ROYAL GOVERNMENT OF CAMBODIA

MINISTRY OF INDUSTRY AND HANDICRAFTS
AND
MINISTRY OF PUBLIC WORKS AND TRANSPORT

TA 8982-CAM

PROVINCIAL WATER SUPPLY AND SANITATION

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Fraser Thomas Partners of New Zealand in association with CamConsult Ltd of Cambodia

Appendix 1
Preliminary Engineering Design Report – Battambang Water Supply Subproject

Appendix H1 Preliminary Engineering Design Report - Battambang Water Supply Subproject

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1 Introduction

1. Battambang is the provincial town of Battambang province and it is located in the North-West area of Cambodia, one of the major rice production areas. The town has a total area of 115.51km^2 and is divided into 10 Sangkats with 62 villages, see Figure 1. . Many economic development activities have increased in the past few years, including tourism activities, and Battambang has become the second largest town after Phnom Penh.

2. Battambang urban infrastructure, especially the water supply system is still limited. Town water supply covers only about 48% of the total households of the current service area. The recently completed WTP with a capacity of 22,000m^3/d, constructed with JICA funding, will likely reach its full capacity in 2019.

3. From the rapid assessment using the selected criteria, a weighting system approved by CDIA, ADB/AfD and MIH, and following site investigation and discussion with Battambang Waterworks in April 2016, Battambang town was ranked highly on the selection shortlist for water supply subprojects. Inclusion under PWSSP was agreed upon by all stakeholders during the Inception Workshop dated 28 April 2016.

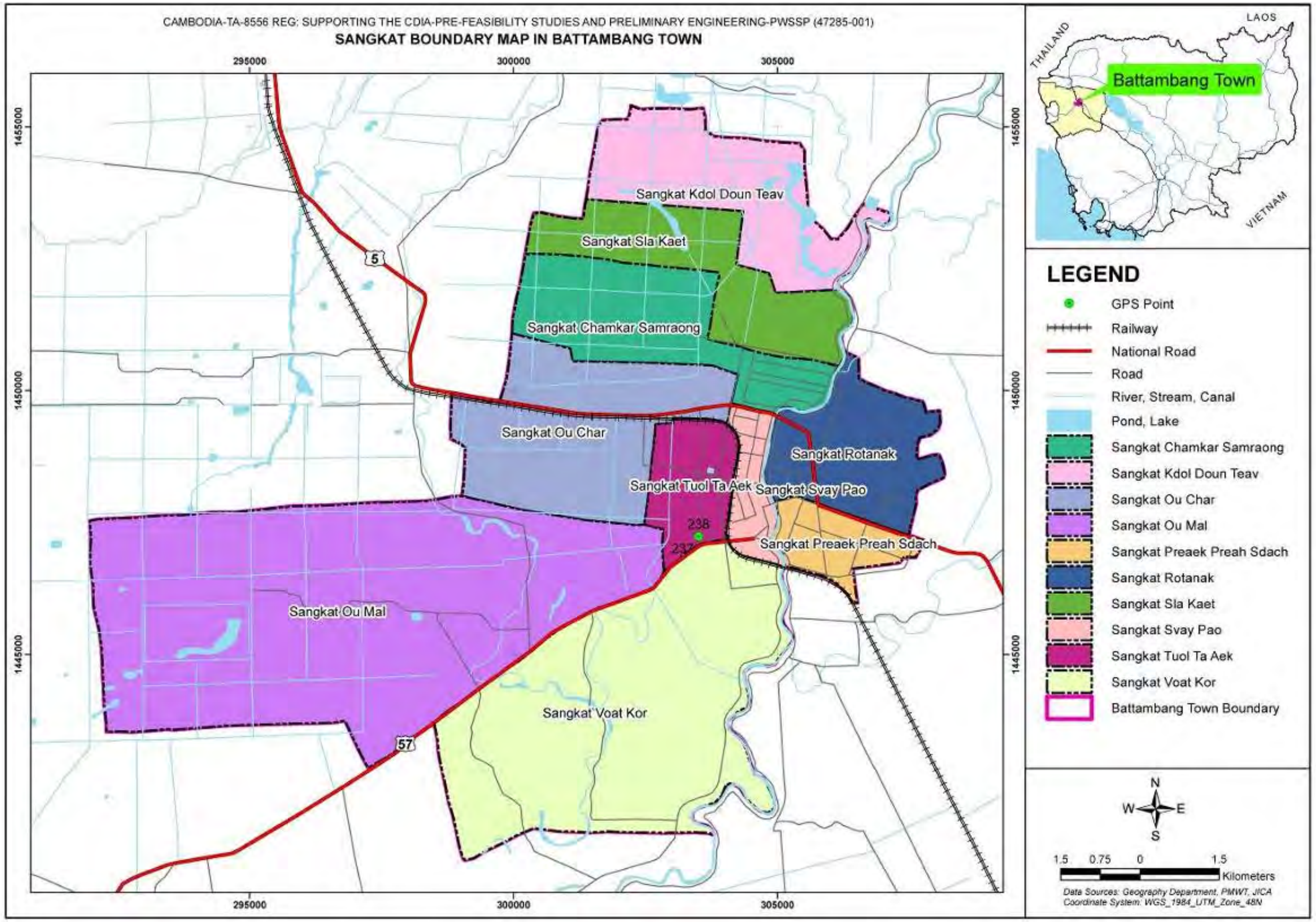
4. This subproject aims at filling the gap between the existing capacity, including the JICA Phase II service area, and the future demand within the proposed extended service areas.

5. The proposed subproject component is to supplement the existing system and to support RGC's policy on water supply in providing safe water to all people by 2025. To cater the needs for next 20 years (2020-2040), and to serve the proposed population (90-100% in Battambang town and 80-90% of communities nearby the town), the main components of the proposed subproject are:

   - A new WTP of at least 80,000 m^3/d for 2040 water demand and 240km of network (coverage area of 19,595 ha)
   - 1,721 additional connections in 2016 and 1,400 new connections annually thereafter from 2017
   - Served population (by 2040): approximately of 75,000 families or 360,000 people, out of which 51% are women.

6. The construction of the WTP would be phased to meet the target year 2025 water demand initially, with a design capacity of 50,000m^3/d. It is to be under a design and build (DB) contract (Yellow Fidic) and tendered on a performance basis within 1 year. The network is to be implemented under a build contract (Red Fidic) after detailed design which will be performed by PIAC/DPIH, and the works are to be implemented over 2.5 years from 2018 to 2020.

Figure 1. Battambang provincial town



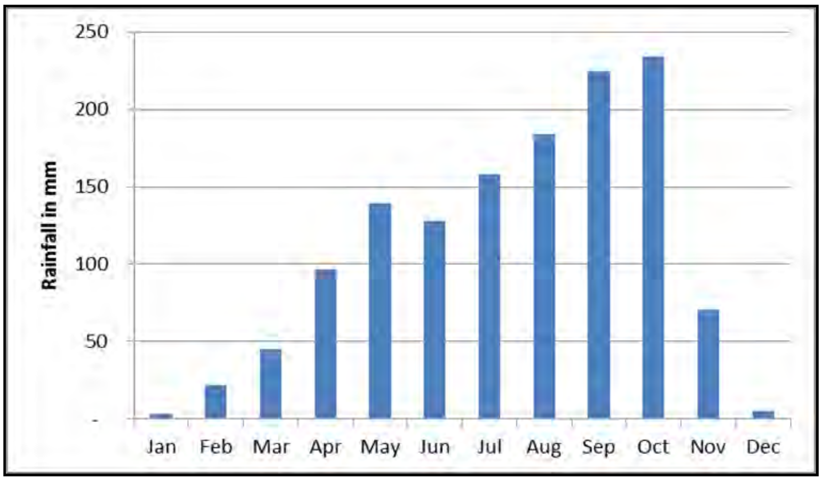
2 Current Situation

2.1 Climate

2.1.1 Rainfall

7. Cambodia's climate is tropical monsoonal divided into two seasons: rainy and dry. As shown on Figure 2. , the rainy season regularly starts in late May and finishes in late October, and the dry season covers November to April. Battambang is situated on the floodplain of Tonle Sap Lake, the main floodplain of Cambodia, and has a relatively low average annual precipitation of 1,311mm (1981-2011)[1].

Figure 2. Monthly rainfall, 1981-2011

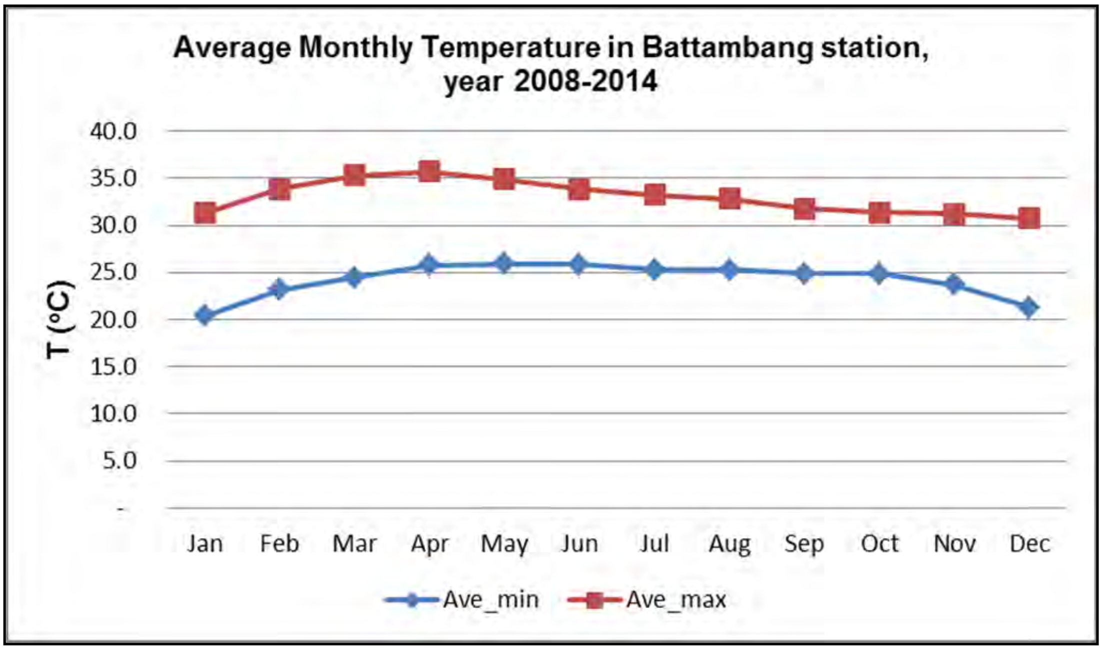


2.1.2 Temperature

8. According to the temperature data recorded from the Meteorology Station in Battambang from 2008 to 2014, the lowest monthly mean temperature was 20.4°C, the average annual minimum temperature was 24.2°C, and the average annual maximum temperature was 33.0°C. However, the hottest monthly temperature was 36.9°C occuring in April, and the coolest monthly was 18.0°C in January[2]. Figure 3. shows the average monthly temperature in Battambang from 2008 to 2014.

[1] Data from Department of Meteorology in Phnom Penh and the Office of Meteorology in Battambang.
[2] Data from Department of Meteorology in Phnom Penh and the Office of Meteorology in Battambang.

