NWIS-2019) published by NSB and NEC 2020, the total waste generation in the country in a day is 172.16 metric tons and per capita waste generation in day is 0.25 kg. On average, each household in urban area is generating 0.7 kg and in rural areas 0.4kg of wastes in a day. Food waste constitutes almost 60 percent of the total waste in the urban households and 40 percent in the rural households.

Based on the above recent data, a proper collection system shall be designed for various zones of Ammochhu LAP.

9.4 Waste generated in a week in each zone of Ammochhu LAP

Using the national figures from the NSB and NECs survey, waste from respective zones of the Ammochhu LAP were quantified. The detailed calculations of waste generated from each zone is given in Annexure F & Table 9-1 shows the wet and dry waste produced in a week in Ammochhu LAP considering the population living in the area. For Ammochhu LAP, 50% of the total waste generated in a week is taken as dry waste and another 50% of the waste is considered as the wet waste. The total waste generated from the LAP in a week is 23,856 kg of wet waste and dry waste each.

[15] NECS,2022; Facebook page of zerowastebhutan.

Table 9-1: Waste generation per Week by zones.

Zone	Population	Wet Waste/week (Kg)	Dry Waste/week(Kg)
Character zone 1	528	462	462
Character zone 2	3696	3234	3234
Character zone 3	7968	6972	6972
Character zone 4	9216	8064	8064
Character zone 5	1536	1344	1344
Character zone 6	4320	3780	3780
Total	27,264	23,856	23,856

The waste collection schedules for different character zones of Ammochhu LAP have been tabulated in Table 9-2. Ammochhu LAP has been already divided into sub areas and waste has been classified into wet and dry waste for collections. Compactor trucks have been considered as the vehicles to collect the waste in shift wise system.

Table 9-2: Waste Collection Schedule for various Zones in Ammochhu LAP.

Area Code	Area	Monday		Tuesday		Wednesday		Thursday		Friday		Saturday		Sunday	
		S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2
A	Zone 1		1				1						1		
B	Zone 2		1				1						1		
C	Zone 3	1					1						1		
D	Zone 4			1		1						1			
E	Zone 5				1	1						1			
F	Zone 6				1	1						1			

Collecting time	Shift1 (S1)	Morning
	Shift2 (S2)	Afternoon
Collection Category	Dry waste	Wet waste
Vehicle type	1	Compactor truck

Wet and dry waste collection route are shown in Table 9-3 and Table 9-4 respectively. Graphically waste collection routes for both dry and wet waste collection routes for all the zones are shown from Figure 9-5 and Figure 9-8. Since most of the dry waste generated will be from homes, offices and commercial centres excluding the industrial wastes containing hazardous chemicals no separate trip for collecting hazardous waste is required. Leachate of pollutants and waste oils from automobile workshops and garages can be routed into proper drains and collected separately in tanks and treated subsequently. The total of four times of waste collection in a week is required with dry waste collecting on Monday and Tuesday, and subsequently collecting wet waste on Wednesday and Saturday.

Table 9-3: Wet Waste Collection Route for all the zones of Ammochhu LAP.

Area	Route
Zone 1	Z1-A; Z1-B
Zone 2	Z2-A1; Z2-A11; Z2-A12; Z2-A2; Z2-B
Zone 3	Z3-B; Z3-A2; Z3-A12; Z3-A112; Z3-A11; Z3-A111
Zone 4	Z4-1; Z4-2; Z4-3; Z4-4
Zone 5	Z5-2; Z5-3; Z5-4
Zone 6	Z6-1; Z6-2; Z6-3; Z6-4

Table 9-4: Dry Waste Collection Routes for all the zones of Ammochhu LAP.

Area	Route
Zone 1 and 2	Z1-A; Z2-B; Z2-A2; Z2-A1; Z2-A12; Z2-A11
Zone 3	Z3-AA111; Z3-AA112; Z3-AA12; Z3-A12; Z3-A11; Z3-AA2; Z3-A2; Z3-B; Z3-BB
Zone 4	Z4-1; Z4-2; Z4-3; Z4-4
Zone 5 and 6	(Z5-2; Z5-1; Z5-3; Z5-4) and (Z6-1; Z6-2; Z6-3; Z6-4)

In order to understand the amount (weight in tonnes) of wet/dry waste collection system, carrying capacity of compactor truck is taken into consideration. In general, the total of 8 tonnes of waste weight is the optimum amount of (weight and volume) waste materials that can be withstood and collected at one time by compactor trucks. Since after the waste generated for each zone is calculated, the number of compactor trucks required for each trip can be determined. Table 5 shows the amount of waste (dry and wet) generated in a week during each trip. For the zones (1 and 2), the total dry waste generated is approximately 3.8 tonnes while for zone 3, dry waste generated is 6.97 tonnes therefore, it requires one compactor truck/ one trip in the morning hours to collect waste from zone 3 and one compactor truck/one trip in the afternoon hours to collect waste from zone 1 and 2. For Zone 4, approximately 8.1 tonnes of dry waste are generated in a week and similarly for zone 5 and 6, total dry waste generated is calculated at 5.12 tonnes. Therefore, one compactor truck/one trip will suffice to collect the waste from zone 4 in the morning hours and subsequently one compactor truck/one trip is enough to collect the waste from zone 5 and 6 in the afternoon hours. In case of collecting wet waste, one compactor truck is enough to collect waste of 5.35 tonnes from zone 1 to 3 during the afternoon hours and similarly it requires one compactor truck/one trip to collect wet waste of total 6.6 tonnes from zone 4 to 6, during the morning hours.

Figure 9-5 represents the dry waste collection routes from zone 1 to 3 of Ammochhu Lap which is collected in a week. Routes for waste collection are marked in the map which shows the possible waste collection routes from zone 3 in the morning hours and zone 1 and 2 in the afternoon hours. For collecting the waste, one compactor truck is required to collect waste from zone 3 from 8 am to 12:30 pm while for collecting waste from zone 1 and 2, one compactor truck of one trip is required from 1:30 pm to 5 pm. For collecting the wet waste, one compactor truck per trip is sufficient to collect 5.35 tonnes of waste during the afternoon hours from zone 1 to 3. Figure 9-6 shows the possible wet waste collection routes from zone 1 to 3 from 1.30 pm to 5 pm. To collect the wet waste of 5.35 tonnes, one compactor truck is sufficient to collect the waste from zone 1 to 3. In zone 2, there lies a patch of service land where it has been allocated in Figure 9-9 for the construction of drop-off Centre. Its location is proposed based on the accessibility of vehicular roads so that it can be accessed by anyone easily at any time.

Table 9-5: Quantification of Dry and Wet waste based on each trip.

Population	Wet waste/week (Kg)	Dry waste/week (kg)	Total waste/week (kg)	Wet waste per trip (tonnes)	Remarks
4,224	3,696	3,696	7,392	1.85	Zone 1 and 2
7,968	6,972	6,972	13,944	3.5	Zone 3
9,216	8,064	8,064	16,128	4.03	Zone 4
5,856	5,124	5,124	10,248	2.6	Zone 5 and 6

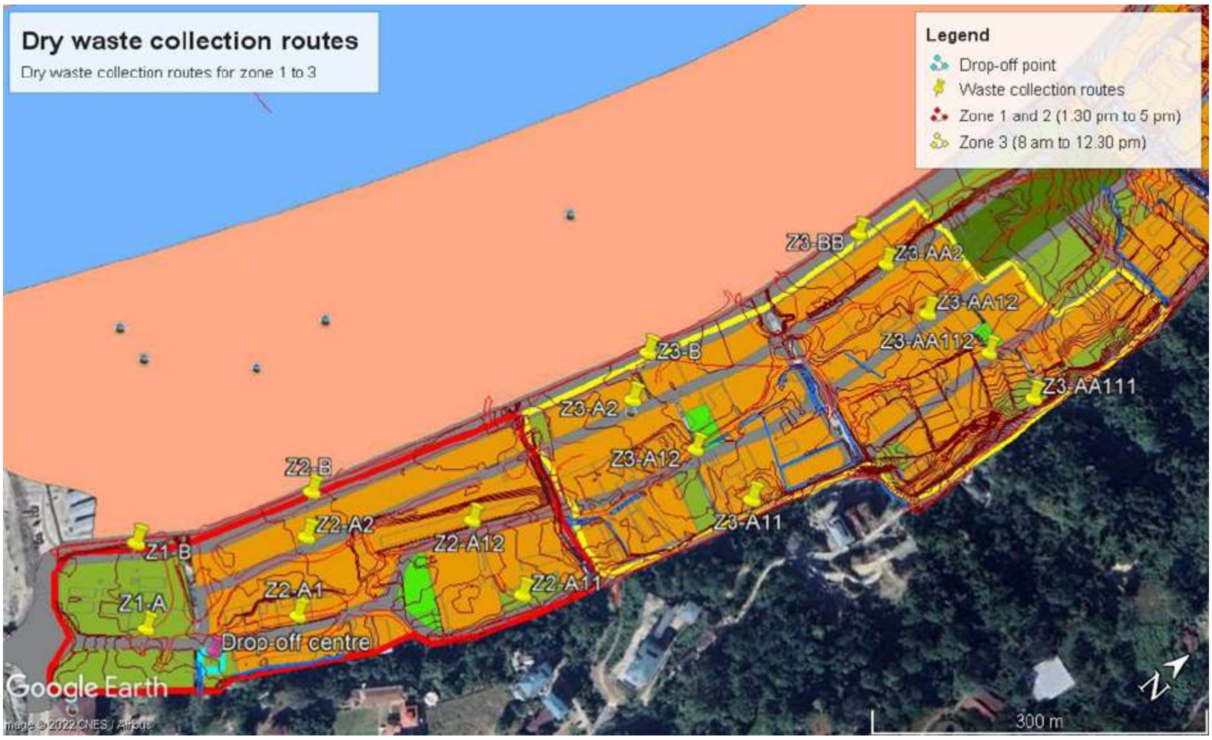

Figure 9-5: Ammochhu LAP (zone 1 to 3) dry waste collection for morning and afternoon hours.

For the truck, to collect waste from each point (location) approximately takes 10 minutes are required. Therefore, in total there should be 20 waste collection points for dry waste and in total accounting to 3.3 hours from 8 AM to 11.30 AM. This waste then needs to be transported to the landfill in Toribari which is 30 minutes' drive- one way. By truck reaches back it will be 12.30 PM. There should be a break of one hour.

In the afternoon, again the same truck will go for the other zones to collect dry waste from 1.30 PM to 4.30 PM for collection and 30 minutes for transportation up to 5.0 PM. Therefore, during the afternoon hours waste collection trip will have 16 collection points which accounts to around 3 hours.

Figure 9-7 shows the zones from 4 to 6 of Ammochhu Lap where dry wastes generated in a week are collected. Routes for dry waste collection are marked in the map which shows the possible waste collection routes from zone 4 during morning hours (8 am to 12.30 pm) and subsequently routes for waste collection from zone 5 and 6 during the afternoon hours (1.30 pm to 5 pm) are also shown in the same figure. For collecting the waste of 8.1 tonnes from zone 4, one compactor truck is required during the morning hours. Similarly, for collecting the waste of 5.12 tonnes from zone 5 and 6 together, one compactor truck is sufficient.

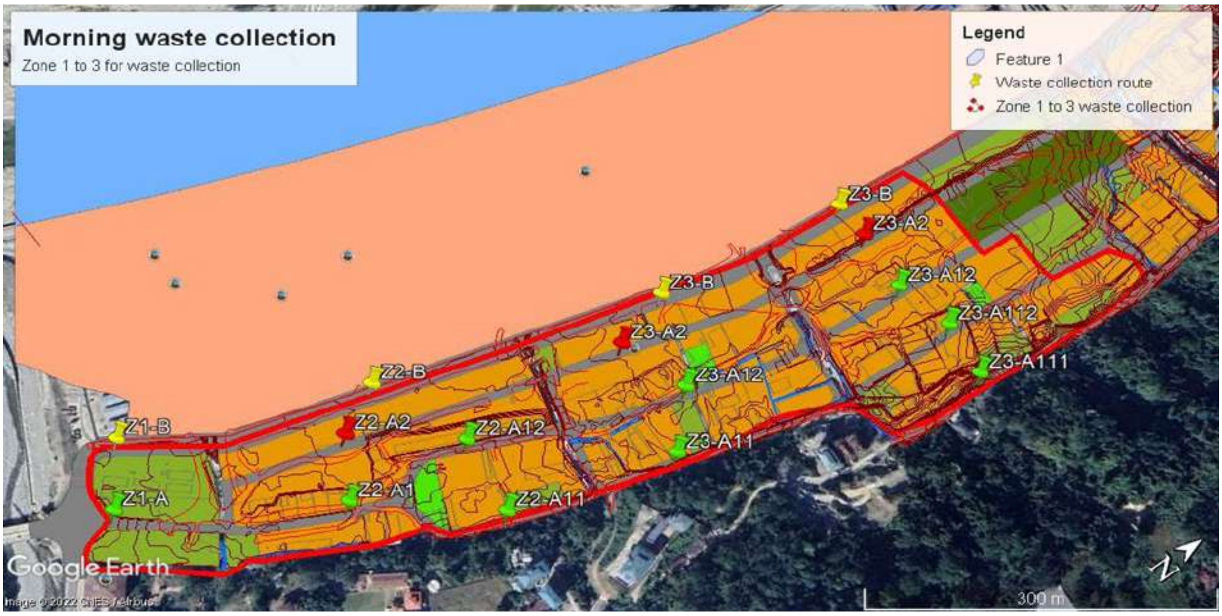

Figure 9-6: Ammochhu LAP (Zone 1 to 3) Wet waste collection for afternoon hours.