Figure 3. Average monthly temperature in Battambang (2008-2014)

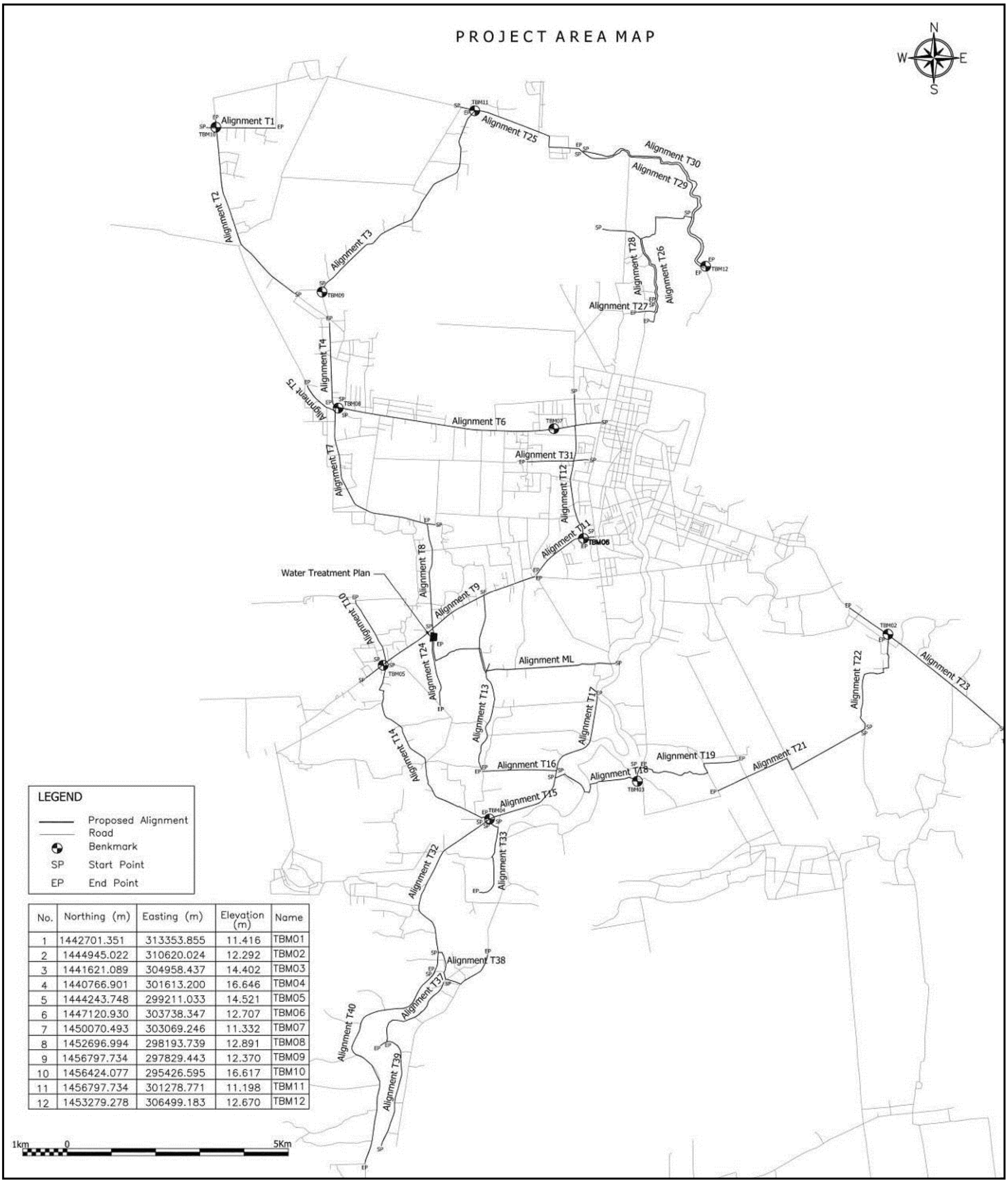


2.2 Geography and Topography

9. Battambang is the provincial town of Battambang province, in the North-West part of Cambodia. The town is situated on the flat terrain of the Tonle Sap floodplain.

10. There is no existing topographical data available in Battambang. The topographical survey performed under this CDIA PWSSP in August 2016, mainly along some of the main roads on the west-side of the town, reveals that the proposed WTP site is very flat with elevations between 12.50-13.76amsl, while the ground level along the national road #5 on the west side of the town has elevations between 11.00-13.0amsl. The ground level at the proposed intake is 8.56amsl, and the ground level along the proposed raw water transmission line to the WTP has elevations between 13.00-15.00amsl. The northern part of the town (alignment T32-T40) gently rises from 16.00m to 21.50amsl. Figure 4. shows the road alignments surveyed under this project.

Figure 4. Scope of topographical survey, August 2016



No.	Northing (m)	Easting (m)	Elevation (m)	Name
1	1442701.351	313353.855	11.416	TBM01
2	1444945.022	310620.024	12.292	TBM02
3	1441621.089	304958.437	14.402	TBM03
4	1440766.901	301613.200	16.646	TBM04
5	1444243.748	299211.033	14.521	TBM05
6	1447120.930	303738.347	12.707	TBM06
7	1450070.493	303069.246	11.332	TBM07
8	1452696.994	298193.739	12.891	TBM08
9	1456797.734	297829.443	12.370	TBM09
10	1456424.077	295426.595	16.617	TBM10
11	1456797.734	301278.771	11.198	TBM11
12	1453279.278	306499.183	12.670	TBM12

2.3 Existing Water Supply System

11. In 2015, Battambang town water supply covered about 48% of the total households in the current service area, with a total of 11,820 connections serving 67,374 persons or 31,490 families. The capacity of the Water Treatment Plant (WTP) has a current capacity of 33,520m^3/d, and the water source is the Sangker River.

12. The water supply system is operated 24 hours/day, as a public service operated by Battambang Waterworks, which is under the management of the Department of Industry and Handicraft (DIH) of Battambang Province.

13. The current tariff is Riel 1,500/m^3 (US$ 0.38/m^3), and the production cost was Riel 1,637.9/m^3 (US$ 0.41/m^3) and Riel 1,366.8/m^3 (US$ 0.34/m^3) in 2013 and 2014 respectively. See Table 1. for breakdowns of these production costs.

Table 1. Breakdowns of production costs in 2013 and 2014

Cost, Riel/m^3	2013	2014
Salary	178.17	179,09
Electricity	395.70	382.56
Diesel Oil	47.96	40.24
Chemical	259.68	160.64
O&M	129.45	182.42
Amortization	333.98	314.42
Administration	194.46	77.35
Others (including Tax)	98.43	30.1
Total	**1637.9**	**1,366.8**

14. There are two separate WTPs currently operating: the original city centre WTP from colonial times rehabilitated by the ADB 1725 project, and a new northern WTP constructed under the JICA II project (commissioned in 2016). The new 22,000m^3/d JICA II WTP pumps water directly into the network. The original central 11,520m^3/d WTP pumps to two elevated tanks inside the WTP grounds before being fed by gravity to the network. There are currently no elevated service reservoirs around the distribution network.

15. There are three completed projects (ADB1725-TPI, JICA Phase I, and JICA Phase II) that have improved the Battambang water supply over the last 15 years. These projects built on the original water supply scheme from the colonial times have been rehabilitated in 1992 by an EU funded project. The ADB1725 Project (2003-06) was the first project in recent times to rehabilitate and extend the existing WTP. The second project was JICA Phase 1, which focused on installing pipelines. Facilities constructed under these projects are summarized in Table 2. while more details are given below on JICA Phase 2 Project.

Table 2. Existing facilities completed under ADB TPI and JICA Phase I

Facilities	Capacity/length	Remarks
WTP (conventional)	11,520m^3/d	Rehabilitation of the existing by ADB1725-TPI. Water towers situated inside WTP grounds. The 864m^3 reservoir was constructed under the EU/SAWA project (1992) whilst the 450m^3 reservoir is from colonial times.
Water tower (Sangker river source)	- No.1: 450m^3; 27m height - No.2: 250m^3; 25m height	
Ground reservoir	- No.1: 864m^3 - No.2: 450m^3	
Pipe network	223km	Existing
	45.6km	Network expansion by ADB1725-TPI
	41km	Network expansion by JICA Phase I

16. In July 2016, a conventional WTP (JICA Phase II) has been commissioned, with a capacity of 22,000m^3/d. JICA Phase II project also included one intake with a capacity of 24,000m^3/d, a 4.4km raw water transmission main, and 65.5km of distribution pipeline. The components and specifications of the water intake and the WTP are described in the table below.

Table 3. Existing facilities completed under JICA Phase II

Facility			Component
Intake Facility	Intake Shaft	Main Chamber	Reinforced Concrete Structure Rectangular Shape Depth 13.90 m (Depth at HWL 13.20 m)
		Operation Room	Reinforced Concrete Structure Equipment: Power Receiving Panel, Operating Panel, Switchboard, Secondary Equipment Panel, Circumference Plumbing of Pump, Overhead Crane (3t)
	Intake Pump Facility	Intake Pump	Vertical Shaft Type Mixed Flow Pump 3 sets (Ordinary Use 2 sets, Spare 1 set) Q=8.40 m^3/min
		Generator	Reinforced Concrete Structure Generator: 260 kV (Soundproofing Type)
Raw Water Transmission	Transmission Main		DIPΦ600, L=4.4km
Water Treatment Plant	Receiving Well		Reinforced Concrete Structure Volume and Detention Time: V=46.7 m3, T=2.8 min in dry season (T≥1.5 min)
	Rapid Mixing Tank		Reinforced Concrete Structure Gravitational force mixing using a weir Volume and Detention Time: V=22.9 m3, T=1.4 min (1 <T< 5 min)
	Flocculation Basin		Reinforced Concrete Structure Slow Mixing Method: Vertical channel bands flocculator Quantity: 4
	Sedimentation Basin		Reinforced Concrete Structure Horizontal-Flow Sedimentation Basin Supernatant Collecting System: Collecting Trough + Submerged Orifice Quantity: 4 Surface Loading: Q/A=20.0 mm/min (15-30 mm/min) Mean Velocity: V=0.14 m/min (below 0.40 m/min)
	Rapid Sand Filter		Reinforced Concrete Structure Quantity: 6 Underdrain System: Porous Filter Bed Method Filtration Rate: V=126.6 m/day (120-150 m/day) Flow Control: Lower Part Control Method Backwash Method: Simultaneous Backwash Method by Air and Water
	Clearwater Reservoir		Reinforced Concrete Structure using Flat Slab Structure Quantity: 2 Volume: 6,000m^3 (supplements original 2 tanks of 864m^3 and 450m^3) Detention Time: T=6.5 hours
	Wastewater Basin		Reinforced Concrete Structure Quantity: 2 Effective Volume: V=230 m3 (115 m3 x 2)
	Sludge Drying Bed (Lagoon)		Reinforced Concrete Structure Quantity: 4
	Chemical Feeding Facility		Alum, Lime Chlorine
	Emergency Generator		450 KVA (Soundproof type, Equipped with Internal Water Tank)
	Administration Building		Reinforced Concrete Structure, Three Stories Building

17. Following the commissioning of the JICA Phase II WTP, the total combined capacity of the two WTP's in Battambang is now 33,520m^3/d and total distribution network is 374km with a coverage area of 5,603ha including some parts of Sangker and Thmakol Districts. Figure 5.

shows the existing water supply facilities and coverage area, and Figure 6. includes the existing distribution network.

18. Battambang Waterworks now plans to increase connection of 1,400 households per year[3]. However, the whole WTP capacity can only provide sufficient water to its existing and JICA II service area up to 2019[4]. Thus, additional capacity is required to provide water for the existing coverage area from 2019 onward, and for the new areas to be covered by Battambang Waterworks proposed under this project.