Figure 9-8 shows the possible wet waste collection routes from zone 4 to 6 from 8 am to 12.30 pm. To collect the wet waste of 6.6 tonnes, one compactor truck is sufficient to collect the waste from zone 4 to 6. In zone 6, there lies a patch of service land where it has been allocated Figure 9-10 for the construction of drop-off centre. Its location is proposed based on the accessibility of vehicular roads so that it can be accessed by anyone easily at any time.

For truck to collect waste from each point (location) approximately takes 10 minutes. Therefore, in total there should be 20 waste collection points during the morning hours waste collection trip which accounts to 4 hours from 8 am to 12.30 pm and with 16 collection points during the afternoon waste collection trip accounting to around 3 hours from 1.30 pm to 5 pm with half an hour drive for disposing the waste at the landfill.

To collect the waste segregated at the drop-off centres, the same truck can be deployed to collect the waste after the waste collection from the zones during each trip is completed. From each waste collection trip certain amount of capacity of the truck remain vacant which can be compensated with the waste from the drop-off centres. No communal bins are advised to be provided at places in Ammochhu Lap since all the places will be developed with clustered infrastructures and settlements which further makes the surroundings unhygienic and dirty if provision is made for communal bins.

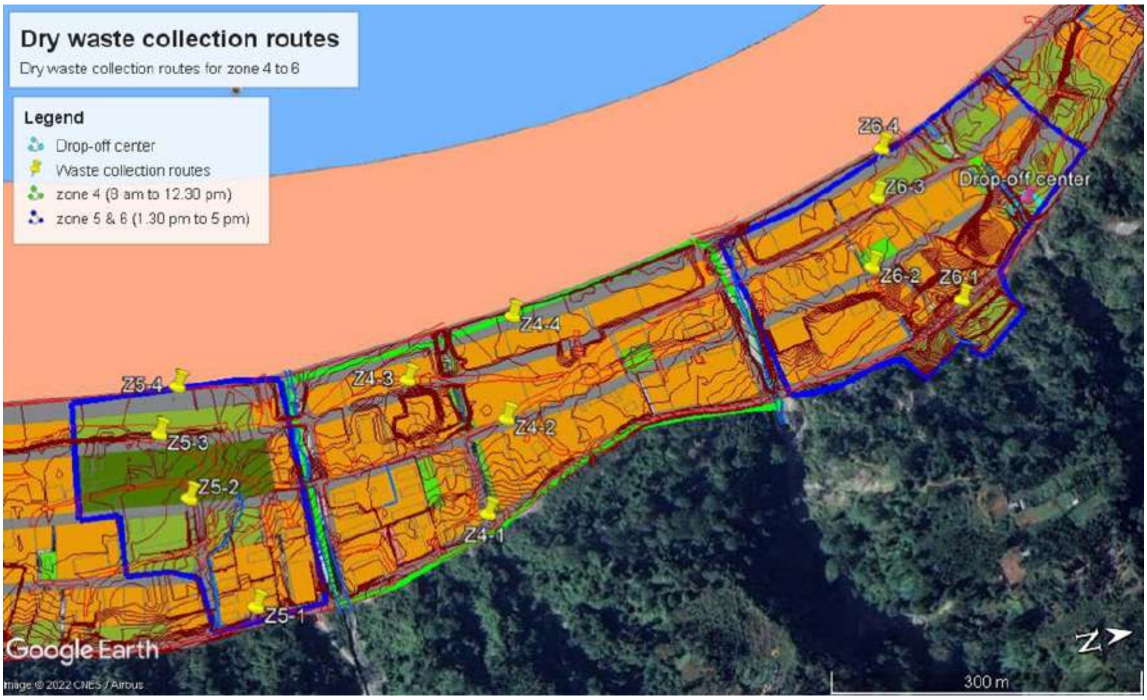

Figure 9-7: Ammochhu LAP (Zone 4 to 6) dry waste collection for morning and afternoon hours.

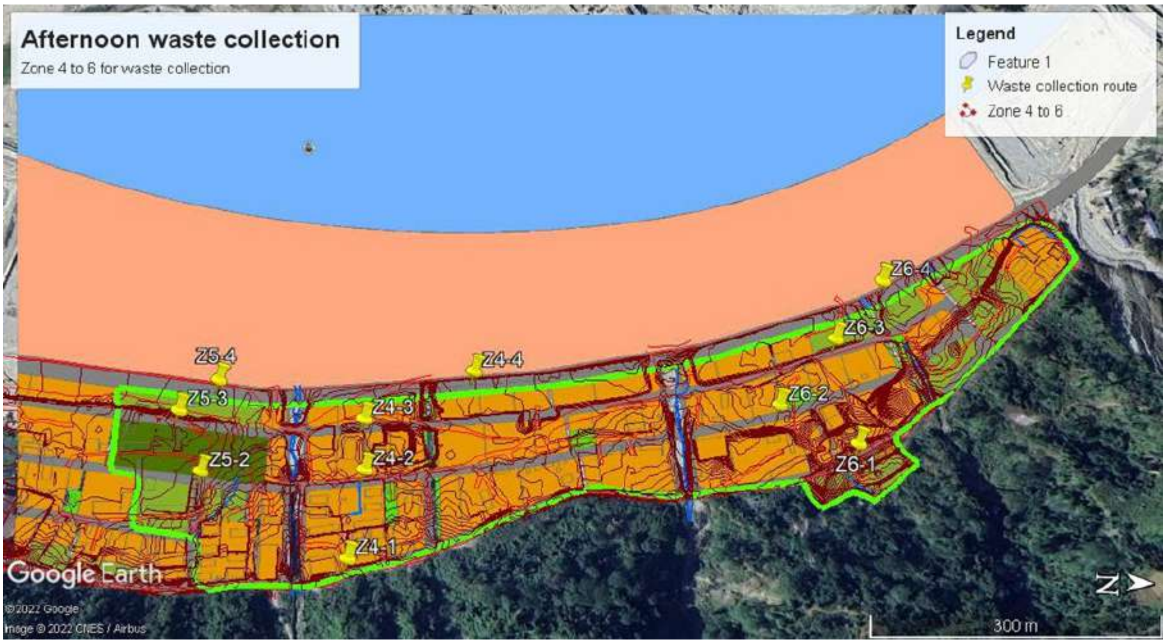

Figure 9-8: Ammochhu LAP (Zone 4 to 6) wet waste collection for morning hours.

Figure 9-9: Proposed Drop-off Centre located at zone 2.

Figure 9-10: Proposed Drop-off Centre located at Zone 6.

10 Project Cost

10.1 Infrastructure project cost

The total cost of implementing the LAP filling, Stormwater Drainage, Water Supply System, Sewer System, Road work, Service Utility Duct, Street Elements, Street Lighting, CCTV, Footpath, Embankment wall & Culvert crossings is Nu. 858,429,471.04. The Summary of breakdown of cost is given in the Table 10-1.

The BSR 2022 rates and market rates for those items not available in BSR 2022 was used to determine the total cost of each of these projects.

Table 10-1: Summary of Cost.

Item No	Items	Cost (Nu.)
1.0	LAP Filling	70,731,374.44
2.0	Stormwater Drainage	102,271,375.62
3.0	Water Supply System	27,856,943.93
4.0	Sewer System	43,364,675.29
5.0	Service Utility Duct	129,793,009.53
6.0	Road Works	181,099,207.25
7.0	Street Lighting	24,657,972.46
	CCTV	28,859,445.00
8.0	Street Elements	
	Bus Stops	978,087.73
9.0	Footpath	32,990,578.28
10.0	Embankment Wall	61,801,761.72
11.0	Culverts	155,342,694.50
	Total Project Cost	**858,429,471.04**

10.2 Action plan's project Cost

The projects outline in Action plans are Pedestrian Under-pass, Pedestrian Footbridge, Public Parks/Children's playground, Sport Complex (OS-1), Multi-Level Car parking integrated with iconic built form & Drop off Centers. The total approximate/rough cost of implementing these projects is Nu. 389,157,672.50.

While working out the rough estimate of these projects, similar past projects were referred and build-up area rate in term of square meter was calculated. The build-up area rate times the build-up area of project was used to determine the rough project cost for each of these projects.

The summary of breakdown cost of these projects is given in Table 10-2.

Table 10-2: Summary of Cost.

Action plan's project	Cost (Nu.)
Pedestrian Underpass	32,500,000.00
Pedestrian footbridge	29,591,100.00
Multi-Level Car parking with iconic Built form	268,800,000.00
Sport Complex (OS-1)	35,000,000.00
Public Parks/ Children's Play ground	21,266,572.50
Drop off Centers	2,000,000.00
Total Project Cost	**389,157,672.50**

10.3 Total project cost

The total cost of implementing immediately needed infrastructure projects and the projects outline in Actions plan is Nu.1,247,587,143.54.

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12 Annexure A-D type Drain's discharges, velocity and sizes

Table 12-1: Major Drain D type details.

Drain ID	Length(m)	Design Discharge(Q), m^3/s	Velocity (V) achieved, m/s	Cross Section Area of drain, m^2	Free Board(m)	Depth, H (m)	Width, W (m)	Invert Level at Start, m	Invert Level at End, m
D1	124.67	0.277	2.33	0.12	0.15	0.45	0.60	205.31	203.11
D2	44.01	0.066	1.59	0.04	0.10	0.30	0.40	207.69	206.95
D3	117.31	0.029	1.51	0.02	0.10	0.30	0.40	212.88	206.70
D4	96.93	0.091	1.88	0.05	0.15	0.40	0.50	212.78	209.40
D5	88.85	0.090	1.55	0.06	0.15	0.40	0.50	203.60	202.49
D6	85.52	0.085	1.52	0.06	0.15	0.40	0.50	203.70	202.63
D7	90.00	0.102	2.01	0.05	0.15	0.40	0.50	207.25	204.39
D8	94.15	0.061	1.52	0.04	0.10	0.30	0.40	211.24	209.21
D9	135.79	0.238	1.56	0.15	0.15	0.45	0.60	202.62	201.72
D10	115.00	0.116	1.58	0.07	0.10	0.30	0.40	203.90	202.62
D11	142.29	1.211	2.19	0.55	0.30	0.85	1.10	201.98	201.19
D12	115.00	0.164	1.65	0.10	0.15	0.40	0.50	203.80	202.65
D13	150.00	0.266	1.80	0.15	0.15	0.45	0.60	204.01	202.65
D14	46.27	0.090	2.28	0.04	0.10	0.30	0.40	205.89	203.08
D15	113.89	0.272	2.01	0.13	0.15	0.45	0.60	204.03	202.65
D16	55.00	0.037	2.32	0.02	0.10	0.30	0.40	208.40	204.10
D17	54.50	0.583	1.82	0.32	0.15	0.55	0.80	202.48	201.98
D18	122.82	0.128	1.69	0.08	0.15	0.40	0.50	212.33	210.79
D19	183.79	0.165	2.06	0.08	0.15	0.40	0.50	212.33	209.10
D20	135.00	0.131	1.63	0.08	0.10	0.30	0.40	203.70	202.20
D21	145.90	0.260	1.86	0.14	0.15	0.45	0.60	202.20	200.74
D22	135.00	0.155	1.65	0.09	0.15	0.40	0.50	203.60	202.20