[3] Following JICA 2 project, which increased the service area. The previous target for the original service area (before JICA II) was to add 600-700 new connections per year.
[4] "Preparatory Survey on the Project on Additional New Water Treatment Plants for Kampong Cham and Battambang Waterworks in the Kingdom of Cambodia, March 2013, JICA"

Figure 5. Coverage area of the existing and JICA Phase II project

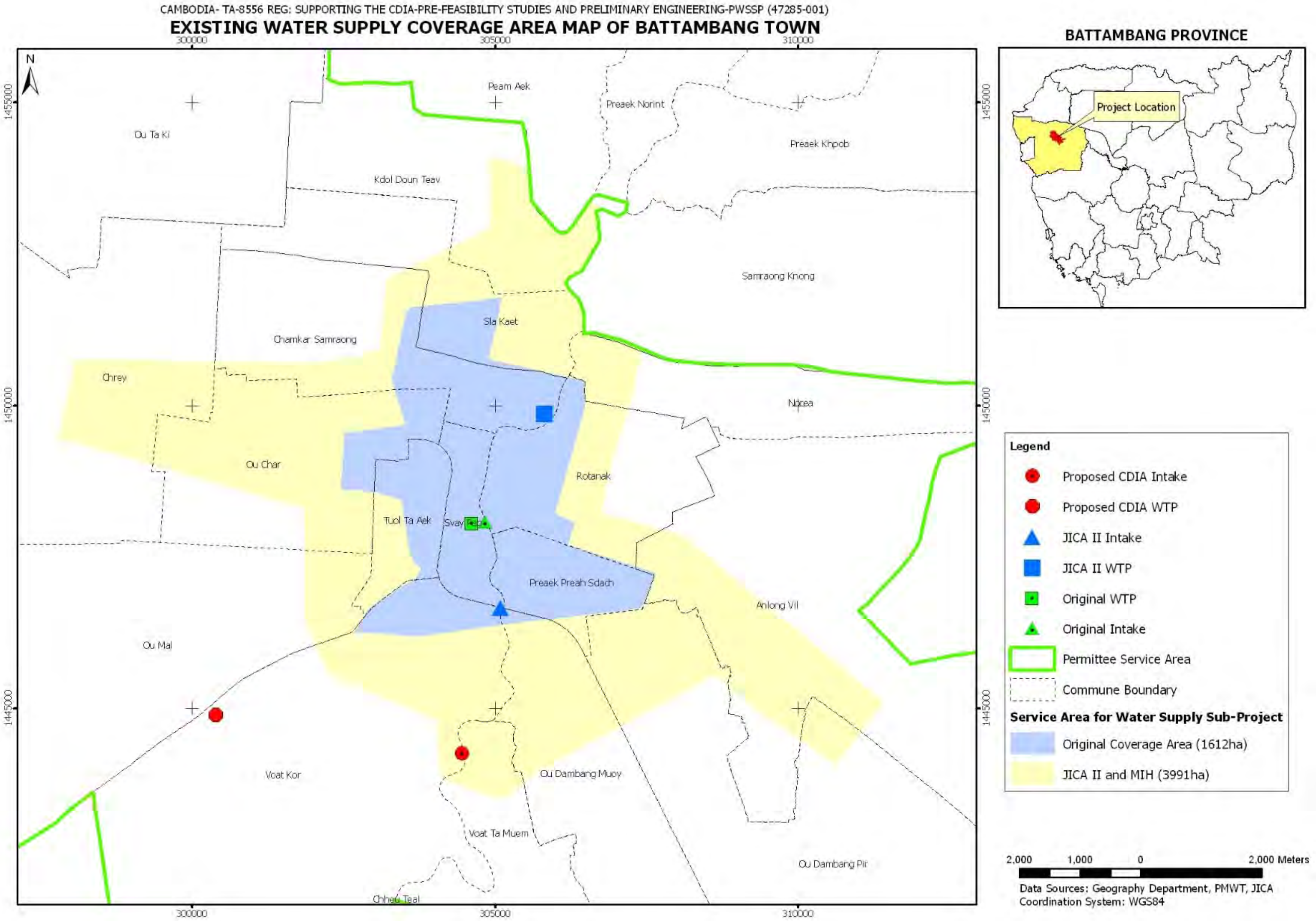
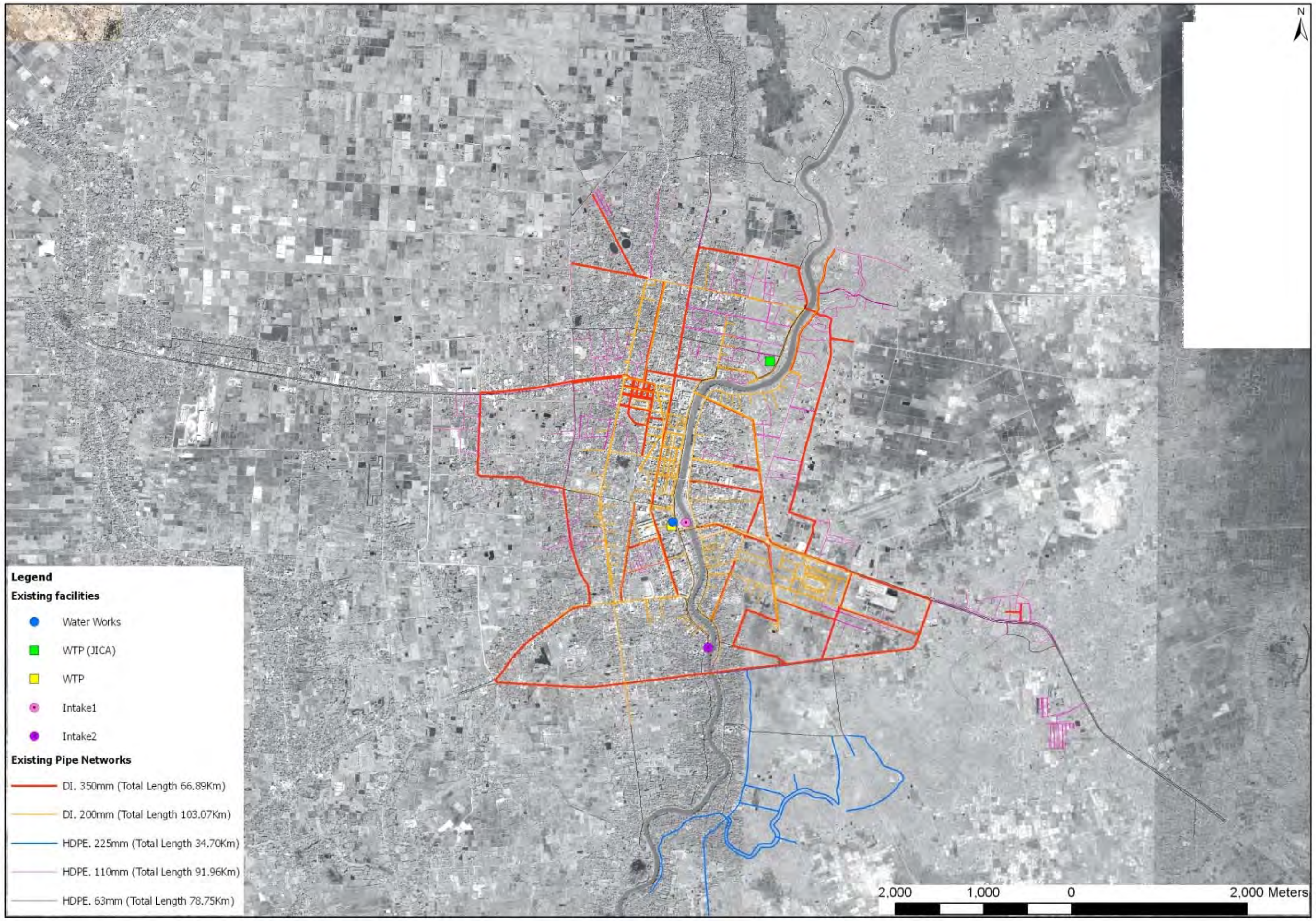


Figure 6. Present water supply distribution network

Source: Battambang Waterworks, April 2016



2.4 Commercial/Tourism Facilities

19. The number of commercial facilities and their total monthly water consumption are summarised in Table 4.

Table 4. Water consumption of commercial/tourism facilities in Battambang

Facility	Unit	2010	2011	2012	2013	2014	2015
Commercial/ tourism facilities	Base	181	185	196	211	223	314
Average daily water consumption	m^3/d	1,240	1,492	1,525	1,561	1,957	2,421

Source: Data obtained from Battambang Waterworks, May 2016

2.5 Public Buildings

20. As the provincial town of Battambang province, many public buildings including schools, hospitals, pagoda/ church/ mosques, and government offices are present in the city. The public buildings served by the town water supply are listed in Table 5.

Table 5. Public buildings served by the Battambang Waterworks

Building	Unit	2010	2011	2012	2013	2014	2015
Institution/ public building	Base	74	74	74	74	74	74
Average daily water consumption	m^3/d	964	975	1,017	1,140	1,187	1,210

Source: Data obtained from Battambang Waterworks, May 2016

2.6 Industries and Handicraft

21. The only large industry in Battambang is the power plant. There are many small industries and handicrafts[5], some of which use their own water supply system, and some of which use and further treat the town water supply. Table 6. indicates the industrial and handicraft manufacturers inside the urban area.

[5] The term "handicrafts" as used by the GoC does not relate to tourist souvenirs etc. It refers to smaller family type businesses – ice making, fish sauce manufacturing, meat processing etc.

Table 6. Industries and handicraft inside current service area

No	Description	Location			Production	Production Per Year		
		Village	Commune	District		Unit	Amount	m^3/ month
1	Rice Mill	Chamkar samraong 2	Chamkar samraong	Battambang	Rice	Ton	480	
2	Pure Water	Chamkar samraong 2	Chamkar samraong	Battambang	Pure Water	Liter	345,60	288.0
3	Wine (Grape)	Chamkar samraong 2	Chamkar samraong	Battambang	Wine	Liter	10,000	8.33
4	Rice Mill	Haisan	Chrey	Thmar Kol	Rice	Ton	1,440	
5	Rice Mill	Kor Ko	Chrey	Thmar Kol	Rice	Ton	1,440	
6	Rice Mill	Kor Ko	Chrey	Thmar Kol	Rice	Ton	4,200	
7	Pure Water	Outakorm	Tuolta ek	Battambang	Pure Water	Liter	144,00	120.0
8	Plastic Bottle	Dongkaoteab	Tuolta ek	Battambang	Blow Bottle	Bottle	1,800,0	
9	Ice-Cream	Outakorm1	Tuolta ek	Battambang	Ice-cream	Piece	200,00	
10	Ice Making	Outakorm2	Tuolta ek	Battambang	Ice	Ton	240	
11	Pure Water	Outakorm2	Tuolta ek	Battambang	Pure Water	Liter	234,00	195.0
12	Pure Water	Outakorm1	Tuolta ek	Battambang	Pure Water	Liter	1,080,0	900.0
13	Pure Water	Dongkaoteab	Tuolta ek	Battambang	Pure Water	Liter	1,116,0	930.0
14	Balm Ointment	Outakorm2	Tuolta ek	Battambang	Balm Ointment	Bottle	3,650	
15	Rice Mill	Chreabkrasaing	Vatkor	Battambang	Rice	Ton	240	
16	Rice Mill	Balang	Vatkor	Battambang	Rice	Ton	672	
17	Rice Mill	Chreabkrasaing	Vatkor	Battambang	Rice	Ton	750	
18	Rice Mill	Chreabkrasaing	Vatkor	Battambang	Rice	Ton	1,152	
19	Rice Mill	Chreabkrasaing	Vatkor	Battambang	Rice	Ton	1,440	
20	Soy Sause Fish Sauce	Ouchar	Ouchar	Battambang	Soy Sauce Fish sauce	Dozen	3,600	
21	Rice Mill	Anhchanh	Ouchar	Battambang	Rice	Ton	600	
22	Ice	Kabkorthmey	Ouchar	Battambang	Ice	Ton	525	
23	Orange Juice-Soda	Ouchar	Ouchar	Battambang	Orange Juice Soda	Liter	1,200	1.00
24	Pure Water	Anhchanh	Ouchar	Battambang	Pure Water	Liter	59,760	49.80
25	Pure Water	Andongchenh	Ouchar	Battambang	Pure Water	Liter	216,00	180.0
26	Concrete	Ang	Ouchar	Battambang	Concrete	m^3	19,800	
27	Meatball-Pasté	Ouchar	Ouchar	Battambang	Meatball-Pasté	Ton	7	
28	Rice Mill	Ang	Ouchar	Battambang	Rice	Ton	240	
29	Rice Mill	Andongchenh	Ouchar	Battambang	Rice	Ton	540	
30	Rice Mill	Ouchar	Ouchar	Battambang	Rice	Ton	2,400	
31	Rice Mill	Konsek	Ouchar	Battambang	Rice	Ton	108,00	
32	Wine (Grape)	Ouchar	Ouchar	Battambang	Wine	Liter	5,000	4.17
33	Rice Mill	Ou kchay	OuDambang	Sangker	Rice	Ton	300	
34	Rice Mill	Ou kchay	Ou Dambang1	Sangker	Rice	Ton	900	
35	Rice Mill	Dambuk Kpuos	Ou Dambang2	Sangker	Rice	Ton	108	
36	Rice Mill	Svay Thom	Ou Dambang2	Sangker	Rice	Ton	600	
37	Rice Mill	Dambuk Kpuos	Ou Dambang2	Sangker	Rice	Ton	648	

Source: Data obtained from DIH of Battambang, August 2016

2.7 Existing Water Source

22. The Sangker River is one of the major tributaries of the Tonle Sap River, it flows through the Battambang town and then into the northwestern part of the Tonle Sap Lake. The catchment area of the Sangker River is 6,053km^2. This river is proposed as the water source for the new intake. Monthly average flow is as high as 154.42m^3/s in October and as low as 2.44m^3/s in February[6]. It is adequate to supply raw water to the existing and the proposed WTPs with flows of 73,616m^3/d by 2025 and 103,616m^3/d by 2040[7].

23. The annual minimum flow during the drought for a 10- year return period is considered to be 0.97m^3/s[8]. From records 1997 to 2010, the most severe drought occurred in 2005, with no river flow for about 6 days and an annual minimum flow of 0.87m^3/s. Supplementary raw water from a surface reservoir (Chrey Reservoir), 8 km northwest of the proposed WTP (shown in Figure 9.) is considered by the Battambang Waterworks to be the best option to supply water to the existing and the proposed water supply systems to meet the water demand during future drought events.

24. It shall be noted that in addition to this back-up reservoir, a storage dam proposed by the Sala Ta Orn Development Project under Korean funding assistance, located about 3km downstream from the existing WW intake (and 5km downstream of the proposed intake location), will keep 147 million m^3 water in the river with a minimum depth of 6m. This is scheduled to be completed in 2019. This in-stream reservoir will provide 73 and 74 million m^3 for irrigation and river maintenance, respectively.

[6] During the period 1997-2010

[7] Preparatory Survey on the Project on Additional New Water Treatment Plants for Kampong Cham and Battambang Waterworks in the Kingdom of Cambodia, March 2013, JICA.

[8] Preparatory Survey on the Project on Additional New Water Treatment Plants for Kampong Cham and Battambang Waterworks in the Kingdom of Cambodia, March 2013, JICA

3 City development and population projections

3.1 Urban Plan & Land Use

25. The proposed water supply coverage area is designed to follow the existing “Development Vision 2020”, part of the Provincial Masterplan.

26. The current city physical planning structure for Battambang was established in December 2008 by Sub-Decree No 223. It consists of 10 Sangkats west of the river with 2 other Sangkats (Prek Preah Sdecdech and Rattanak) located on the East side of the Sangker River, as shown in Figure 7.

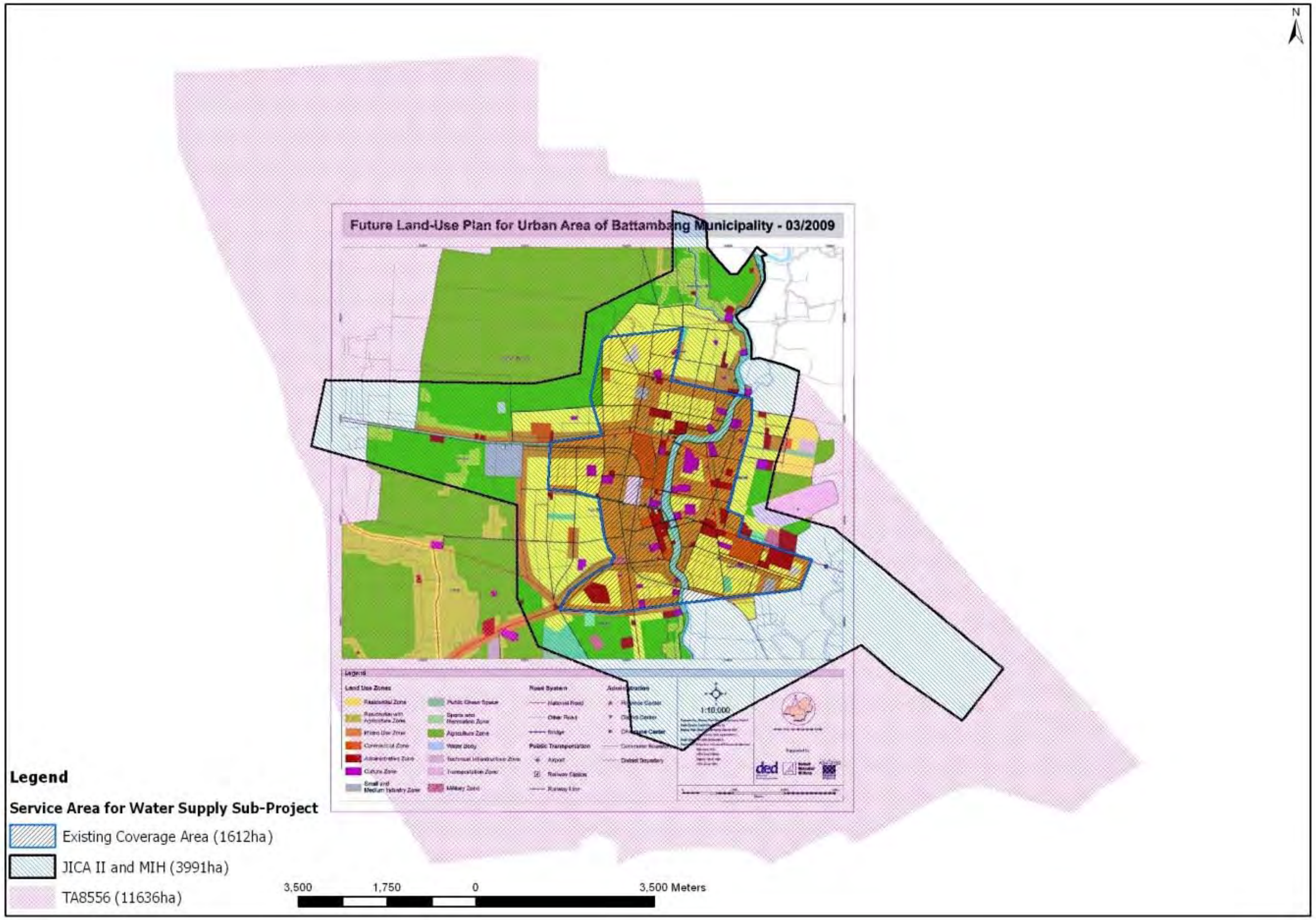
27. In the Development Vision 2020, prepared by the Master Planning Team of Battambang Municipality in March 2009, the objective for good governance, and a green and healthy town is clearly indicated. The specific goals and objectives, which are also supported by CDIA include;

- Strengthening the capacity of public servants and administrative services;
- Setting up socially acceptable solutions for informal settlements;
- Providing sufficient technical infrastructure for tourists (water, waste management);
- Improve living conditions of local people and public sanitation;
- Setting up and maintain appropriate supply and service systems (drinking water, wastewater treatment, drainage, solid waste management);
- Reconstructing the existing water treatment plant to its full capacity;
- Reserve a suitable place for a new water treatment plant on the east bank of the town;
- Setting up a joint network for better coordination of the infrastructure systems;
- Develop Battambang Municipality as a Green and Healthy Town and promote good environmental conditions in towns and villages.

28. Accordingly, the existing water supply system is to be expanded to fulfill the future needs of Battambang. The proposed service area is superimposed on the Battambang Land Use Map and shown in figure 7 below. In the 7 years since this map was produced, residential land has spilled over into land formerly identified as agricultural.

29. According to the Master Plan on Land Use of Battambang, prepared by National Land Management Urban Planning, in March 2009, the planned development of Battambang is towards expansion of the urban areas on the western and eastern sides of the Sangker River.

Figure 7. Battambang Urban Land Use in the Future and project coverage area



3.2 Coverage Area Extension

30. Based on the results of field investigation and consultation with Battambang Waterworks, and in accordance with the future development plan, the coverage area can be divided into three target periods: Year 2015 (Existing system), Year 2019 (JICA Phase II), and Year 2040 (CDIA), as shown in Figure 8. below. The central blue area (1,612 ha) shows the present coverage area as served by the existing system funded by ADB and JICA Phase I; the yellow area (3,991 ha) is the coverage area funded by JICA Phase II, and the outer pink area is the proposed extended coverage area (11,636 ha) under this current project funded by CDIA/ADB. The current and proposed coverage areas, by sangkat/commune are described in Table 7.

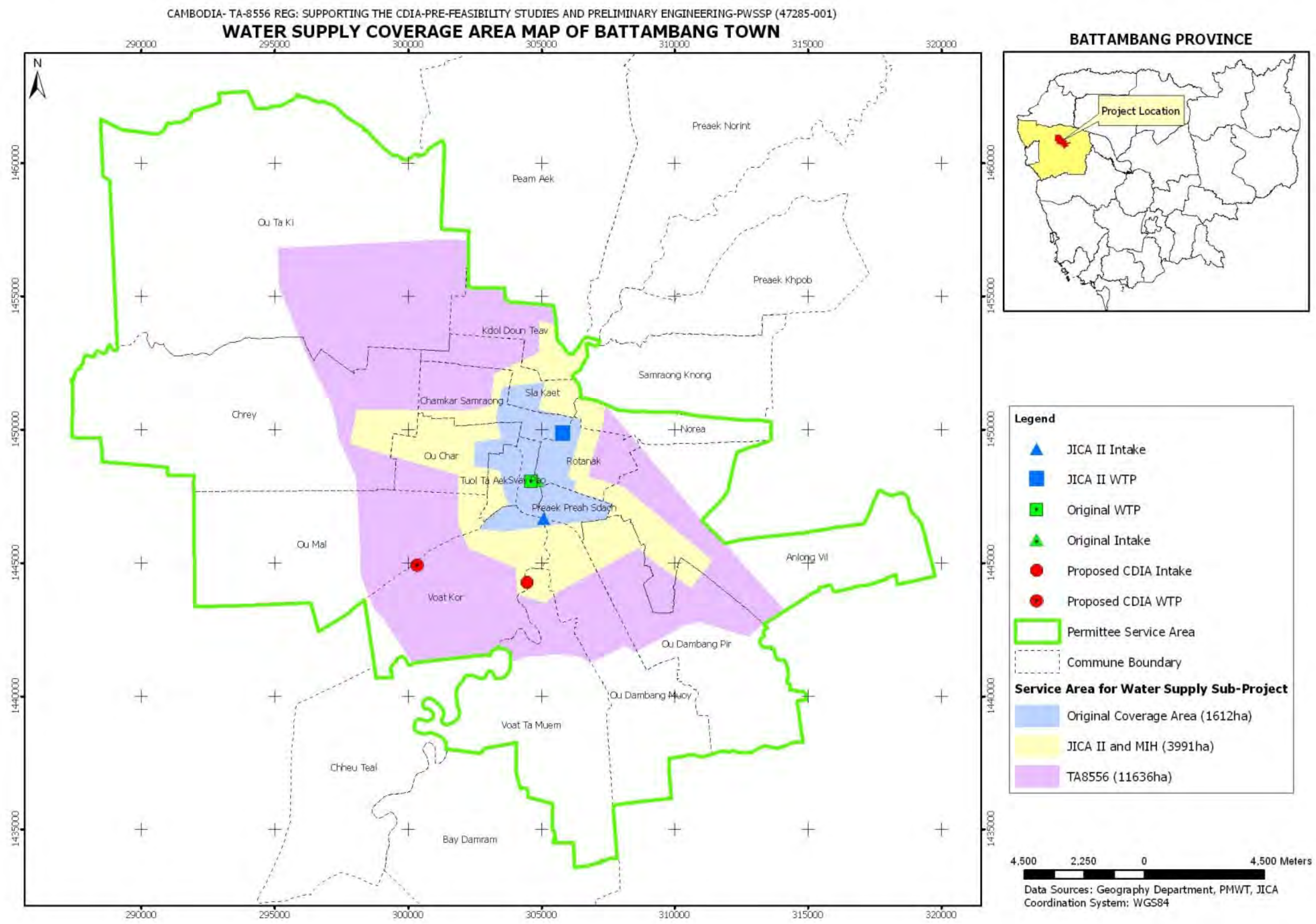
31. The proposed service expansion area (triple in size from the existing service area) is within the approved service boundary signed by the DIH director, and agreed by the Battambang Provincial Governor[9]. The permitted boundary by DIH is shown as green line on Figure 8.