D23	146.48	0.380	2.08	0.18	0.15	0.45	0.60	202.20	200.66
D24	135.00	0.181	1.87	0.10	0.15	0.40	0.50	206.29	203.09
D25	176.04	0.527	2.21	0.24	0.15	0.50	0.70	202.33	200.57
D26	107.55	0.086	2.13	0.04	0.15	0.40	0.50	208.60	203.67
D27	179.71	0.115	1.51	0.08	0.15	0.40	0.50	206.10	203.16
D28	46.87	0.250	1.66	0.15	0.15	0.45	0.60	202.69	202.33
D29	144.00	0.139	1.53	0.09	0.15	0.40	0.50	201.60	200.29
D30	144.25	0.248	1.66	0.15	0.15	0.45	0.60	201.55	200.44
D31	60.00	0.104	3.50	0.03	0.10	0.30	0.40	207.85	201.87
	90.00	0.238	1.91	0.12	0.15	0.40	0.50	201.87	200.84
D32	101.33	0.129	1.72	0.07	0.10	0.30	0.40	204.50	201.10
D33	57.53	0.415	1.95	0.21	0.15	0.50	0.70	200.80	200.32
D34	155.44	0.084	1.97	0.04	0.15	0.40	0.50	212.24	206.25
D35	215.00	0.196	1.67	0.12	0.15	0.40	0.50	201.60	199.65
D36	187.09	0.367	1.95	0.19	0.15	0.50	0.70	199.65	197.94
D37	215.00	0.336	1.85	0.18	0.15	0.45	0.60	201.55	199.76
D38	225.13	0.719	2.31	0.31	0.15	0.55	0.80	199.76	197.71
D39	150.00	0.201	1.95	0.10	0.15	0.40	0.50	207.98	200.69
D40	125.00	0.610	3.56	0.17	0.15	0.50	0.70	206.00	201.00
D41	150.00	1.061	2.26	0.47	0.30	0.80	1.00	200.36	199.36
D42	174.48	1.332	2.79	0.48	0.30	0.85	1.10	199.29	197.55
D43	135.00	0.239	1.60	0.15	0.15	0.45	0.60	203.90	202.94
D44	187.12	0.883	2.16	0.41	0.15	0.60	0.90	202.94	201.69
D45	102.48	0.108	2.66	0.04	0.10	0.30	0.40	211.98	207.19
D46	150.51	0.275	1.82	0.15	0.15	0.45	0.60	208.52	206.79
D47	64.51	0.370	2.03	0.18	0.15	0.45	0.60	207.50	202.96
D48	135.00	0.128	1.55	0.08	0.15	0.40	0.50	198.10	196.75
D49	118.51	0.230	1.63	0.14	0.15	0.45	0.60	196.75	195.84

D50	120.00	0.193	1.52	0.13	0.15	0.40	0.50	198.10	197.24
D51	142.44	0.420	1.73	0.24	0.15	0.50	0.70	197.24	196.38
D52	135.00	0.233	1.45	0.16	0.15	0.45	0.60	198.05	197.30
D53	132.63	0.451	1.51	0.30	0.15	0.55	0.80	197.30	196.77
D54	160.00	0.147	1.50	0.10	0.15	0.40	0.50	196.30	194.97
D55	163.32	0.300	1.51	0.20	0.15	0.50	0.70	194.97	194.11
D56	160.00	0.196	1.55	0.13	0.15	0.40	0.50	196.30	195.11
D57	164.85	0.475	1.73	0.27	0.15	0.55	0.80	195.11	194.20
D58	186.65	0.304	1.50	0.20	0.15	0.50	0.70	196.65	195.69
D59	202.01	0.401	1.86	0.22	0.15	0.50	0.70	197.92	196.39
D60	172.11	0.911	1.80	0.51	0.30	0.85	1.10	195.69	195.00
D61	87.36	0.204	2.05	0.10	0.15	0.40	0.50	197.13	195.50
D62	111.98	0.136	1.77	0.08	0.10	0.30	0.40	198.47	195.40
*	*	*	*	*	*	*	*	*	*

13 Annexure B- RD type Drain's discharge, Velocity & sizes

Table 13-1: Drain RD type Details.

Drain ID	Length(m)	Design Discharge(Q), m³/s	Velocity (V) achieved, m/s	Cross Section Area of drain, m²	Depth, H (m)	Width, W (m)	Invert Level at Start, m	Invert Level at End, m
RD1	118.60	0.022	1.08	0.02	0.28	0.35	212.91	206.00
RD2	95.26	0.019	1.21	0.02	0.28	0.35	212.98	209.48
RD3	79.22	0.023	1.40	0.02	0.28	0.35	206.36	204.47
RD4	97.14	0.020	0.81	0.02	0.28	0.35	211.24	209.37
RD5	148.29	0.029	0.90	0.03	0.28	0.35	204.18	203.258
RD6	113.65	0.036	1.06	0.03	0.28	0.35	204.23	203.29
RD7	123.42	0.020	0.98	0.02	0.28	0.35	212.45	211.22
RD8	182.97	0.029	0.99	0.03	0.28	0.35	212.45	208.99
RD9	110.00	0.034	1.12	0.03	0.28	0.35	204.25	203.17
RD10	166.17	0.062	1.28	0.05	0.28	0.35	203.17	201.57
RD11	108.28	0.019	1.34	0.01	0.28	0.35	208.73	203.91
RD12	180.32	0.029	1.15	0.03	0.28	0.35	206.23	203.63
RD13	141.93	0.025	0.84	0.03	0.28	0.35	202.02	201.20
RD14	157.74	0.027	1.49	0.02	0.28	0.35	212.37	206.34
RD15	180.00	0.031	1.04	0.03	0.28	0.35	201.68	200.11
RD16	220.00	0.067	1.23	0.05	0.28	0.35	200.11	198.28
RD17	155.00	0.021	0.66	0.03	0.28	0.35	204.05	203.53
RD18	155.50	0.049	0.81	0.06	0.28	0.35	203.53	203.01
RD19	105.13	0.024	1.82	0.01	0.28	0.35	211.91	206.41
RD20	152.15	0.023	0.91	0.03	0.28	0.35	208.69	206.19
RD21	96.12	0.020	1.27	0.02	0.28	0.35	212.17	208.68
RD22	117.19	0.016	1.30	0.01	0.28	0.35	212.15	203.35

RD23	120.00	0.026	0.83	0.03	0.28	0.35	198.27	197.61
RD24	134.00	0.051	0.95	0.05	0.28	0.35	197.61	196.94
RD25	125.00	0.023	1.21	0.02	0.28	0.35	202.88	199.93
RD26	119.51	0.044	1.24	0.04	0.28	0.35	199.93	198.31
RD27	180.00	0.031	0.66	0.05	0.28	0.35	196.87	196.40
RD28	173.97	0.056	0.98	0.06	0.28	0.35	196.40	195.53
RD29	195.12	0.034	1.01	0.03	0.28	0.35	198.14	196.67
RD30	110.83	0.018	1.34	0.01	0.28	0.35	198.50	195.72
*	*	*	*	*	*	*	*	*

14 Annexure C- Required NP3 Drainage Hume pipe sizes at road crossings

Table 14-1: NP3 Hume Pipe Details.

Sl. No	Drain ID	Length(m)	Slope	Design Discharge(Q), m^3/s	Velocity (V) achieved, m/s	Diameter, D (m)
1.0	D13	10.00	0.91%	0.266	1.45	0.45
2.0	D17	6.60	0.83%	0.583	1.71	0.60
3.0	D28	6.62	0.76%	0.250	1.34	0.45
4.0	D33	8.31	0.83%	0.415	1.57	0.60
5.0	D36	7.79	0.91%	0.367	1.57	0.60
6.0	D38	6.77	0.91%	0.719	1.86	0.75
7.0	D40	14.50	5.00%	0.610	3.39	0.45
8.0	D41	17.80	0.67%	1.061	1.83	0.90
9.0	D42	6.69	1.00%	1.332	2.25	0.90
10.0	D44	14.20	0.67%	0.883	1.75	0.75
11.0	D47	6.95	0.98%	0.370	1.62	0.45
12.0	D51	8.73	0.67%	0.420	1.45	0.60
13.0	D53	8.24	0.50%	0.451	1.33	0.60
14.0	D57	14.00	0.56%	0.475	1.40	0.60
15.0	D58	29.60	0.51%	0.304	1.21	0.60
16.0	RD2	7.70	0.61%	0.019	0.64	0.30
17.0	RD3	6.64	0.61%	0.023	0.67	0.30
18.0	RD4	8.13	0.61%	0.020	0.65	0.30
19.0	RD7	6.60	0.61%	0.020	0.66	0.30
20.0	RD8	9.44	1.82%	0.029	1.08	0.30
21.0	RD10	6.76	0.61%	0.062	0.87	0.30
22.0	RD13	10.07	0.61%	0.025	0.69	0.30
23.0	RD14	6.65	0.61%	0.027	0.71	0.30
24.0	RD16	7.45	0.61%	0.067	0.88	0.30
25.0	RD18	7.00	0.61%	0.049	0.82	0.30
26.0	RD20	7.00	6.88%	0.023	1.69	0.30
27.0	RD22	8.50	10.00%	0.016	1.76	0.30
28.0	RD24	8.02	0.61%	0.051	0.83	0.30
29.0	RD26	6.60	0.61%	0.044	0.79	0.30
30.0	RD28	6.75	0.61%	0.056	0.85	0.30
31.0	RD30	6.60	0.61%	0.018	0.63	0.30
32.0	ND5	11.35	10.00%	2.291	6.12	0.90
*	*	*	*	*	*	*

15 Annexure D- Water Supply Hydraulic Modelling Results

Table 15-1: Node Details-Peak Demand Hour and Off-peak Demand Hour[16]

Node Physical Data				Peak Demand Hour			Off Peak Demand Hour		
Node Label	X (m)	Y (m)	Elevation (m)	Demand (L/min)	Hydraulic Grade (m)	Pressure Head (m)	Demand (L/min)	Hydraulic Grade (m)	Pressure Head (m)
J-1	188324.4	2974821	214.31	0	221.88	7.57	0	222.22	7.91
J-4	188192.1	2974972	209.39	0	219.93	10.55	0	222.15	12.77
J-5	188182.6	2974982	210.9	0	219.87	8.97	0	222.15	11.26
J-6	188161.2	2975003	211.16	0	219.79	8.62	0	222.15	10.99
J-8	188117.8	2975045	213.84	0	219.71	5.88	0	222.15	8.31
J-11	188163.8	2974944	208.25	0	219.7	11.46	0	222.15	13.9
J-12	188147.4	2974910	203.24	28	219.05	15.8	3	222.12	18.88
J-13	188158.1	2974892	203.15	28	218.8	15.65	3	222.11	18.96
J-14	188169.5	2974873	203.32	28	218.57	15.25	3	222.1	18.78
J-15	188180.7	2974853	202.54	21	218.35	15.82	2	222.09	19.55
J-16	188190.5	2974833	202.42	28	218.18	15.76	3	222.08	19.66
J-17	188199.8	2974806	202.72	28	217.97	15.25	3	222.07	19.35
J-18	188209.5	2974776	202.75	42	217.78	15.04	4	222.06	19.32
J-19	188216.6	2974753	203.06	28	217.68	14.61	3	222.05	18.99
J-20	188231.1	2974711	202.94	42	217.51	14.57	4	222.04	19.1
J-21	188238.8	2974687	202.75	28	217.32	14.57	3	222.03	19.28
J-22	188250.9	2974651	202.16	56	217.04	14.87	6	222.02	19.86
J-23	188259.7	2974610	202.33	28	216.76	14.43	3	222.01	19.68
J-24	188175	2974936	207.75	14	219.58	11.83	1	222.14	14.39
J-25	188189.1	2974919	207.41	28	219.4	11.99	3	222.13	14.72
J-26	188207.8	2974896	206.83	28	219.19	12.36	3	222.12	15.3
J-27	188224.8	2974874	206.18	14	219.02	12.84	1	222.11	15.93
J-28	188233.9	2974856	203.03	28	218.91	15.88	3	222.11	19.08
J-29	188244.6	2974828	202.67	28	218.78	16.1	3	222.1	19.43
J-30	188249.5	2974813	202.54	14	218.72	16.18	1	222.1	19.56
J-31	188259.6	2974785	202.35	28	218.62	16.28	3	222.09	19.74
J-32	188265.4	2974772	202.34	28	218.59	16.25	3	222.09	19.75
J-33	188270.9	2974758	201.92	7	218.56	16.65	1	222.09	20.17
J-34	188281.6	2974729	202.26	14	217.8	15.54	1	222.05	19.79
J-35	188286.5	2974713	201.8	28	217.65	15.85	3	222.05	20.25
J-36	188294.1	2974685	202	28	217.41	15.41	3	222.04	20.04
J-37	188300.8	2974655	203.22	28	217.17	13.95	3	222.02	18.8
J-38	188307.1	2974626	203.66	28	216.95	13.28	3	222.01	18.35
J-39	188308.6	2974773	207.23	49	219.44	12.21	12	222.12	14.89

[16] Node with 0 demand represents pipe junction with no consumer connection. X and Y value represents the actual coordinates of the node location.

J-40	188332.6	2974808	214.5	0	221.03	6.53	0	222.18	7.68
J-41	188268.9	2974564	202.17	28	216.47	14.31	3	221.99	19.83
J-42	188273.4	2974542	202.11	28	216.34	14.23	3	221.98	19.88
J-43	188279.4	2974512	201.66	28	216.18	14.52	3	221.98	20.32
J-44	188284.9	2974483	201.47	28	216.02	14.56	3	221.97	20.5
J-45	188289.3	2974460	201	42	215.91	14.91	4	221.96	20.96
J-46	188295.2	2974428	203.39	56	215.75	12.37	6	221.95	18.57
J-47	188300.1	2974385	201.17	42	215.57	14.4	4	221.94	20.77
J-48	188304.3	2974346	200.75	28	215.42	14.67	3	221.93	21.18
J-49	188308	2974313	200.12	28	215.31	15.19	3	221.92	21.8
J-50	188312.3	2974273	200.73	28	215.18	14.46	3	221.91	21.18
J-51	188315.6	2974240	201.3	28	215.08	13.79	3	221.9	20.61
J-52	188318.8	2974206	200.23	42	214.99	14.77	4	221.9	21.67
J-53	188323.9	2974152	200.8	28	214.87	14.07	3	221.89	21.09
J-54	188326	2974117	200.38	42	214.66	14.29	4	221.87	21.49
J-55	188326.2	2974099	201.29	14	214.58	13.3	1	221.86	20.57
J-56	188324.2	2974065	200.31	21	214.45	14.13	2	221.84	21.53
J-57	188321	2974032	198.51	28	214.33	15.82	3	221.83	23.32
J-58	188318	2973999	198.25	28	214.25	16	3	221.81	23.56
J-59	188314.4	2973962	197.78	14	214.18	16.4	1	221.8	24.02
J-60	188310.4	2973923	197.5	28	214.11	16.61	3	221.79	24.29
J-61	188306.4	2973884	198.5	42	214.06	15.56	4	221.77	23.27
J-62	188302.2	2973844	198.5	42	214.03	15.53	4	221.75	23.25
J-63	188298.5	2973808	198.5	42	214.03	15.53	4	221.74	23.24
J-64	188294.3	2973771	198.42	28	214.03	15.61	3	221.73	23.32
J-65	188290.4	2973738	198.44	28	214.03	15.59	3	221.72	23.28
J-66	188317.8	2974572	204.5	28	216.55	12.05	3	221.99	17.5
J-67	188324.2	2974543	204.86	28	216.35	11.49	3	221.98	17.12
J-68	188327.6	2974528	204.63	28	216.26	11.63	3	221.98	17.35
J-69	188333.8	2974498	204.02	34	216.07	12.06	4	221.97	17.95
J-70	188340.3	2974470	203.08	28	215.91	12.83	3	221.96	18.88
J-71	188347	2974440	202.6	28	215.77	13.17	3	221.95	19.35
J-72	188352.8	2974411	201.94	28	215.64	13.7	3	221.94	20
J-73	188361	2974353	200.5	28	215.41	14.91	3	221.93	21.43
J-74	188365.1	2974323	200.5	28	215.31	14.81	3	221.92	21.42
J-75	188370.9	2974285	199.63	28	215.19	15.56	3	221.91	22.28
J-76	188375.8	2974256	200.62	14	215.1	14.48	1	221.91	21.29
J-77	188380.1	2974226	202.24	28	215.02	12.78	3	221.9	19.66
J-78	188385.7	2974157	203.5	28	214.84	11.35	3	221.88	18.39
J-79	188391.5	2974113	204.3	42	214.68	10.38	4	221.87	17.57
J-80	188391.4	2974083	202.82	28	214.61	11.79	3	221.86	19.05
J-81	188388.9	2974053	201.51	14	214.54	13.03	1	221.85	20.35
J-82	188386.4	2974024	199.43	7	214.47	15.04	1	221.85	22.41
J-83	188383.2	2973993	198.95	14	214.35	15.4	1	221.83	22.88
J-84	188380.2	2973963	198.73	21	214.24	15.51	2	221.81	23.08
J-85	188377.3	2973933	197.84	21	214.15	16.31	2	221.79	23.95

J-86	188371.4	2973874	197.88	42	214.06	16.18	4	221.77	23.88
J-87	188367.9	2973843	197.5	28	214.05	16.55	3	221.76	24.26
J-88	188363.3	2973799	197	42	214.04	17.04	4	221.74	24.74
J-89	188352.4	2973726	197.51	28	214.04	16.53	3	221.72	24.21
J-94	188414.1	2974023	199.5	14	214.35	14.85	1	221.83	22.33
J-95	188444.1	2974024	203.03	28	214.24	11.21	3	221.82	18.79
J-96	188450.6	2973996	201.5	42	214.12	12.62	4	221.81	20.31
J-97	188449	2973981	199.91	28	214.08	14.17	3	221.81	21.89
J-98	188445	2973952	199.87	28	214.02	14.15	3	221.8	21.93
J-99	188442.9	2973936	199.46	28	214	14.54	3	221.79	22.33
J-100	188440.6	2973922	198.66	28	213.98	15.32	3	221.79	23.12
J-101	188438.4	2973906	198.44	28	213.98	15.53	3	221.78	23.34
J-102	188434.1	2973877	198.96	21	213.97	15.01	2	221.78	22.82
J-103	188431.5	2973848	199.46	28	213.97	14.51	3	221.77	22.31
J-104	188429.2	2973817	200.96	28	213.97	13.01	3	221.77	20.8
J-105	188428.1	2973802	199.94	28	213.97	14.03	3	221.76	21.82
J-106	188425.9	2973772	199.98	28	213.98	13.99	3	221.76	21.78
J-107	188423.5	2973742	199.97	28	213.99	14.03	3	221.76	21.79
J-108	188435.1	2973707	197.67	0	214.04	16.37	0	221.75	24.08
J-109	188356.6	2974381	201.65	28	215.52	13.88	3	221.94	20.29
J-112	188375.7	2973919	197.75	42	214.11	16.36	4	221.79	24.04
J-113	188378.5	2973948	198.06	14	214.19	16.13	1	221.8	23.74
J-115	188109.4	2974962	204.53	28	219.77	15.24	3	222.15	17.62
J-116	188127.2	2974942	203.14	14	219.86	16.71	1	222.15	19.01
J-119	188095.6	2975026	211.74	125	219.68	7.93	13	222.15	10.41
J-120	188134.1	2974982	210.06	42	219.77	9.71	4	222.15	12.09
J-121	188153.8	2974961	207.75	42	219.86	12.11	4	222.15	14.41
J-122	188287.7	2973715	197.68	28	214.03	16.35	3	221.72	24.04
J-123	188283.1	2973685	197.82	42	214.03	16.21	4	221.71	23.89
J-124	188274.1	2973656	197.96	28	214.04	16.08	3	221.7	23.74
J-125	188267.4	2973635	197.86	42	214.06	16.2	4	221.7	23.84
J-126	188259.9	2973612	197.6	14	214.1	16.5	1	221.7	24.1
J-127	188254.2	2973595	197.47	28	214.1	16.63	3	221.69	24.22
J-128	188245.7	2973569	197.25	42	214.12	16.87	4	221.69	24.44
J-129	188237.5	2973543	197.03	28	214.15	17.13	3	221.68	24.66
J-130	188228	2973513	196.75	42	214.21	17.47	4	221.68	24.93
J-131	188219.3	2973486	196.5	28	214.29	17.79	3	221.68	25.18
J-132	188349	2973707	196.57	28	214.05	17.48	3	221.72	25.15
J-133	188348.3	2973679	196.59	42	214.07	17.48	4	221.72	25.13
J-134	188336.5	2973635	196.57	28	214.13	17.56	3	221.71	25.14
J-135	188331.7	2973621	196.35	21	214.13	17.78	2	221.7	25.35
J-136	188322.7	2973593	196.08	28	214.15	18.06	3	221.7	25.61
J-137	188318.1	2973579	196	14	214.18	18.18	1	221.69	25.69
J-138	188308.7	2973550	196.5	28	214.27	17.77	3	221.69	25.19
J-139	188303.9	2973535	196.43	28	214.33	17.91	3	221.69	25.26
J-140	188294.9	2973507	196.97	28	214.47	17.5	3	221.68	24.71

J-141	188281.7	2973480	196.89	28	214.64	17.75	3	221.68	24.79
J-142	188269	2973471	197.51	0	214.75	17.25	0	221.68	24.17
J-144	188409.9	2973662	197.46	28	214.11	16.65	3	221.73	24.27
J-145	188398.4	2973618	200.34	14	214.2	13.86	1	221.71	21.37
J-146	188395.1	2973604	200.08	14	214.29	14.22	1	221.71	21.63
J-147	188386.3	2973575	199.88	14	214.49	14.62	1	221.7	21.83
J-148	188377.5	2973546	199.61	14	214.71	15.1	1	221.7	22.09
J-149	188371.9	2973532	198.02	14	214.82	16.8	1	221.7	23.68
J-150	188362.5	2973504	199.59	14	215.07	15.49	1	221.69	22.11
J-151	188354.6	2973475	199.81	14	215.34	15.53	1	221.69	21.88
J-152	188339.5	2973446	199.32	14	215.68	16.36	1	221.68	22.36
J-153	188305.2	2973454	197.34	28	215.22	17.88	3	221.68	24.34
J-154	188202.7	2973436	196.25	42	214.22	17.96	4	221.67	25.42
J-155	188188.6	2973392	196.49	28	214.18	17.69	3	221.66	25.17
J-156	188174.3	2973348	196.04	42	214.16	18.13	4	221.66	25.62
J-157	188157.8	2973297	195.91	28	214.16	18.25	3	221.66	25.75
J-158	188143.4	2973252	195.49	56	214.16	18.67	6	221.65	26.16
J-159	188127.9	2973204	195.19	28	214.17	18.98	3	221.65	26.46
J-160	188109.7	2973161	194.85	70	214.2	19.36	7	221.65	26.8
J-161	188132.2	2973150	195	14	214.25	19.25	1	221.65	26.65
J-162	188145	2973143	195.56	0	214.28	18.72	0	221.65	26.09
J-163	188134.1	2973117	195.83	28	214.22	18.39	3	221.65	25.81
J-164	188114.1	2973095	196.3	42	214.19	17.89	4	221.65	25.35
J-165	188093.4	2973073	197.35	42	214.17	16.82	4	221.65	24.3
J-166	188082.8	2973062	198.74	42	214.17	15.43	4	221.65	22.91
J-167	188259.3	2973436	196.32	28	214.93	18.61	3	221.67	25.35
J-168	188245.5	2973408	196.5	28	215.1	18.6	3	221.67	25.17
J-169	188232.1	2973381	196.58	28	215.3	18.72	3	221.66	25.09
J-170	188219.2	2973355	196.25	28	215.52	19.27	3	221.66	25.41
J-171	188212.7	2973341	196	14	215.66	19.66	1	221.66	25.66
J-172	188217	2973313	195.5	21	215.94	20.44	2	221.66	26.16
J-173	188228.7	2973286	196.24	14	216.26	20.02	1	221.66	25.41
J-174	188218.9	2973258	196.47	0	215.82	19.35	0	221.65	25.18
J-175	188206.8	2973229	195.8	28	215.34	19.55	3	221.65	25.86
J-176	188199.6	2973216	195.72	28	215.14	19.42	3	221.65	25.93
J-177	188192.4	2973203	195.71	28	214.96	19.25	3	221.65	25.94
J-178	188185	2973190	195.7	28	214.8	19.1	3	221.65	25.95
J-179	188178.1	2973177	195.5	42	214.66	19.16	4	221.65	26.15
J-180	188163.4	2973151	195.69	14	214.44	18.75	1	221.65	25.96
J-181	188343.7	2973435	198.67	14	215.75	17.08	1	221.68	23.01
J-182	188332.9	2973404	197.66	14	215.94	18.28	1	221.68	24.02
J-183	188322.7	2973377	197.61	28	216.12	18.51	3	221.68	24.07
J-184	188309.8	2973344	198.17	28	216.36	18.18	3	221.68	23.51
J-185	188304.4	2973330	197.34	14	216.46	19.12	1	221.68	24.34
J-186	188275.8	2973300	196.86	0	216.58	19.72	0	221.66	24.79
J-187	188260.8	2973297	196.07	0	216.48	20.42	0	221.66	25.59