Table 7. Coverage area by Sangkat/commune

City/ district	Commune	Area (ha)	Service Area							
			TA8556-CDIA Coverage Area		JICA II and MIH Coverage Area		Existing Coverage Area		Total Coverage Area	
			(ha)	(%)	(ha)	(%)	(ha)	(%)	(ha)	(%)
Battambang	Chamkar Samraong	925.0	470.6	50.88	194.96	21.08	259.36	28.04	925	100
	Kdol Doun Teav	906.4	604.2	66.66	288.18	31.79	0	0.00	892.43	98.45
	Ou Char	1020.8	226.8	22.22	651.29	63.80	142.78	13.99	1020.87	100
	Ou Mal	3942.2	1050.8	26.66	135.8	3.44	0.05	0.00	1186.72	30.10
	Preaek Preah Sdach	294.4	0	0.00	21.36	7.25	273.09	92.75	294.45	100
	Rotanak	747.8	261.5	34.97	211.33	28.26	271.97	36.37	744.81	99.60
	Sla Kaet	713.8	350.7	49.13	244.42	34.24	118.71	16.63	713.83	100
	Svay Pao	217.1	0	0.00	0	0.00	217.09	100.00	217.09	100
	Tuol Ta Aek	275.51	0	0.00	107.12	38.88	168.39	61.12	275.51	100
	Voat Kor	2648.5	1872.9	70.72	398.66	15.05	147.29	5.56	2418.89	91.33
	Sub-total	**11,691.8**	**4,837.**		**2,253.1**		**1,598.7**		**8,689.6**	
Sangker	Anlong Vil	3903.0	1042.1	26.70	499.23	12.79	1.51	0.04	1542.87	39.53
	Norea	723.4	62.5	8.65	106.1	14.67	0.03	0.00	168.7	23.32
	Ou Dambang Muoy	2541.6	544.2	21.41	333.4	13.12	11.49	0.45	889.09	34.98
	Ou Dambang Pir	3126.1	982.9	31.44	369.94	11.83	0	0.00	1352.9	43.28
	Voat Ta Muem	3566.1	408.2	11.45	113.45	3.18	0	0.00	521.7	14.63
	Sub-total	**13,860.4**	**3,040.1**		**1,422.12**		**13.03**		**4,475.26**	
Thmakoul	Chrey	6,189.2	1089.9	17.61	316.03	5.11	0	0.00	1406	22.72
	Ou Ta Ki	10,919.8	2668.5	24.44	0	0.00	0	0.00	2668.53	24.44
	Sub-total	17,109.1	3,758.5		316.03		-		4,074.53	
	Total		**11,636.3**		**3,991.27**		**1,611.76**		**17,239.39**	

[9] see supporting documents attached in Annex 1

Figure 8. Coverage area of Battambang Water Supply System



3.3 Population Projection

32. Recently, Battambang has seen many development activities such as urban/residential development, tourism and commercial development, and other development activities. However, the permanent population has only slightly increased, due to migration to Thailand for higher salaries. The population growth rate in each Sangkat and commune of Battambang, Thmakoul, and Sangker district is described in 0The average Growth Rate (GR) over 6 years, shown in the last column of this table has been used to calculate the population projections for water demand purposes. In Table 8, the “adopted GR” differs to the calculated “Average GR” for 5 communes due to trend corrections made for some anomalous year-to-year growth rates from which the average figures are derived.

33. JICA Phase II covered some areas outside of the 10 sangkhats of the main town area, to the south east and north west. There are some remaining part and whole communes in the adjacent districts still without main water supply, which shall be covered by this proposal.

34. This subproject intends to serve people in the extended coverage area up to 85-100% of the population by 2025 and 95-100% by 2040 with more communities nearby the town boundary to be included for the Year 2025. The service area extension was approved by the government (PDIH/MIH) and further changes are not likely to occur unless it is approved by PDIH and MIH. The population projection is calculated with the formula below, and is commonly used for population projections in SEA and elsewhere. The results of the projection up to 2040 are detailed in Table 9.

Formula: $\boldsymbol{P_t = P_0 * (1 + e)^t}$

Where P_t - *Future population at projection time*

P_o - *Present population*

e - Population grow rate (%)

t - The period of projection

Table 8. Population Growth Rate (GR) by Sangkat and commune

Town/ District	Commune	Population							Ave.	Adopted GR %
		2009	2010	2011	2012	2013	2014	2015	GR %	
Battambang	Chamkar Samraong	16356	17179	17722	17799	18254	18512	18765	2.33	2.33
	Kdol Doun Teav	9072	9359	9403	9357	9716	9876	10059	1.75	1.75
	Ou Char	17049	16993	17836	17971	17840	17865	17918	0.85	0.85
	Ou Mal	10523	10720	10877	10971	11140	11430	11393	1.34	1.34
	Preaek Preah Sdach	12549	13277	13987	13543	13842	13836	13264	1.00	1.00
	Rotanak	14648	14569	15335	15290	15194	15218	15411	0.87	0.87
	Sla Kaet	6897	6474	7003	7476	7781	8152	7583	1.78	1.78
	Svay Pao	23521	23532	21064	21033	21338	21588	21588	(1.33)	0.20
	Tuol Ta Aek	17343	18276	18676	18530	19144	19173	19404	1.91	1.91
	Voat Kor	16095	15845	15579	15692	16462	18077	17145	1.18	1.18
	Sub-Total	***144053***	***146224***	***147482***	***147662***	***150711***	***153727***	***152530***		1.32
Sangker	Anlong Vil	18509	19164	21486	20745	19872	19301	19301	0.85	1.02
	Norea	5700	5986	6114	5620	5899	5933	5933	0.77	0.92
	Ou Dambang Mouy	13839	14023	14572	14283	14856	15202	15202	1.60	1.60
	Ou Dambang Pir	12323	12763	13015	12849	12914	13632	13632	1.72	1.72
	Voat Ta Moem	14614	14808	14835	14890	16680	16519	16519	2.16	2.16
	Sub-Total	***64985***	***66744***	***70022***	***68387***	***70221***	***70587***	***70587***		
Thma Koul	Chrey	15259	14884	15729	16145	16563	16884	16884	1.73	2.08
	Ou Ta ki	16007	16158	16938	16782	17314	17067	17607	1.63	1.94
	Sub-Total	***31266***	***31042***	***32667***	***32927***	***33877***	***33951***	***34491***		
	Total	**240304**	**244010**	**250171**	**248976**	**254809**	**258265**	**257608**		

Source: Commune Database (CDB), Ministry of Planning

Table 9. Population projection

Town/ District	Sangkat/ Commune name	People in Baseline	Grow rate (1)	Population projection (persons)				
		2015		2020	2025	2030	2035	2040
BTB city	Chamkar Samraong	18,765	2.33	21,055	23,625	26,509	29,745	33,375
	Kdol Doun Teav	10,059	1.75	10,971	11,965	13,049	14,231	15,521
	Ou Char	17,918	0.85	18,693	19,501	20,344	21,223	22,140
	Au Mal	11,393	1.34	12,177	13,015	13,911	14,868	15,891
	Preaek Preah Sdach	13,264	1.00	13,941	14,652	15,399	16,185	17,010
	Rotanak	15,411	0.87	16,093	16,805	17,549	18,326	19,137
	Sla Kaet	7,583	1.78	8,282	9,046	9,880	10,792	11,787
	Svay Pao	21,588	0.2	21,805	22,024	22,245	22,468	22,694
	Tuol Ta Aek	19,404	1.91	21,329	23,445	25,772	28,329	31,139
	Voat Kor	17,145	1.18	18,181	19,279	20,444	21,679	22,988
	Sub-total	***152,530***	***1.32***	***162,526***	***173,357***	***185,101***	***197,845***	***211,684***
Sangker	Anlong Vil	19,301	1.02	20,311	21,373	22,491	23,668	24,906
	Norea	5,933	0.92	6,212	6,504	6,810	7,130	7,466
	Ou Dambang 1	15,202	1.60	16,458	17,818	19,290	20,884	22,609
	Ou Dambang 2	13,632	1.72	14,847	16,171	17,612	19,182	20,893
	Voat Ta Muem	16,519	2.16	18,378	20,447	22,748	25,309	28,158
	Sub-total	***70,587***		***76,206***	***82,313***	***88,952***	***96,173***	***104,030***
Thma Kuol	Chrey	16,884	2.08	18,713	20,740	22,987	25,477	28,237
	Ou Ta Ki	17,607	1.94	19,385	21,342	23,497	25,869	28,481
	Sub-total	***34,491***		***38,098***	***42,082***	***46,484***	***51,346***	***56,718***
	Total:	**257,608**		**276,830**	**297,752**	**320,537**	**345,364**	**372,432**

Note: (i) Adopted growth rate for future projection

4 Served Population and Other Facilities

4.1 Served Population

35. From the proposed coverage service area of Battambang Waterworks (Figure 8. above), the served population for different target years is described in the Table 10. , which indicates that two districts with 15 communes are currently served by the existing system, 14 of 15 communes are partially covered by the system and only one commune (Svay Pao) is close to 100% coverage. The overall population coverage is currently about 42% plus a small percentage on nearby communities in Sangker District.

36. According to the national policy on urban water supply, the RGC will provide clean and safe water to all Cambodian people. Due to the geographical spread of the community and the settlement pattern, it is assumed that 85-100% of people will be served by 2025 and 90-100% of people will be served by 2040. The remaining people are proposed to be served by other source (rural water supply system such as groundwater or rainwater collection).

Table 10. Served population

City/District	Sangkat/ Commune name	Population 2015		Served Population, 2015				Total Population 2025	Served Population in 2025		Total Pupolation 2040	Served Population in 2040	
		Family	person	Family	Family size	person	%	Persons	%	Persons	Persons	%	Persons
BTB	Chamkar Samraong	3,455	18,765	1,278	5.43	6,941	36.99	23,625	100	23,625	33,375	100	33,375
	Kdol Doun Teav	2,150	10,059	0	4.68	-		11,965	100	11,965	15,521	100	15,521
	Ou Char	2,874	17,918	1,046	6.23	6,521	36.40	19,501	100	19,501	22,140	100	22,140
	Ou Mal	2,378	11,393	40	4.79	192	1.68	13,015	95	12,364	15,891	100	15,891
	Preaek Preah Sdach	2,531	13,264	1,433	5.24	7,510	56.62	14,652	95	13,919	17,010	100	17,010
	Rotanak	2,710	15,411	1,470	5.69	8,359	54.24	16,805	95	15,965	19,137	100	19,137
	Sla Kaet	1,404	7,583	307	5.40	1,658	21.87	9,046	95	8,594	11,787	100	11,787
	Svay Pao	3,286	21,588	3,276	6.57	21,522	99.70	22,024	100	22,024	22,694	100	22,694
	Tuol Ta Aek	4,107	19,404	2,109	4.72	9,964	51.35	23,445	100	23,445	31,139	100	31,139
	Voat Kor	3,114	17,145	346	5.51	1,905	11.11	19,279	95	18,315	22,988	100	22,988
	Sub-total	***28,009***	***152,530***	***11,305***		***64,573***		***173,357***		***169,717***	***211,684***		***211,684***
Sangker	Anlong Vil	4,059	19,301	469	4.76	2,230	11.55	21,373	85	18,167	24,906	90	22,415
	Norea	1,112	5,933	555	5.34	2,961	49.91	6,504	85	5,529	7,466	90	6,719
	Ou Dambang 1	2,688	15,202	24	5.66	136	0.89	17,818	85	15,145	22,609	90	20,348
	Ou Dambang 2	2,774	13,632	6	4.91	29	0.22	16,171	85	13,745	20,893	90	18,803
	Voat Ta Muem	3,506	16,519	2	4.71	9	0.06	20,447	85	17,380	28,158	90	25,342
	Sub-total	***14,139***	***70,587***	***1,056***		***5,366***		***82,313***		***69,966***	***104,030***		***93,627***
Thma Kuol	Chrey	3,247	16,884	0		0	0	20,740	85	17,629	28,237	95	26,825
	Ou Ta Ki	3,668	17,607	0		0	0	21,342	85	18,141	28,481	95	27,057
	Sub-total	***6,915***	***34,491***	***0***		***0***	***0***	***42,082***		***35,770***	***56,718***		***53,882***
	Total:	**49,063**	**257,608**	**12,361**		**69,939**		**297,752**		**275,453**	**372,432**		**359,193**

4.2 Served Commercial/ Tourism Facilities

37. Battambang has a high potential for further development in tourism and other sectors. Currently this town is the third most important for tourism activities and expenditure after Siem Reap and Phnom Penh. The town still has open land for future development of tourism facilities and other sector service facilities.

38. Based on water bills from Battambang waterworks in 2015, there are currently 314 commercial facilities. Assuming that tourism facilities are to increase by 2% per year, projected future associated commercial establishments are presented below.

Table 11. Served commercial facilities in the future

Facility	2015(i)	2020	2025	2030	2035	2040
Hotel and Guesthouse						
Restaurant	314	347	383	423	467	515
Supermarket/store						
Total :	**314**	**347**	**383**	**423**	**467**	**515**

Note: (i) *Data obtained from Battambang Waterworks, April 2016*

4.3 Served Public Buildings

39. As the provincial town of Battambang province, there are many public buildings including government offices, hospitals, schools, university, and religious buildings.

40. Based on the water bills of Battambang waterworks in 2015, there are 74 public buildings. Assuming that the public buildings will increase by 1% per year, projected future commercial establishments are presented below.

Table 12. Estimation of served public buildings in the future

Public building	2015(i)	2020	2025	2030	2035	2040
School/Pagoda/church/ government building	74	78	82	86	90	95
Total :	74	78	82	86	90	95

Note: (i) *Data obtained from Battambang Waterworks, April 2016*

5 Water Demand Projections

5.1 Domestic Water Demand

41. According to available consumption data[10] shown in Table 13. , the average daily water consumption per capita is 86 liters. However, following review by MIH, and at their specific request, the future daily water consumption per person for the purposes of design was increased to 140 and 155 liters for Year 2025 and Year 2040 respectively. The domestic water demand for the target years 2025 and 2040 are shown in Table 14.

Table 13. Daily water consumption from 2012 to 2015

Parameter	2012	2013	2014	2015	Average
Domestic water consumption per year (m^3)	1,649,070	1,752,365	2,039,620	2,356,075	
Total served population (person)	59,262	60,507	60,835	67,374	
Water consumption lpcpd	76	79	92	96	86

Source: Battambang Waterworks, June 2016

Table 14. Domestic water demand in 2025 & 2040

City/District	Sangkat/ Commune name	2025			2040		
		Served Population	Water consumption	Water demand	Served Population	Water consumption	Water demand
		Persons	lpcpd	m3/d	Persons	lpcpd	m3/d
BTB	Chamkar Samraong	23,625	140	3,308	33,375	155	5,173
	Kdol Doun Teav	11,965	140	1,675	15,521	155	2,406
	Ou Char	19,501	140	2,730	22,140	155	3,432
	Ou Mal	12,364	140	1,731	15,891	155	2,463
	Preaek Preah Sdach	13,919	140	1,949	17,010	155	2,637
	Rotanak	15,965	140	2,235	19,137	155	2,966
	Sla Kaet	8,594	140	1,203	11,787	155	1,827
	Svay Pao	22,024	140	3,083	22,694	155	3,518
	Tuol Ta Aek	23,445	140	3,282	31,139	155	4,827
	Voat Kor	18,315	140	2,564	22,988	155	3,563
	Sub-total	***169,717***		***23,760***	***211,684***		***32,811***
Sangker	Anlong Vil	18,167	140	2,543	22,415	155	3,474
	Norea	5,529	140	774	6,719	155	1,041
	Ou Dambang 1	15,145	140	2,120	20,348	155	3,154
	Ou Dambang 2	13,745	140	1,924	18,803	155	2,915
	Voat Ta Muem	17,380	140	2,433	25,342	155	3,928
	Sub-total	***69,966***		***9,795***	***93,627***		***14,512***
Thma Kuol	Chrey	17,629	140	2,468	26,825	155	4,158
	Ou Ta Ki	18,141	140	2,540	27,057	155	4,194
	Sub-total	***35,770***		***5,008***	***53,882***		***8,352***
	Total:	**275,453**		**38,563**	**359,193**		**55,675**

[10] Data from Battambang Waterworks 2012-2015

5.2 Commercial, Public Building, and Industrial Water Demand

42. Water consumption data from Battambang Waterworks from 2012 to 2015 indicates that the annual average water demand of commercial properties is 34.7% of annual domestic water demand, and the annual average water demand of public buildings is 21.6%. The water demand of the two identified industrial connections was 0.9% in 2015, as shown in Table 15. However, there are currently 37 small businesses classified as industrial or handicraft settled within the proposed service area (refer to Table 6.), and as expected by the MIH/DIH most of these will use the town water supply once they can be connected. MIH/DIH informed from experience that the industrial water demand can be up to 15% of the total domestic water demand[11]. Consequently, 10% is adopted for design purposes, as these industries settled in Battambang are mostly small scale operations with smaller water demands.

43. Following the trend of urban development and as suggested by MIH/DIH, the adopted commercial water demand is 35% of domestic demand, public building water demand is 20% of domestic water demand, and industrial water demand is 10% of domestic water demand for Year 2025. For the Year 2040 we assume the water demand of commercial, public building, and industrial to be 30%, 15%, and 10% of the domestic water demand. The water demand by customer category for 2025 and 2040 is shown in Table 16.

Table 15. Actual measured consumption by customer category

No.	Category	Water consumption, m^3				Compare to domestic
		2012	2013	2014	2015	Average, %
1	Domestic	1,649,070	1,752,365	2,039,620	2,356,075	
2	Commercial	556,625	569,765	714,305	883,665	34.7
3	Public buildings	371,205	416,100	433,255	441,650	21.6
4	Industries	0	0	0	22,236	0.9
	Total :	2,576,900	2,738,230	3,187,180	3,703,626	

Source : Battambang Waterworks, June 2016

Table 16. Projected water demand by customer category for 2025 & 2040

		2025			2040		
No.	Description	Adopted criteria		Water Demand (m3/d)	Adopted criteria[(2)]		Water Demand (m3/d)
1	Domestic	140 lpcpd[(1)]	daily consumption	38,563	155 lpcpd	daily consumption	55,675
2	Commercial	35%	of domestic	13,497	30%	of domestic	16,702
3	Public building	20%	of domestic	7,712	15%	of domestic	8,351
4	Industrial	10%	of domestic	3,856	10%	of domestic	5,567

<u>Note</u>: (1) lpcpd- Liter per capita per day
(2) It was assumed that in 2040 the commercial and public building consumption will be lower in percentage of the domestic daily consumption than in 2025, as the expansion is toward the suburban/rural areas with less commercial activities and public services building.

[11] Consultant advised to use this percentage by Secretary of State, MIH at mid-term workshop

5.3 Non-Revenue Water

44. Based on four years' recorded data the trend of non-revenue (NRW) dropped from 20.47% in 2012 to 10.96 in 2015, as shown in Table 17. A larger amount of water was lost in 2012 and 2013 due to older leaking pipeline which was replaced by JICA Phase II Project (2014-2016), reducing the overall NRW figure. For design purposes a NRW figure of 15% is adopted.

Table 17. Non-Revenue Water in the existing system

No.	Description	2012	2013	2014	2015	Average
1	Total production per year, m^3	3,240,182	3,414,393	3,726,846	4,134,726	
2	Total collection water per year from bills, m^3	2,576,986	2,738,076	3,187,355	3,681,558	
3	NRW (%)	20.47	19.81	14.48	10.96	16.43

Source: Data obtained from Battambang Waterworks, June 2016

5.4 Daily Peak Factor

45. According to the data recorded by Battambang Waterworks from 2006-2010 the actual daily peak factor from flow records (ratio of daily maximum to daily average) was 1.17, as summarized in Table 18. This data includes the seasonal variations during the dry (March/Early April) and wet (Late April/August) periods. Therefore 1.2 is adopted as the daily peak factor used in calculating the required WTP capacity.

Table 18. Daily peak factor in Battambang

Battambang			2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	Ave.
Daily average		m^3/day	4,587	7,452	7,412	7,903	8,243	8,649	8,877	9,355	10,211	11,328	
Daily Max.	amount	m^3/day	5,720	8,100	8,700	9,215	9,564	9,668	9,772	10,868	11,596	15,009	
	day		17-Aug.	8-Apr	10-Mar	30-Apr	6-Apr	5th Apr	1st May	1st June	5th Mar	20th Aug	
Peak Factor			1.25	1.09	1.17	1.17	1.16	1,11	1,10	1,16	1,13	1,32	1.17

Source: Obtained from BTB waterworks, September 2016

5.5 Summary of Water Demand

46. The total water demand by the target Year 2025 and 2040 is summarized in the table below. The total water demand including daily peak factor has been used in determining the water treatment plant (WTP) capacity.

Table 19. Summary total water demand & water to be produced in Year 2025 & 2040

No.	Description	Unit	2025	2040
1	Total Population in coverage area	person	297,752	372,432
2	Served population in coverage area	person	275,453	359,193
3	Domestic	m^3/d	38,563	55,675
4	Commercial	m^3/d	13,497	16,702
5	Public building	m^3/d	7,713	8,351
6	Industrial	m^3/d	3,856	5,567
7	***Total water demand***	**m^3/d**	***63,630***	***86,296***
8	NRW (15%)	m^3/d	9,544	12,944
9	***Total water to be distributed***	m^3/d	***73,174***	***99,240***
10	Existing WTP (ADB+JICA phase I)	m^3/d	11,520	11,520
11	Existing WTP (JICA phase II)	m^3/d	22,000	22,000
12	Total additional water required	m^3/d	39,654	65,720
13	Daily peak		1.20	1.20
14	**Total water to be produced with peak factor**	**m^3/d**	**47,585**	**78,865**

6 Design Criteria

6.1 Phasing of the facilities

47. The water supply subproject is designed to provide the urban water supply for a 20 year period, i.e. the design capacity is to serve 2040 demand. However in an economic point of view, the implementation of the project is proposed to be phased, to meet the water demand of the target Year 2025 initially as the government's development goal on urban water supply and economical vision, and later to meet 2040 water demand. Thus the subproject will be implemented into two phases as shown in Table 20.

Table 20. Components for proposed Phases 1 and 2.

	Target year	Components	Civil Works to be implemented	Equipment to be implemented
Phase I	2025	Intake	For 2040 target year	For 2025: 4 pumps
		Raw water transmission main	5 km	Pipeline diameter 710mm
		WTP Phase I	50,000 m^3/d	4 booster pumps
		Distribution network	Primary mains (160-630mm) 109 km + 11km secondary mains Target areas (> 11,636 ha)	
Phase II	2040	Intake	*Already provided in 2025*	For 2040: 2 additional pumps
		Raw water transmission main	Additional 5 km in parallel	Pipeline diameter 500mm
		WTP Phase II	Extension 30,000 m^3/d	2025 pumps replaced by 6 pumps, due to large head difference (25m)
		Distribution network	*Additional pipelines to cover infill areas* *Primary mains are estimated as same size, length and the same target area as for 2025, ie. 120km*	

6.2 WTP & Intake capacity

48. The table below summarizes the calculation for the proposed WTP & Intake capacity. The existing WTPs (ADB, JICA Phase I, and JICA Phase II) capacity is 33,520m^3/d. The proposal is to construct a new WTP to meet 2040 demands in two stages, to avoid having a larger WTP operating under capacity for the initial decade[12]. The first stage will be to meet demand of 2025, with land reserved for the following extension for 2040 demand.