J-188	188298.3	2973316	197.24	0	216.56	19.32	0	221.68	24.44
J-189	188384.9	2974191	202.94	14	214.93	11.99	1	221.89	18.95
J-191	188343.3	2974791	213.91	0	220.32	6.41	0	222.15	8.24
J-192	188360.9	2974739	208.93	14	219.51	10.58	1	222.12	13.19
J-193	188379.9	2974635	209.37	28	218.14	8.77	3	222.06	12.69
J-194	188386.6	2974586	208.59	14	217.58	8.99	1	222.03	13.44
J-195	188387	2974557	207.75	14	217.29	9.54	1	222.02	14.27
J-196	188383.8	2974527	206.42	14	217.02	10.6	1	222	15.58
J-197	188383.4	2974497	205.58	14	216.76	11.18	1	221.99	16.41
J-198	188408.4	2974417	202.61	14	216.11	13.5	1	221.95	19.34
J-199	188421	2974389	204.5	28	215.9	11.4	3	221.94	17.44
J-200	188444.1	2974310	206.72	0	215.38	8.66	0	221.91	15.19
J-201	188465.9	2974280	206.5	14	215.16	8.66	1	221.9	15.4
J-202	188474.7	2974252	208.17	14	215.02	6.84	1	221.89	13.72
J-203	188482.4	2974228	208.54	14	214.9	6.36	1	221.89	13.34
J-204	188502.3	2974173	212.07	14	214.73	2.66	1	221.87	9.8
J-205	188506.7	2974115	211.07	14	214.56	3.48	1	221.86	10.78
J-206	188506.3	2974069	208.77	42	214.38	5.61	4	221.84	13.07
J-207	188511.5	2974019	208.06	0	214.24	6.18	0	221.82	13.76
J-208	188514	2974000	208.5	28	214.2	5.7	3	221.82	13.32
J-209	188508.1	2973956	206.63	14	214.14	7.51	1	221.81	15.18
J-210	188507.4	2973927	207.31	28	214.1	6.79	3	221.8	14.49
J-211	188502.2	2973897	206.66	14	214.08	7.42	1	221.79	15.13
J-212	188494.9	2973850	205.77	28	214.05	8.28	3	221.78	16.01
J-213	188486.7	2973807	210.8	28	214.04	3.24	3	221.77	10.97
J-214	188485.8	2973776	210.09	14	214.04	3.94	1	221.77	11.68
J-215	188485.5	2973747	210.08	14	214.03	3.95	1	221.76	11.68
J-216	188483.5	2973717	211.27	14	214.03	2.76	1	221.76	10.49
J-217	188455.1	2973710	206.21	14	214.04	7.82	1	221.75	15.54
J-218	188221.9	2974944	209.98	14	220.3	10.32	1	222.17	12.19
J-219	188232.9	2974933	210.53	14	220.44	9.91	1	222.17	11.64
J-220	188254.7	2974912	211.29	14	220.73	9.44	1	222.18	10.89
J-221	188281.5	2974881	212.3	14	221.12	8.82	1	222.19	9.89
J-222	188290.2	2974869	212.95	14	221.27	8.32	1	222.2	9.25
J-223	188317.2	2974833	213.94	14	221.73	7.79	1	222.21	8.27
J-224	188308.3	2974846	213.57	14	221.57	7.99	1	222.21	8.63
J-225	188366.6	2974709	209.04	14	219.08	10.04	1	222.1	13.06
J-226	188372	2974679	209.14	14	218.68	9.53	1	222.08	12.94
J-227	188374.6	2974664	209.23	14	218.49	9.26	1	222.07	12.84
J-228	188392	2974458	204.29	7	216.45	12.16	7	221.97	17.68
J-229	188445.6	2974336	205.79	14	215.55	9.76	1	221.92	16.14
J-232	188452.5	2974213	205	56	214.82	9.82	6	221.88	16.88
J-234	188505.2	2974134	211.39	0	214.64	3.25	0	221.86	10.48
J-235	188461.3	2974135	208	21	214.66	6.66	9	221.87	13.87
J-236	188432.1	2974134	206	14	214.69	8.69	1	221.87	15.87
J-237	188388.2	2974130	204	0	214.73	10.73	0	221.87	17.87

J-238	188359.7	2973769	197.2	0	214.04	16.84	0	221.74	24.53
J-239	188357	2973739	197.42	0	214.04	16.62	0	221.73	24.31
J-240	188171	2973164	195.59	0	214.55	18.96	0	221.65	26.06
J-242	188497.4	2973864	206.05	0	214.06	8.01	0	221.78	15.74
J-245	188433.3	2974362	205.14	0	215.72	10.58	0	221.93	16.79
J-246	188386.4	2974473	204.87	0	216.57	11.7	0	221.98	17.1
J-248	188397.1	2974446	203.77	0	216.35	12.57	0	221.97	18.19
J-249	188226.9	2974723	202.98	0	217.56	14.58	0	222.05	19.07

Table 15-2: Pipe Details-Physical, Flow and Velocity at Peak and Off-Peak Demand Hour[17]

Pipe Physical Data			Peak Demand Hour			Off Peak Demand Hour		
Pipe Label	Diameter (mm)	Length (m)	Head loss (m)	Flow (L/min)	Velocity (m/s)	Head loss (m)	Flow (L/min)	Velocity (m/s)
P-5	100	14	0.06	251	0.53	0	26	0.06
P-6	100	30	0.1	209	0.44	0	22	0.05
P-14	100	49	0.62	440	0.93	0.02	77	0.16
P-15	100	21	0.25	427	0.91	0.01	76	0.16
P-16	100	22	0.23	399	0.85	0.01	73	0.15
P-17	100	23	0.21	371	0.79	0.01	70	0.15
P-18	100	22	0.18	357	0.76	0.01	68	0.15
P-19	100	29	0.21	329	0.7	0.01	65	0.14
P-20	100	31	0.19	301	0.64	0.01	62	0.13
P-21	100	24	0.11	259	0.55	0.01	58	0.12
P-26	100	14	0.13	381	0.81	0.01	67	0.14
P-27	100	22	0.18	353	0.75	0.01	64	0.14
P-28	100	30	0.21	325	0.69	0.01	62	0.13
P-29	100	28	0.15	284	0.6	0.01	57	0.12
P-30	100	20	0.1	277	0.59	0.01	56	0.12
P-31	100	30	0.13	249	0.53	0.01	54	0.11
P-32	100	15	0.05	221	0.47	0	51	0.11
P-33	100	30	0.09	207	0.44	0.01	49	0.1
P-34	100	14	0.03	179	0.38	0	46	0.1
P-35	100	15	0.03	152	0.32	0	43	0.09
P-58	100	18	0.08	247	0.52	0.01	76	0.16
P-59	100	34	0.13	233	0.49	0.02	75	0.16
P-60	100	33	0.11	219	0.46	0.01	73	0.16
P-61	100	33	0.09	191	0.4	0.01	70	0.15
P-62	100	37	0.07	163	0.35	0.01	67	0.14
P-64	100	40	0.06	136	0.29	0.02	70	0.15

[17] Pipes are labeled in CAD file which shows the master layout of the distribution system.

P-65	100	40	0.03	94	0.2	0.01	65	0.14
P-66	100	36	0.01	52	0.11	0.01	61	0.13
P-67	100	37	0	10	0.02	0.01	57	0.12
P-68	100	33	0	18	0.04	0.01	54	0.11
P-87	100	30	0.1	213	0.45	0.02	83	0.18
P-90	100	30	0.01	74	0.16	0.01	62	0.13
P-91	100	45	0.01	46	0.1	0.01	60	0.13
P-97	100	28	0.13	260	0.55	0.01	70	0.15
P-99	100	32	0.12	229	0.49	0.01	68	0.14
P-100	100	15	0.04	187	0.4	0.01	64	0.14
P-101	100	30	0.06	159	0.34	0.01	61	0.13
P-102	100	15	0.02	131	0.28	0	58	0.12
P-103	100	15	0.01	104	0.22	0	55	0.12
P-104	100	15	0.01	90	0.19	0	54	0.11
P-105	100	30	0.01	48	0.1	0.01	49	0.1
P-106	100	30	0	27	0.06	0.01	47	0.1
P-107	100	30	0	1	0	0.01	44	0.09
P-108	100	15	0	28	0.06	0	41	0.09
P-109	100	30	0.01	56	0.12	0	38	0.08
P-110	100	30	0.02	84	0.18	0	35	0.07
P-111	100	47	0.05	112	0.24	0	32	0.07
P-116	100	14	0.03	172	0.36	0.01	78	0.17
P-117	100	46	0.06	130	0.28	0.02	68	0.14
P-118	100	15	0.04	192	0.41	0.01	80	0.17
P-119	100	15	0.04	192	0.41	0.01	80	0.17
P-134	100	30	0	26	0.06	0.01	53	0.11
P-135	100	30	0.01	68	0.14	0.01	49	0.1
P-136	100	22	0.02	110	0.23	0	44	0.09
P-137	100	24	0.04	138	0.29	0	41	0.09
P-138	100	18	0.01	67	0.14	0	47	0.1
P-139	100	28	0.02	95	0.2	0	44	0.09
P-140	100	27	0.04	137	0.29	0	39	0.08
P-141	100	31	0.06	165	0.35	0	37	0.08
P-142	100	28	0.09	206	0.44	0	32	0.07
P-144	100	29	0.02	85	0.18	0.01	45	0.1
P-145	100	46	0.06	126	0.27	0.01	41	0.09
P-146	100	15	0	44	0.09	0	57	0.12
P-147	100	29	0.01	72	0.15	0.01	54	0.12
P-148	100	15	0.03	177	0.38	0	45	0.1
P-149	100	30	0.09	205	0.43	0	43	0.09
P-150	100	15	0.06	233	0.49	0	40	0.08
P-151	100	30	0.14	261	0.55	0	37	0.08
P-152	100	30	0.17	289	0.61	0	34	0.07
P-153	100	17	0.11	302	0.64	0	32	0.07
P-154	100	52	0.43	352	0.75	0	10	0.02
P-157	100	45	0.09	168	0.36	0.02	67	0.14

P-158	100	15	0.09	293	0.62	0	46	0.1
P-159	100	30	0.19	306	0.65	0.01	45	0.1
P-160	100	30	0.21	320	0.68	0.01	43	0.09
P-161	100	15	0.11	334	0.71	0	42	0.09
P-162	100	30	0.24	348	0.74	0	40	0.09
P-163	100	30	0.26	362	0.77	0	39	0.08
P-164	100	36	0.35	390	0.83	0	36	0.08
P-165	100	35	0.46	452	0.96	0	26	0.05
P-166	100	40	0.44	410	0.87	0	21	0.05
P-167	100	53	0.06	117	0.25	0.01	39	0.08
P-168	100	46	0.03	90	0.19	0.01	36	0.08
P-169	100	46	0.02	62	0.13	0	33	0.07
P-170	100	53	0.01	34	0.07	0	30	0.06
P-171	100	47	0	6	0.01	0	27	0.06
P-172	100	51	0.01	50	0.11	0	21	0.04
P-173	100	47	0.02	78	0.16	0	18	0.04
P-174	100	25	0.03	133	0.28	0	12	0.03
P-175	100	15	0.02	147	0.31	0	11	0.02
P-176	100	31	0.05	153	0.33	0	16	0.03
P-177	100	30	0.04	125	0.27	0	13	0.03
P-178	100	30	0.02	84	0.18	0	9	0.02
P-179	100	15	0	42	0.09	0	4	0.01
P-180	100	39	0.16	244	0.52	0.01	44	0.09
P-181	100	31	0.17	286	0.61	0	40	0.08
P-182	100	30	0.2	314	0.67	0	37	0.08
P-183	100	29	0.23	342	0.73	0	34	0.07
P-184	100	16	0.13	356	0.75	0	32	0.07
P-185	100	29	0.26	370	0.78	0	31	0.07
P-186	100	30	0.29	384	0.81	0	29	0.06
P-187	100	29	0.43	482	1.02	0	24	0.05
P-188	100	31	0.44	468	0.99	0	23	0.05
P-189	100	15	0.19	454	0.96	0	21	0.05
P-190	100	15	0.18	426	0.9	0	18	0.04
P-191	100	15	0.15	398	0.84	0	16	0.03
P-192	100	15	0.14	370	0.79	0	13	0.03
P-194	100	23	0.14	301	0.64	0	5	0.01
P-210	100	66	0.04	85	0.18	0	7	0.01
P-211	100	64	0.06	110	0.23	0	19	0.04
P-212	100	65	0	14	0.03	0	7	0.01
P-215	100	35	0.18	274	0.58	0.02	79	0.17
P-298	100	30	0	18	0.04	0.01	57	0.12
P-300	100	31	0	4	0.01	0.01	55	0.12
P-301	100	14	0	24	0.05	0	52	0.11
P-302	100	14	0.11	342	0.73	0	10	0.02
P-303	100	15	0.11	328	0.7	0	8	0.02
P-313	100	32	0.12	231	0.49	0.01	55	0.12

P-314	100	13	0.04	204	0.43	0	52	0.11
P-126	100	35	0.02	84	0.18	0	9	0.02
P-127	100	32	0	28	0.06	0	3	0.01
P-128	100	36	0.01	42	0.09	0	4	0.01
P-129	100	32	0	14	0.03	0	1	0
P-130	100	29	0.04	125	0.27	0	13	0.03
P-132	100	60	0.07	125	0.27	0	13	0.03
P-217	100	63	0.01	47	0.1	0	5	0.01
P-218	100	39	0.07	149	0.32	0.01	66	0.14
P-219	100	31	0.11	227	0.48	0.02	84	0.18
P-223	100	30	0.11	232	0.49	0.01	67	0.14
P-224	100	53	0.08	140	0.3	0.02	70	0.15
P-225	100	24	0	1	0	0.01	56	0.12
P-226	100	20	0.01	71	0.15	0	47	0.1
P-232	100	55	0.83	489	1.04	0.03	88	0.19
P-235	100	30	0.3	392	0.83	0.01	78	0.17
P-236	100	30	0.28	378	0.8	0.01	76	0.16
P-237	100	30	0.27	364	0.77	0.01	75	0.16
P-239	100	31	0.2	315	0.67	0.01	63	0.13
P-241	100	41	0.21	273	0.58	0.01	59	0.12
P-242	100	29	0.13	259	0.55	0.01	57	0.12
P-243	100	25	0.1	231	0.49	0.01	54	0.12
P-246	100	47	0.18	233	0.5	0.02	67	0.14
P-247	100	50	0.13	191	0.41	0.02	62	0.13
P-248	100	69	0	25	0.05	0	4	0.01
P-249	100	20	0.03	153	0.32	0.01	57	0.12
P-250	100	45	0.07	139	0.29	0.01	55	0.12
P-251	100	29	0.04	125	0.27	0.01	54	0.11
P-252	100	30	0.02	97	0.21	0.01	51	0.11
P-254	100	44	0.01	55	0.12	0.01	47	0.1
P-255	100	31	0	27	0.06	0.01	44	0.09
P-256	100	30	0	13	0.03	0	42	0.09
P-257	100	30	0	0	0	0	41	0.09
P-258	100	30	0	14	0.03	0	39	0.08
P-259	100	20	0	28	0.06	0	38	0.08
P-274	100	30	0.44	475	1.01	0.02	87	0.18
P-276	100	30	0.41	461	0.98	0.02	85	0.18
P-278	100	15	0.19	447	0.95	0.01	84	0.18
P-279	100	30	0.36	433	0.92	0.02	82	0.17
P-283	100	31	0.16	273	0.58	0.01	59	0.12
P-285	100	29	0.07	175	0.37	0.01	49	0.1
P-286	100	17	0.06	231	0.49	0	54	0.12
P-287	100	14	0.03	175	0.37	0	49	0.1
P-288	100	37	0.01	56	0.12	0	6	0.01
P-291	100	41	0.08	161	0.34	0.01	47	0.1
P-292	100	19	0.08	247	0.52	0.01	68	0.14

P-293	100	44	0.03	86	0.18	0	21	0.04
P-294	100	29	0.03	107	0.23	0	30	0.06
P-297	100	44	0.05	121	0.26	0	31	0.07
P-304	100	33	0.02	83	0.18	0.01	49	0.1
P-305	100	14	0.01	69	0.15	0	48	0.1
P-306	100	30	0.18	301	0.64	0.01	62	0.13
P-307	100	30	0.17	287	0.61	0.01	60	0.13
P-308	100	24	0.19	350	0.74	0.01	73	0.16
P-309	100	16	0.12	336	0.71	0.01	72	0.15
P-310	100	14	0.1	329	0.7	0	65	0.14
P-311	100	31	0.22	329	0.7	0.01	65	0.14
P-312	100	53	0.57	406	0.86	0.03	79	0.17
P-1	150	33	0.35	1170	1.1	0.01	181	0.17
P-13	150	41	0.23	822	0.77	0.01	145	0.14
P-23	150	25	0.19	984	0.93	0.01	188	0.18
P-24	150	39	0.28	956	0.9	0.01	185	0.17
P-25	150	42	0.27	901	0.85	0.01	180	0.17
P-37	150	17	0.15	1047	0.99	0.01	197	0.19
P-38	150	29	0.24	1019	0.96	0.01	194	0.18
P-39	150	30	0.23	991	0.93	0.01	191	0.18
P-40	150	30	0.22	963	0.91	0.01	188	0.18
P-41	150	41	0.87	1710	1.61	0.03	293	0.28
P-42	150	34	1.2	2248	2.12	0.05	393	0.37
P-44	150	46	0.28	873	0.82	0.01	177	0.17
P-45	150	23	0.13	845	0.8	0.01	174	0.16
P-46	150	30	0.16	817	0.77	0.01	171	0.16
P-47	150	30	0.15	789	0.74	0.01	168	0.16
P-48	150	23	0.11	747	0.7	0.01	163	0.15
P-49	150	32	0.16	767	0.72	0.01	166	0.16
P-50	150	43	0.19	725	0.68	0.01	162	0.15
P-51	150	39	0.15	670	0.63	0.01	156	0.15
P-52	150	33	0.11	642	0.61	0.01	153	0.14
P-53	150	40	0.13	614	0.58	0.01	150	0.14
P-54	150	34	0.1	586	0.55	0.01	147	0.14
P-55	150	33	0.09	558	0.53	0.01	144	0.14
P-56	150	55	0.13	516	0.49	0.01	140	0.13
P-69	150	57	0.4	935	0.88	0.02	185	0.17
P-70	150	30	0.2	907	0.86	0.01	182	0.17
P-71	150	15	0.09	879	0.83	0	179	0.17
P-72	150	31	0.18	851	0.8	0.01	176	0.17
P-73	150	30	0.16	817	0.77	0.01	173	0.16
P-74	150	30	0.14	755	0.71	0.01	166	0.16
P-75	150	30	0.13	728	0.69	0.01	163	0.15
P-77	150	30	0.11	644	0.61	0.01	154	0.15
P-78	150	38	0.12	616	0.58	0.01	151	0.14
P-79	150	30	0.09	588	0.55	0.01	148	0.14

P-80	150	30	0.08	574	0.54	0.01	147	0.14
P-83	150	30	0.08	542	0.51	0.01	160	0.15
P-84	150	30	0.07	507	0.48	0.01	156	0.15
P-85	150	29	0.06	493	0.47	0.01	155	0.15
P-112	150	29	0.12	700	0.66	0.01	160	0.15
P-113	150	29	0.11	672	0.63	0.01	157	0.15
P-195	150	12	0.07	842	0.79	0	10	0.01
P-196	150	32	0.19	856	0.81	0	9	0.01
P-197	150	29	0.18	870	0.82	0	7	0.01
P-198	150	36	0.23	898	0.85	0	4	0
P-199	150	15	0.1	925	0.87	0	1	0
P-201	150	15	0.1	914	0.86	0	0	0
P-202	150	34	0.22	893	0.84	0	2	0
P-203	150	52	0	48	0.04	0	6	0.01
P-204	150	64	0.43	914	0.86	0	0	0
P-205	150	15	0.1	925	0.87	0	1	0
P-207	150	61	0.43	939	0.89	0	0	0
P-208	150	62	0.03	214	0.2	0	58	0.05
P-209	150	54	0.28	794	0.75	0.01	138	0.13
P-213	150	36	0.09	546	0.52	0.01	144	0.14
P-214	150	34	0.08	532	0.5	0.01	142	0.13
P-229	150	30	0.75	1855	1.75	0.03	336	0.32
P-230	150	20	0.71	2248	2.12	0.03	393	0.37
P-231	150	39	0.88	1759	1.66	0.03	305	0.29
P-260	150	41	0.37	1072	1.01	0.01	171	0.16
P-262	150	16	0.14	1086	1.02	0	172	0.16
P-264	150	30	0.29	1100	1.04	0.01	174	0.16
P-266	150	41	0.39	1114	1.05	0.01	175	0.17
P-268	150	15	0.15	1128	1.06	0	177	0.17
P-271	150	14	0.15	1170	1.1	0	181	0.17
P-272	150	30	0.3	1142	1.08	0.01	178	0.17
P-273	150	16	0.16	1156	1.09	0.01	180	0.17
P-295	150	28	0.12	719	0.68	0.01	197	0.19
P-296	150	17	0.05	584	0.55	0	164	0.15

Table 15-3: Reservoir 1-LAP reservoir Capacity requirement Details.

Time	Demand	Supply	Balance	TOTAL m^3
00:00	36	0	-36	-36
01:00	36	0	-36	-72
02:00	36	0	-36	-108
03:00	36	150	114	6
04:00	36	150	114	120
05:00	71	150	79	199

06:00	121	150	29	228
07:00	235	150	-85	143
08:00	235	150	-85	58
09:00	235	150	-85	-27
10:00	235	150	-85	-112
11:00	134	150	16	-96
12:00	134	150	16	-80
13:00	121	150	29	-51
14:00	235	150	-85	-136
15:00	113	150	37	-99
16:00	113	150	37	-62
17:00	113	150	37	-25
18:00	235	150	-85	-110
19:00	171	150	-21	-131
20:00	121	150	29	-102
21:00	113	150	37	-65
22:00	113	150	37	-28
23:00	71	99	28	0
Total	3099	3099		
Required tank volume (excluding fire reserve)				364
Total Daily Demand			3099	m3
Pumping rate			150	m3/hr
Required pumping hour			20.66	hr
Required Reservoir Volume			673.9	m3
Existing reservoir capacity			250	m3
Additional reservoir Capacity required			**423.9**	**m3**

Table 15-4: Reservoir 2-Temporary Shelter Reservoir Capacity Requirement Details

Time	Demand	Supply	Balance	TOTAL m^3
00:00	0	0	0	0
01:00	0	0	0	0
02:00	0	0	0	0
03:00	0	0	0	0
04:00	0	0	0	0
05:00	0	0	0	0
06:00	19	150	131	131
07:00	156	0	-156	-25
08:00	156	150	-6	-31
09:00	156	150	-6	-37
10:00	156	150	-6	-43

11:00	40	150	110	67
12:00	40	0	-40	27
13:00	19	0	-19	8
14:00	156	150	-6	2
15:00	9	0	-9	-7
16:00	9	150	141	134
17:00	9	0	-9	125
18:00	156	150	-6	119
19:00	88	0	-88	31
20:00	19	6	-13	18
21:00	9	0	-9	9
22:00	9	0	-9	0
23:00	0	0	0	0
Total	1206	1206		
Required tank volume (excluding fire reserve)				**177**
Total Daily Demand			1206	m3
Pumping rate			150	m3/hr
Required pumping hour			8	hr
Required Reservoir Volume			298	m3
Existing reservoir capacity			300	m3
Additional reservoir Capacity required			**0**	**m3**