Table 21. Total water demand in 2025 & 2040

No.	Description	Unit	2025	2040
1	Total Population in coverage area	person	297,752	372,432
2	Served population in coverage area	person	275,453	359,193
3	Domestic demand	m^3/d	38,563	55,675
4	Commercial demand	m^3/d	13,497	16,702
5	Public building demand	m^3/d	7,713	8,351
6	Industrial deman	m^3/d	3,856	5,567
7	**Total water demand**	**m^3/d**	**63,629**	**86,295**
8	NRW (15%)	m^3/d	9,544	12,944
9	**Total water to be distributed**	**m^3/d**	**73,173**	**99,239**
10	Existing WTP (ADB+JICA phase I)	m^3/d	11,520	11,520
11	Existing WTP (JICA phase II)	m^3/d	22,000	22,000
12	**Base water demand for CDIA project**	**m^3/d**	**39,653**	**65,719**
13	Daily peak		1.20	1.20
14	CDIA Ultimate WTP capacity (2025+2040)	m^3/d	**47,584**	78,863
15	CDIA WTP capacity for 2025	m^3/d		50,000
16	Total additional water required for 2040	m^3/d		**28,863**
17	WTP process losses (5%)	m^3/d	2,379	1,443
18	Total WTP production requirement/ intake extraction (by phase)	m^3/d	**49,963**	**30,306**
19	Design capacity of the new phase of WTP	m^3/d	**50,000**	**30,000**

49. The additional 30,000m^3/d WTP capacity is less in 2040 than in 2025 because the new population to be served by the significantly expanded service area of 2025 is larger than the additional population due to the expected growth in the service area between 2025 and 2040. Most of the main lines and the distribution network of the entire proposed service area are to be laid by 2025. Distribution pipelines laid during Phase 2 will primarily serve only infill areas, with most key distribution mains provided under the 2025 phase.

6.3 Intake location & design criteria

6.3.1 Intake location

50. Three sites have been proposed by the Battambang Waterworks for the proposed intake location as shown on Figure 9. Site 1 (Intake 1) is located close to the JICA Phase II intake, Site 2 (Intake 2) is located about 3 km upstream of JICA Phase II intake, and Site 3 (Intake 3)

[12] As requested by HE Ek Son Chan.

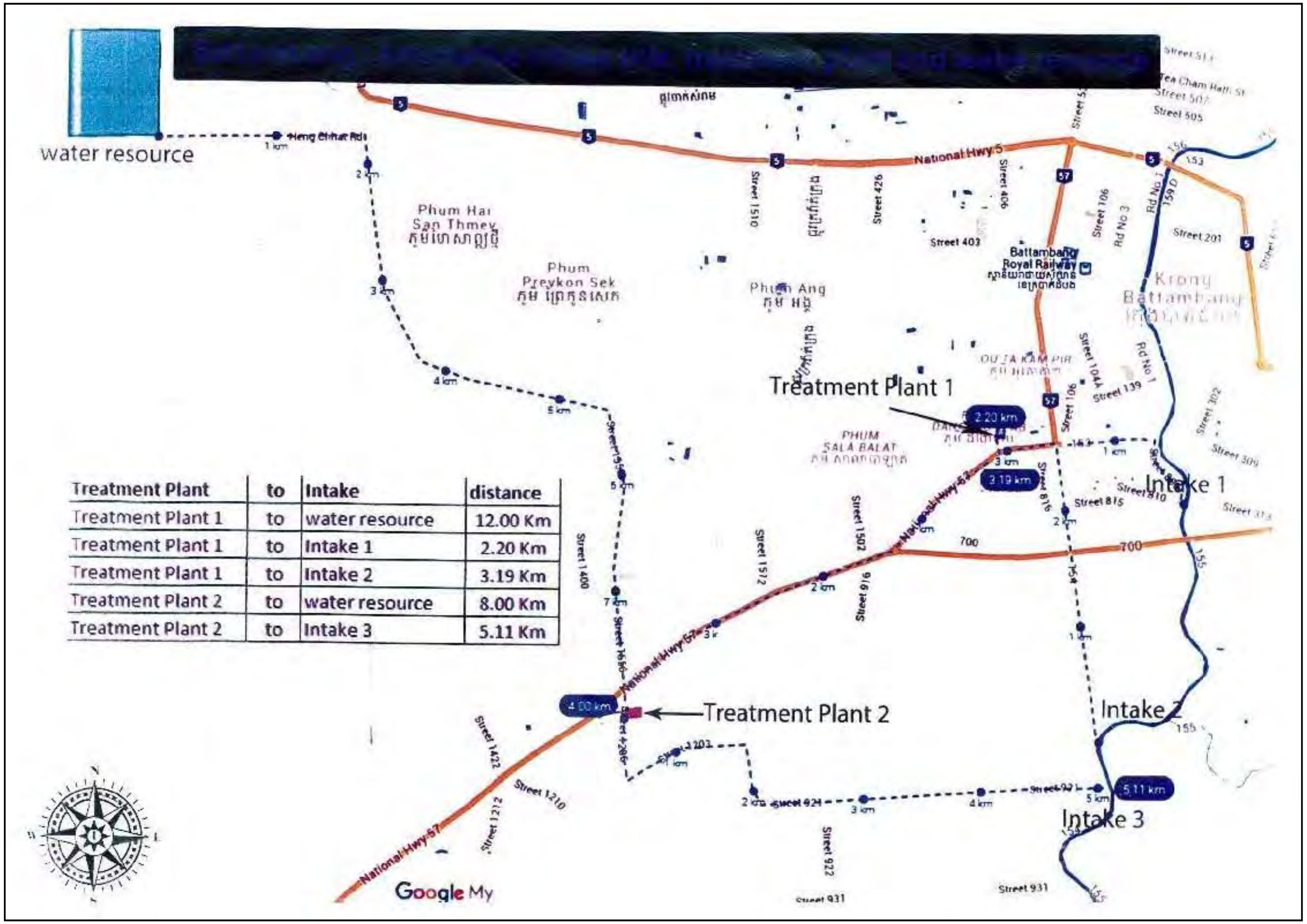
is located upstream of Site 2. The table below summarises the advantages and disadvantages of each proposed intake site.

Table 22. Advantage and disadvantage of the intake sites

	Advantages	Disadvantages
Site 1	- Close to the existing WTP and waterworks office - Easy in terms of connection to the power source to run the pump - Less cost on power connection - Easy in terms of maintenance and security	- Far away from the new WTP site -about 6km - Higher investment on the raw water transmission pipe installation - Higher operation cost for the pump - Potential polluted water source from the domestic wastewater and solid waste in the downtown area, and downstream of a local market - Land acquisition is needed in the downtown area (expensive)
Site 2	- Located upstream of the town, less community settlement - Lower risk of pollution from both solid and wastewater - 1.2 km shorter length than Site 1 to the new WTP	- More study required on the power source connection - Higher cost for power connection - No available land
Site 3	- Located upstream of the town, less community settlements - Lower risk of pollution, both solid and wastewater - 1 km shorter length than Site 1 to the new WTP - More stable river bank than Site 1 and 2 - Available public land confirmed by waterworks	- More study required on the power source connection - High cost for power connection

51. Based on field observations and engineering assessment, Option 3 was selected, approximately 3km upstream from the JICA Phase II intake in Kampong Seima village of Sangkat Vaot Kor. The intake structure will provide enough space for elecro-mechanical equipment to meet the full capacity for year 2040, but initially only equipment to meet year 2025 will be installed. Additional pumps and controls will be installed later to meet the capacity required for year 2040.

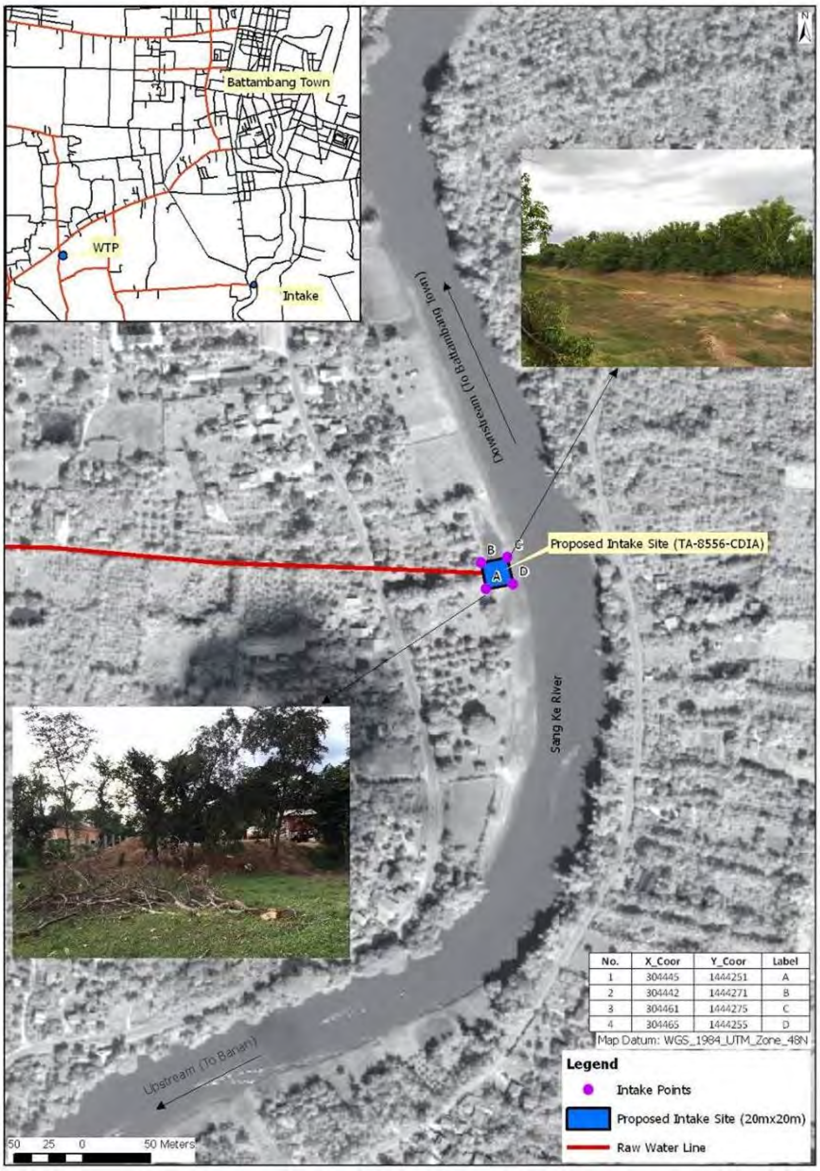
Figure 9. Proposed intake locations



Treatment Plant	to	Intake	distance
Treatment Plant 1	to	water resource	12.00 Km
Treatment Plant 1	to	Intake 1	2.20 Km
Treatment Plant 1	to	Intake 2	3.19 Km
Treatment Plant 2	to	water resource	8.00 Km
Treatment Plant 2	to	Intake 3	5.11 Km

Source: Obtained from BTB's Waterworks

Figure 10. Selected Intake location



(Source: "Preparatory Survey on the Project on Additional New Water Treatment Plants for Kampong Cham and Battambang Waterworks in the Kingdom of Cambodia, March 2013, JICA")

6.3.2 Type of intake

52. Different types of intakes have been considered, including an intake tower and riverbank side intake. As the river on which the intake will be constructed is relatively narrow and will be used for boat traffic, the intake tower cannot be considered and the side intake type is recommended.

53. Key design considerations are the stability of the river bank, and positioning the intake pipe below the dry season water level to avoid floating debris.

54. This intake facility will deliver raw water towards the west, about 5km along an existing road to the proposed WTP (shown as WTP 2 on Figure 9.).

55. Based on records[13], the Sangker river high-water level (HWL) is 15.57m, and the low water level (LWL) is 4.81m above mean sea level.