16 Annexure E- Sewer System Design Results

Table 16-1: Primary Sewer pipe Data Sheet

Primary Sewer Pipe Data Sheet												
SL No.	Manhole Label	Chainage	X (m)	Y (m)	Distance (m)	Pipe Diameter (mm)	Manhole Diameter (mm)	Invert Elevation (m)	Ground Elevation (m)	Invert Depth (m)	Slope(%)	Flow* (Total In) (L/min)
1	MH69	0+000.000	188143.86	2974917.11	0.000	225	1500	201.39	203.66	2.27	0	69
2	MH68	0+025.307	188156.98	2974895.47	25.307	225	1500	200.72	203.94	3.22	0.71	96
3	MH67	0+051.035	188170.22	2974873.41	25.728	225	1500	200.06	203.72	3.66	0.7	123
4	MH66	0+073.863	188181.28	2974853.44	22.828	225	1500	199.47	203.54	4.07	0.7	145
5	MH65	0+095.956	188190.73	2974833.47	22.093	225	1500	199.31	203.62	4.31	0.72	171
6	MH64	0+119.416	188198.6	2974811.37	23.459	225	1500	199.15	203.63	4.48	0.68	198
7	MH63	0+154.246	188209.86	2974778.41	34.830	225	1500	198.9	203.77	4.87	0.72	227
8	MH62	0+184.062	188218.75	2974749.95	29.816	225	1500	198.69	203.89	5.2	0.7	255
9	MH61	0+213.876	188228.13	2974721.65	29.814	225	1500	198.49	204	5.51	0.67	283
10	MH60	0+242.628	188237.42	2974694.44	28.752	225	1500	198.28	204	5.72	0.73	310
11	MH59	0+263.885	188243.79	2974674.16	21.257	225	1500	198.14	204	5.86	0.66	894
12	MH58	0+286.974	188251.31	2974652.33	23.089	225	1500	197.97	204	6.03	0.74	926
13	MH57	0+313.346	188256.82	2974626.54	26.372	225	1500	197.79	204.04	6.25	0.68	959
14	MH56	0+331.047	188260.52	2974609.23	17.701	250	1500	197.67	204.19	6.52	0.68	988
15	MH55	0+372.816	188268.9	2974568.31	41.769	250	1500	197.41	203.88	6.47	0.62	1028
16	MH54	0+403.964	188275.22	2974537.81	31.148	250	1500	197.23	203.62	6.39	0.58	1063
17	MH53	0+430.103	188280.35	2974512.18	26.138	250	1500	197.07	203.41	6.34	0.61	1105
18	MH52	0+457.936	188286.02	2974484.93	27.834	250	1500	196.9	203.18	6.28	0.61	1137
19	MH51	0+484.102	188290.61	2974459.17	26.166	250	1500	196.75	202.97	6.22	0.57	1180
20	MH50	0+514.296	188296.16	2974429.49	30.194	250	1500	196.57	202.71	6.14	0.6	1234
21	MH49	0+546.911	188300.15	2974397.12	32.615	250	1500	196.37	202.44	6.07	0.61	1290
22	MH48	0+581.979	188304.05	2974362.27	35.068	250	1500	196.16	202.15	5.99	0.6	1868
23	MH47	0+610.085	188306.71	2974334.29	28.106	250	1500	195.99	201.89	5.9	0.6	1906
24	MH46	0+637.992	188309.67	2974306.54	27.907	250	1500	195.82	201.57	5.75	0.61	1933
25	MH45	0+656.859	188311.76	2974287.79	18.866	250	1500	195.71	202	6.29	0.58	1943
26	MH44	0+680.537	188314.32	2974264.25	23.679	250	1500	195.57	201.94	6.37	0.59	1987
27	MH43	0+704.522	188316.56	2974240.37	23.985	250	1500	195.43	201.85	6.42	0.58	2042
28	MH42	0+734.134	188319.24	2974210.88	29.612	250	1500	195.25	201.75	6.5	0.61	2078

29	MH41	0+765.035	188322.09	2974180.11	30.902	250	1500	195.06	201.71	6.65	0.61	2115
30	MH40	0+788.812	188324.23	2974156.43	23.777	250	1500	194.92	201.71	6.79	0.59	2158
31	MH39	0+828.780	188327.55	2974116.6	39.968	315	1500	194.68	201.89	7.21	0.6	2230
32	MH38	0+847.910	188327.51	2974097.47	19.130	315	1500	194.59	201.72	7.13	0.47	2251
33	MH37	0+876.244	188325.61	2974069.2	28.334	315	1500	194.47	201.46	6.99	0.42	2282
34	MH36	0+902.048	188323.3	2974043.5	25.804	315	1500	194.38	201.23	6.85	0.35	2323
35	MH35	0+929.620	188320.7	2974016.05	27.573	315	1500	194.24	200.97	6.73	0.51	2345
36	MH34	0+953.832	188318.28	2973991.96	24.211	315	1500	194.13	200.75	6.62	0.45	2372
37	MH33	0+983.223	188315.61	2973962.69	29.392	315	1500	194	200.48	6.48	0.44	2395
38	MH32	1+013.536	188312.47	2973932.54	30.313	315	1500	193.87	200.19	6.32	0.43	2435
39	MH31	1+042.465	188309.74	2973903.74	28.929	315	1500	193.74	199.92	6.18	0.45	2478
40	MH30	1+072.464	188306.66	2973873.9	29.999	315	1500	193.6	199.64	6.04	0.47	2521
41	MH29	1+101.643	188303.43	2973844.9	29.179	315	1500	193.48	199.37	5.89	0.41	2554
42	MH28	1+130.448	188300.55	2973816.24	28.804	315	1500	193.35	199.1	5.75	0.45	2585
43	MH27	1+156.288	188297.96	2973790.53	25.840	315	1500	193.23	198.86	5.63	0.46	2605
44	MH26	1+182.427	188295.08	2973764.55	26.139	315	1500	193.12	198.62	5.5	0.42	2647
45	MH25	1+205.299	188292.2	2973741.86	22.872	315	1500	193.02	198.28	5.26	0.44	3927
46	MH24	1+235.344	188288.07	2973712.1	30.045	315	1500	192.89	198.26	5.37	0.43	3975
47	MH23	1+263.973	188284.23	2973683.73	28.629	315	1500	192.76	197.98	5.22	0.45	4019
48	MH22	1+286.901	188277.22	2973661.9	22.928	315	1500	192.66	197.83	5.17	0.44	4058
49	MH21	1+311.210	188269.77	2973638.76	24.310	315	1500	192.55	197.67	5.12	0.45	4108
50	MH20	1+339.119	188261.23	2973612.19	27.909	315	1500	192.43	197.49	5.06	0.43	4139
51	MH19	1+367.148	188252.67	2973585.5	28.029	315	1500	192.31	197.31	5	0.43	4201
52	MH18	1+396.182	188243.66	2973557.9	29.033	315	1500	192.18	197.12	4.94	0.45	4242
53	MH17	1+425.996	188234.65	2973529.48	29.814	315	1500	192.05	196.9	4.85	0.44	4282
54	MH16	1+452.412	188226.57	2973504.33	26.416	315	1500	191.94	196.7	4.76	0.42	4331
55	MH15	1+475.093	188219.65	2973482.73	22.681	315	1500	191.84	196.54	4.7	0.44	4822
56	MH14	1+495.097	188213.64	2973463.65	20.004	315	1500	191.75	196.48	4.73	0.45	4869
57	MH13	1+519.649	188205.93	2973440.34	24.552	315	1500	191.64	196.38	4.74	0.45	4910
58	MH12	1+548.797	188196.98	2973412.6	29.148	315	1500	191.51	196.27	4.76	0.45	4953
59	MH11	1+575.667	188188.6	2973387.07	26.870	315	1500	191.39	196.17	4.78	0.45	4993
60	MH10	1+597.570	188181.8	2973366.25	21.902	315	1500	191.3	196.08	4.78	0.41	5030
61	MH9	1+617.788	188175.65	2973346.99	20.218	315	1500	191.21	196.02	4.81	0.45	5065
62	MH8	1+646.952	188166.74	2973319.22	29.164	315	1500	191.08	195.89	4.81	0.45	5116

63	MH7	1+676.272	188158.01	2973291.23	29.320	315	1500	190.95	195.77	4.82	0.44	5155
64	MH6	1+701.246	188150.35	2973267.46	24.974	315	1500	190.84	195.67	4.83	0.44	5192
65	MH5	1+730.745	188141.28	2973239.39	29.499	315	1500	190.71	195.56	4.85	0.44	5241
66	MH4	1+760.366	188132.31	2973211.16	29.621	315	1500	190.58	195.39	4.81	0.44	5280
67	MH3	1+788.089	188123.51	2973184.87	27.724	315	1500	190.46	195.19	4.73	0.43	5317
68	MH2	1+812.525	188113.17	2973162.73	24.436	315	1500	190.35	195.02	4.67	0.45	5942
69	MH1	1+846.787	188082.41	2973177.82	34.262	315	1500	190.2	196	5.8	0.44	5966

Table 16-2: Secondary Sewer pipe Data sheet

Secondary Sewer Pipe Data Sheet											
SL No.	**Manhole Label**	**Chainage**	**X (m)**	**Y (m)**	**Distance (m)**	**Pipe Diameter (mm)**	**Manhole Diameter (mm)**	**Invert Elevation (m)**	**Ground Elevation (m)**	**Invert Depth (m)**	**Slope (%)**
	MH2	0+000.000	188113.17	2973162.73			1500	191.75	195.02	3.27	
1	MH2-1	0+024.852	188136.2	2973153.39	24.852	150	1350	191.95	195.11	3.16	0.80
2	MH2-2	0+054.024	188160.46	2973137.19	29.172	150	1350	192.16	195.82	3.66	0.72
3	MH2-3	0+066.457	188168.79	2973146.42	12.433	150	1350	192.27	195.96	3.69	0.88
4	MH2-4	0+091.545	188180.83	2973168.43	25.088	150	1350	192.47	196	3.53	0.80
5	MH2-5	0+121.372	188195.03	2973194.66	29.827	150	1350	192.71	196	3.29	0.80
6	MH2-6	0+152.253	188210.09	2973221.62	30.881	150	1350	192.96	196.26	3.30	0.81
7	MH2-7	0+176.580	188220.26	2973243.72	24.328	150	1350	193.16	196.41	3.25	0.82
8	MH2-8	0+197.994	188227.99	2973263.69	21.414	150	1350	193.32	196.5	3.18	0.75
9	MH2-9	0+224.835	188238.11	2973288.55	26.841	150	1350	193.51	196.01	2.50	0.71
10	MH2-10	0+244.818	188227.29	2973305.35	19.983	150	1350	193.67	196.39	2.72	0.80
11	MH2-11	0+264.040	188217.95	2973322.15	19.222	150	1350	193.82	196.33	2.51	0.78
12	MH2-12	0+274.601	188215.95	2973332.52	10.561	150	1350	193.92	196.33	2.41	0.95
13	MH2-13	0+287.087	188220.67	2973344.08	12.486	150	1350	194.02	196.42	2.40	0.80
14	MH2-14	0+307.817	188229.78	2973362.7	20.729	150	1350	194.17	196.59	2.42	0.72
15	MH2-15	0+326.564	188238.22	2973379.44	18.747	150	1350	194.34	196.74	2.40	0.91
16	MH2-16	0+347.538	188247.6	2973398.2	20.974	150	1350	194.51	196.94	2.43	0.81
17	MH2-17	0+369.076	188257.2	2973417.48	21.538	150	1350	194.68	197.12	2.44	0.79
18	MH2-18	0+389.688	188266.49	2973435.88	20.612	150	1350	194.84	197.27	2.43	0.78
	MH2-2	0+000.000	188160.46	2973137.19			1350	192.16	195.82	3.66	
19	MH2-2-1	0+027.317	188142.27	2973116.81	27.317	150	1350	193.1	195.97	2.87	3.44
20	MH2-2-2	0+056.221	188122.21	2973096	28.904	150	1350	193.6	196.33	2.73	1.73
21	MH2-2-3	0+079.506	188106.67	2973078.66	23.284	150	1350	194.1	196.9	2.80	2.15
22	MH2-2-4	0+104.971	188089.67	2973059.7	25.465	150	1350	196	198.25	2.25	7.46
	MH15	0+000.000	188219.65	2973482.73			1500	193.24	196.54	3.30	
23	MH15-1	0+031.669	188249.98	2973473.62	31.669	200	1350	193.88	197.09	3.21	2.02
24	MH15-2	0+063.762	188280.61	2973464.04	32.093	200	1350	194.12	197.32	3.20	0.75
25	MH15-3	0+082.116	188288.89	2973480.42	18.354	200	1350	194.27	197	2.73	0.82
26	MH15-4	0+108.673	188300.47	2973504.32	26.558	200	1350	194.47	197.1	2.63	0.75

27	MH15-5	0+131.521	188307.34	2973526.11	22.847	200	1350	194.66	197.15	2.49	0.83
28	MH15-6	0+154.017	188314.12	2973547.56	22.496	200	1350	194.84	197.33	2.49	0.80
29	MH15-7	0+184.380	188323.4	2973576.47	30.363	200	1350	195.08	197.55	2.47	0.79
30	MH15-8	0+214.564	188332.72	2973605.18	30.185	200	1350	195.32	197.8	2.48	0.80
31	MH15-9	0+244.359	188342.13	2973633.45	29.795	200	1350	195.56	198.09	2.53	0.81
32	MH15-10	0+274.581	188351.1	2973662.31	30.222	200	1350	195.79	198.23	2.44	0.76
33	MH15-11	0+299.892	188355.64	2973687.21	25.311	200	1350	196	198.2	2.20	0.83
	MH15-2	0+000.000	188280.61	2973464.04		200	1350	194.12	197.32	3.20	
34	MH15-2-1	0+037.862	188317.32	2973454.77	37.862	200	1350	195.2	197.65	2.45	2.85
35	MH15-2-2	0+066.016	188344.64	2973447.97	28.154	200	1350	196.6	198.47	1.87	4.97
	MH25	0+000.000	188292.2	2973741.86			1500	194.42	198.28	3.86	
36	MH25-1	0+036.906	188327.01	2973729.6	36.906	200	1350	195.14	198.52	3.38	1.95
37	MH25-2	0+071.472	188360.94	2973723	34.566	200	1350	195.36	198.94	3.58	0.64
38	MH25-3	0+103.300	188363.9	2973754.69	31.828	200	1350	195.56	199.22	3.66	0.63
39	MH25-4	0+136.262	188367.55	2973787.45	32.963	200	1350	195.78	199.46	3.68	0.67
40	MH25-5	0+173.840	188371.49	2973824.82	37.577	200	1350	196.03	199.73	3.70	0.67
41	MH25-6	0+211.571	188375.84	2973862.3	37.732	200	1350	196.3	200.02	3.72	0.72
42	MH25-7	0+237.834	188378.38	2973888.44	26.263	200	1350	196.47	200.11	3.64	0.65
43	MH25-8	0+264.631	188381.27	2973915.08	26.796	200	1350	197.55	200.1	2.55	4.03
44	MH25-9	0+290.784	188383.91	2973941.1	26.154	200	1350	197.75	200.33	2.58	0.76
45	MH25-10	0+319.361	188386.99	2973969.51	28.576	200	1350	198.05	200.59	2.54	1.05
46	MH25-11	0+348.215	188389.97	2973998.21	28.854	200	1350	198.35	200.85	2.50	1.04
47	MH25-12	0+378.422	188392.95	2974028.27	30.207	200	1350	198.84	201.32	2.48	1.62
48	MH25-13	0+397.201	188394.56	2974046.98	18.779	150	1350	199.35	201.78	2.43	2.72
49	MH25-14	0+416.052	188396.41	2974065.74	18.851	150	1350	199.92	202.38	2.46	3.02
50	MH25-15	0+438.883	188397.93	2974088.52	22.831	150	1350	200.6	202.95	2.35	2.98
51	MH25-16	0+468.929	188397.31	2974118.56	30.046	150	1350	201.67	204.14	2.47	3.56
	MH25-2	0+000.000	188360.94	2973723			1350	195.36	198.94	3.58	
52	MH25-2-1	0+034.171	188394.51	2973716.62	34.171	200	1350	195.6	199.26	3.66	0.70
53	MH25-2-2	0+065.035	188424.88	2973711.12	30.864	200	1350	197.6	199.5	1.90	6.48
	MH25-7	0+000.000	188378.38	2973888.44			1350	196.47	200.11	3.64	

54	MH25-7-1	0+030.125	188408.43	2973886.31	30.125	200	1350	196.69	200.41	3.72	0.73
55	MH25-7-2	0+063.632	188441.86	2973884.05	33.506	200	1350	196.89	200.64	3.75	0.60
56	MH25-7-3	0+082.544	188439.24	2973865.32	18.912	200	1350	197.02	200.76	3.74	0.69
57	MH25-7-4	0+104.749	188437.42	2973843.19	22.205	200	1350	197.17	201	3.83	0.68
58	MH25-7-5	0+132.765	188435.35	2973815.25	28.017	200	1350	197.36	200.98	3.62	0.68
59	MH25-7-6	0+160.307	188433.1	2973787.8	27.542	200	1350	197.54	200.31	2.77	0.65
60	MH25-7-7	0+186.935	188431.07	2973761.25	26.627	200	1350	197.72	200.53	2.81	0.68
61	MH25-7-8	0+215.710	188428.73	2973732.57	28.775	200	1350	197.91	200.41	2.50	0.66
	MH25-12	0+000.000	188392.95	2974028.27			1350	198.84	201.32	2.48	
62	MH25-12-1	0+030.942	188423.89	2974027.94	30.942	150	1350	199.4	201.77	2.37	1.81
	MH25-7-2	0+000.000	188441.86	2973884.05			1350	196.89	200.64	3.75	
63	MH25-7-2-1	0+034.554	188446.86	2973918.24	34.554	150	1350	197.6	200.37	2.77	2.05
64	MH25-7-2-2	0+064.135	188451.07	2973947.52	29.581	150	1350	197.94	200.41	2.47	1.15
65	MH25-7-2-3	0+092.995	188455.36	2973976.06	28.861	150	1350	198.43	200.84	2.41	1.70
66	MH25-7-2-4	0+119.037	188457.87	2974001.98	26.041	150	1350	199.32	201.72	2.40	3.42
	MH48	0+000.000	188304.05	2974362.27			1500	197.56	202.15	4.59	
67	MH48-1	0+030.869	188334.72	2974365.77	30.869	200	1350	198.35	202.14	3.79	2.56
68	MH48-2	0+060.938	188364.57	2974369.39	30.069	200	1350	198.83	202.15	3.32	1.60
69	MH48-3	0+087.835	188368.42	2974342.77	26.897	200	1350	199.04	201.9	2.86	0.78
70	MH48-4	0+112.106	188371.9	2974318.75	24.271	200	1350	199.24	201.64	2.40	0.82
71	MH48-5	0+133.300	188374.98	2974297.78	21.195	200	1350	199.4	202	2.60	0.75
72	MH48-6	0+157.112	188378.76	2974274.27	23.812	200	1350	199.59	202	2.41	0.80
73	MH48-7	0+188.042	188383.26	2974243.67	30.929	200	1350	199.82	202.01	2.19	0.74
74	MH48-8	0+216.244	188387.17	2974215.74	28.202	200	1350	200.07	202.53	2.46	0.89
75	MH48-9	0+235.856	188389.69	2974196.29	19.613	200	1350	200.32	203.02	2.70	1.27
76	MH48-10	0+262.894	188393.11	2974169.47	27.037	200	1350	200.64	203.09	2.45	1.18
	MH48-2	0+000.000	188364.57	2974369.39			1350	198.83	202.15	3.32	
77	MH48-2-1	0+028.930	188360.7	2974398.06	28.930	150	1350	199.29	202.38	3.09	1.59
78	MH48-2-2	0+059.381	188356.43	2974428.21	30.451	150	1350	199.81	202.66	2.85	1.71
79	MH48-2-3	0+089.562	188350.01	2974457.7	30.181	150	1350	200.32	202.88	2.56	1.69
80	MH48-2-4	0+112.627	188344.89	2974480.19	23.065	150	1350	200.71	203.77	3.06	1.69

81	MH48-2-5	0+136.591	188339.33	2974503.5	23.964	150	1350	201.1	204.12	3.02	1.63
82	MH48-2-6	0+160.635	188334.02	2974526.95	24.044	150	1350	201.5	204.7	3.20	1.66
83	MH48-2-7	0+184.270	188328.75	2974549.99	23.635	150	1350	201.9	205.01	3.11	1.69
84	MH48-2-8	0+208.805	188323.29	2974573.91	24.535	150	1350	202.31	204.54	2.23	1.67
	MH48-5	0+000.000	188374.98	2974297.78			1350	199.4	202	2.60	
85	MH48-5-1	0+027.293	188402.13	2974300.57	27.293	150	1350	200	202.23	2.23	2.20
	MH59	0+000.000	188243.79	2974674.16			1500	199.54	204	4.46	
86	MH59-1	0+029.273	188271.92	2974682.26	29.273	200	1350	200	204	4.00	1.57
87	MH59-2	0+057.573	188299.03	2974690.38	28.300	200	1350	200.2	204	3.80	0.71
88	MH59-3	0+097.466	188288.36	2974728.82	39.893	200	1350	200.5	204	3.50	0.75
89	MH59-4	0+127.537	188277.84	2974756.99	30.070	200	1350	200.76	204	3.24	0.86
90	MH59-5	0+146.365	188270.97	2974774.52	18.828	200	1350	200.91	203.92	3.01	0.80
91	MH59-6	0+176.477	188259.64	2974802.42	30.113	200	1350	201.15	203.91	2.76	0.80
92	MH59-7	0+200.688	188251.47	2974825.21	24.210	200	1350	201.35	203.95	2.60	0.83
93	MH59-8	0+234.341	188239.31	2974856.59	33.654	200	1350	201.57	204.47	2.90	0.65
94	MH59-9	0+258.182	188228.58	2974877.88	23.841	200	1350	201.75	206.66	4.91	0.75
95	MH59-10	0+276.920	188217.39	2974892.91	18.738	150	1350	203.36	206.3	2.94	8.59
96	MH59-11	0+298.974	188203.33	2974909.9	22.053	150	1350	204.66	207.18	2.52	5.89
97	MH59-12	0+323.490	188188.34	2974929.3	24.517	150	1350	205.16	207.61	2.45	2.04
98	MH59-13	0+343.966	188175.39	2974945.16	20.475	150	1350	205.6	208	2.40	2.15
	MH59-2	0+000.000	188299.03	2974690.38			1350	200.2	204	3.80	
99	MH59-2-1	0+036.294	188306.72	2974654.91	36.294	150	1350	201.4	204	2.60	3.31
100	MH59-2-2	0+061.058	188311.69	2974630.65	24.764	150	1350	201.77	204.17	2.40	1.49
	MH59-4	0+000.000	188277.84	2974756.99		200	1350	200.76	204	3.24	
101	MH59-4-1	0+029.972	188304.96	2974769.75	29.972	150	1350	203.6	206.81	3.21	9.48

17 Annexure F-Waste Calculation

17.1 Ammochhu LAP Waste Calculation by Zones

17.1.1 CHARACTER ZONE 1

- Total area = 4428 m^2
- Average built-up area of each building = 414/420 m^2
- No of storied = 6
- Units in each floor = 2
- Average no of member in each household = 4
- No of people in zone 1 = (11 x 2 x 6 x 4) = 528 people
- Waste generated per capita per day = 0.25 kg/day
- Total waste generated per week = (0.25 x 528 x 7) = 924 kg/week
- Dry waste generated per week = 924/2 = 462 kg/week
- Wet waste generated per week = 462 kg/week

17.1.2 CHARACTER ZONE 2

- Total plot area = 32474 m^2
- No. of people in zone 2 = (77 x 2 x 6 x 4) = 3696 people
- Total waste generated per week = (0.25 x 3696 x 7) = 6468 kg/week
- Dry waste generated per week = 3234 kg/week
- Wet waste generated per week = 3234 kg/week

17.1.3 CHARACTER ZONE 3

- Total plot area = 70026 m^2
- No. of people in zone 3 = (166 x 2 x 6 x 4) = 7968 people
- Total waste generated per week = (0.25 x 7968 x 7) = 13944 kg/week
- Dry waste generated per week = 6972 kg/week
- Wet waste generated per week = 6972 kg/week

17.1.4 CHARACTER ZONE 4

- Total plot area = 80452 m^2
- No. of people in zone 4 = (192 x 2 x 6 x 4) = 9216 people
- Total waste generated per week = (0.25 x 9216 x 7) = 16128 kg/week
- Dry waste generated per week = 8064 kg/week
- Wet waste generated per week = 8064 kg/week

17.1.5 CHARACTER ZONE 5

- Total plot area = 13578 m^2
- No of people in zone 5 = (32 x 2 x 6 x 4) = 1536 people
- Total waste generated per week = (0.25 x 1536 x 7) = 2688 kg/week
- Dry waste generated per week = 1344 kg/week
- Wet waste generated per week = 1344 kg/week

17.1.6 CHARACTER ZONE 6

- Total plot area = 37914 m^2
- No of people in zone 6 = (90 x 2 x 6 x 4) = 4320 people

- Total waste generated per week = (0.25 x 4320 x 7) = 7560 kg/week
- Dry waste generated per week = 3780 kg/week
- Wet waste generated per week = 3780 kg/week