56. The raw water pumping mains for 2025 and for 2040 are proposed as two separate parallel pipes to meet demands at those design years.

6.4 Water Treatment Plant

6.4.1 WTP site location

57. At an early stage of the project the Battambang Waterworks indicated a potential new water treatment plant site about 2.2km from the downtown area near NR#57, as shown in Figure 11. However, after the CDIA study team completed field observations and requested supporting documents, the Battambang Waterworks indicated that this parcel of land was no longer available, and proposed a new site 2km further southwest.

This second proposed WTP site is located about 200m from the National Road #57, 4km from the town center, as shown in Figure 12. According to the available public land records from Battambang DIH inventory (land title No.02030802-1278), this site has an area of 2.4ha in Chrabkrosaing village, Vaot Kor commune.

[13] "Preparatory Survey on the Project on Additional New Water Treatment Plants for Kampong Cham and Battambang Waterworks in the Kingdom of Cambodia, March 2013, JICA"

Figure 11. Initial proposed location for the new WTP, since discarded

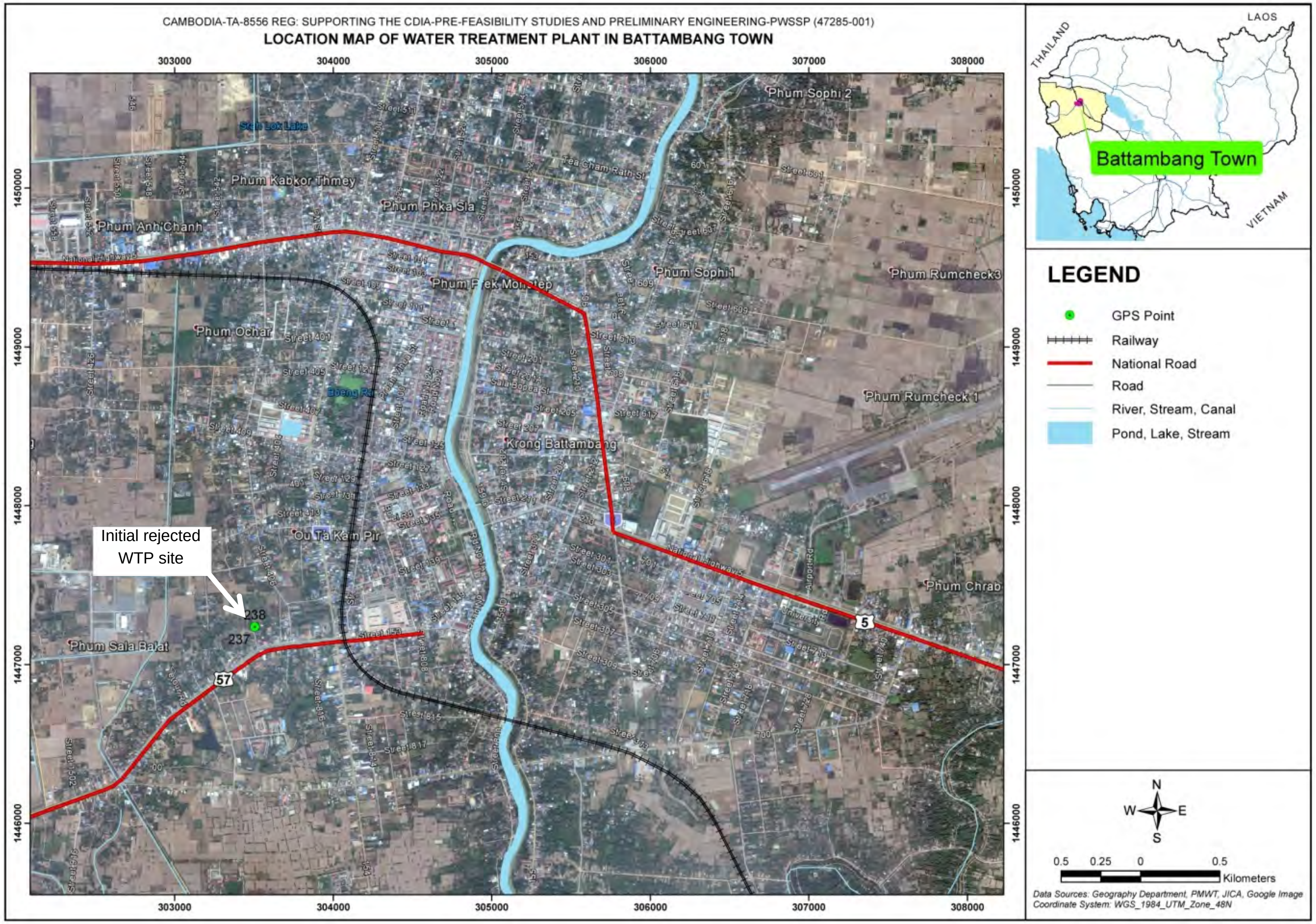
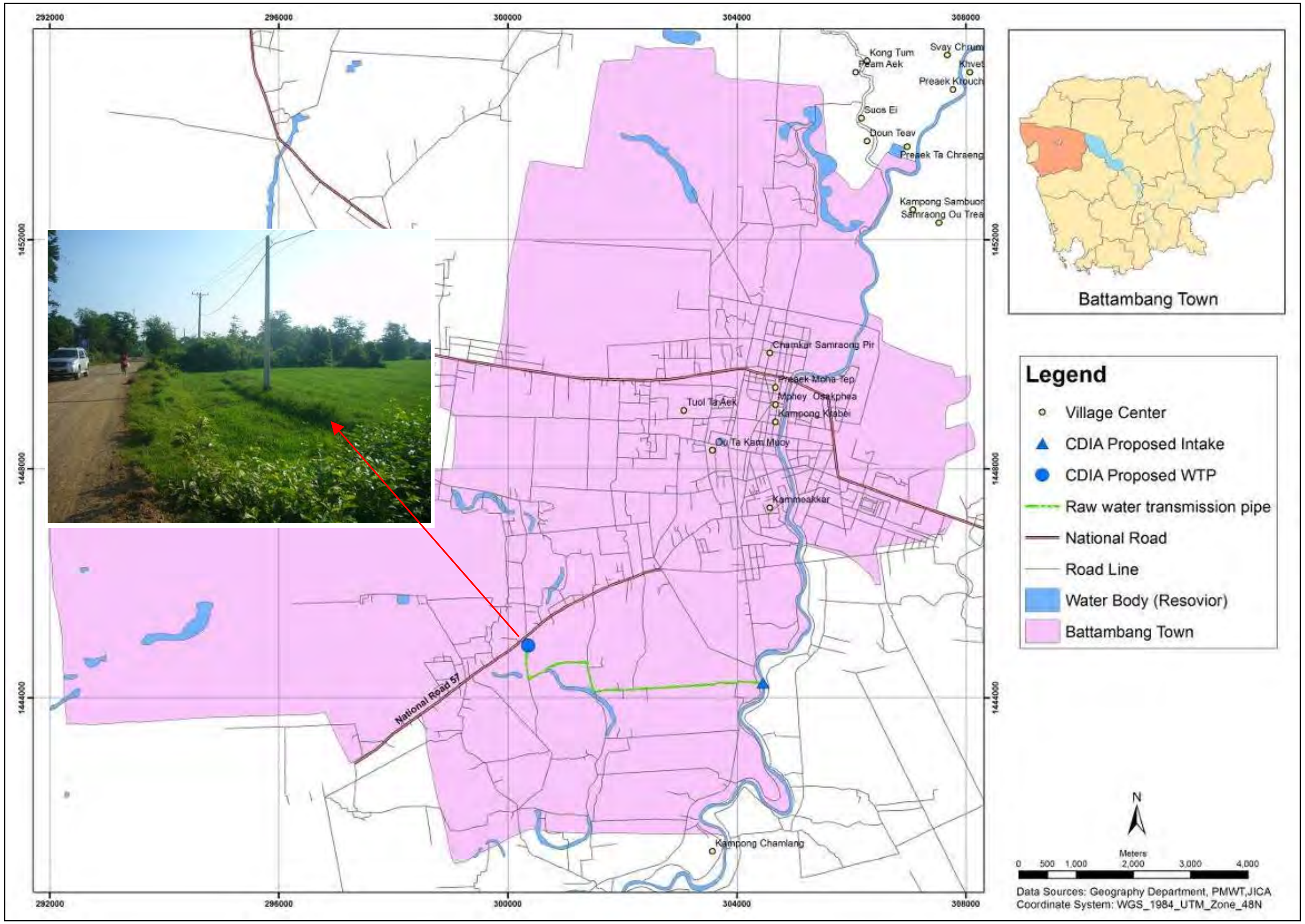


Figure 12. Selected WTP site location



6.4.2 Water quality analysis & process selection

58. Based on the National Drinking Water Standard of Cambodia 2015, there are 27 parameters to be treated for urban water supply as shown in Table 23. The Battambang Waterworks have conducted water quality monitoring from both the Sangker River water source and treated water from the distribution network. Table 24. shows the results of monthly water quality monitoring from the Sangker River source, and from treated water from the WTP.

Table 23. Drinking Water Quality Standard of Cambodia

No.	Parameter	Unit	Maximum value
A	***Micro-organism***		
1	E-coli	MPN/100ml	0
B	***Chemical***		
2	Aluminum (Al)	mg/l	0.2
3	Ammonia (NH_3)	mg/l	1.5
4	Arsenic (As)	mg/l	0.05
5	Barium (Ba)	mg/l	0.7
6	Cadmium (Cd)	mg/l	0.003
7	Chloride (Cl^-)	mg/l	250
8	Residue chlorine	mg/l	0.1-1.0
9	Chromium (Cr)	mg/l	0.05
10	Copper (Cu)	mg/l	1
11	Fluoride (F)	mg/l	1.5
12	Total Hardness ($CaCO_3$)	mg/l	300
13	Iron (Fe)	mg/l	0.3
14	Lead (Pb)	mg/l	0.01
15	Manganese (Mn)	mg/l	0.1
16	Mercury (Hg)	mg/l	0,001
17	Nitrate (NO_3)	mg/l	50
18	Nitrite (NO_2)	mg/l	3
19	Sodium (Na)	mg/l	250
20	Sulphate (PO_4)	mg/l	250
21	Zinc (Zn)	mg/l	3
C	***Physical***		
22	Color	TCU	5
23	pH		6.5-8.5
24	Total Dissolved Solid (TDS)	mg/l	800
25	Conductivity	NCU	5
26	Taste		Acceptable
27	Odor		Acceptable

Source: Potable Water Quality Standards of Cambodia, MIH, Sept. 2015

Table 24. Water Quality Monitoring

No	Parameter	Unit	PWQS of Cambodia	WHO guideline values, 2011	Results 29/01/2016		Results 28/04/2016	
					Raw water	Treated water	Raw water	Treated water
1	Color	TCU	5	15	232	1	186	1
2	Turbidity	NTU	5	5	147	2.4	40	1
	Residual chlorine	mg/l	0.2-0.5			0.96		0.95
3	pH		6.5-8.5	6.5-8.5	7.94	7.84	8.27	7.72
4	Total dissolved solids	mg/l	800	1000	119	124	174	176.5
5	Manganese (Mn)	mg/l	0.1	0.01	0	0.07	0.06	0.05
6	Zinc (Zn)	mg/l	3	3	1.02	1.39		
7	Sulfate (SO_4)	mg/l	250	250	4	6		
8	Copper (Cu)	mg/l	1	2	0	0.004		
9	Hydrogen Sulfide (H_2S)	mg/l	0.05		0.074	0.034		
10	Hardness	mg/l	300		100	100		
11	Aluminum (Al)	mg/l	0.2/	0.2	0.03	0.04		
12	Chloride (Cl)	mg/l	250	250	10.14	13.9	52	55.52
13	Iron (Fe)	mg/l	0.3	0.3	1.28	0.12	0.635	0.07
14	Ammonia (NH3-N)	mg/L	1.5	1.5	0	0	0.06	0.01
15	E. Coli	MPN/100ml	0	0		0	580	0
16	Total Coliform	MPN/100ml	0	0		0	820	0
17	Alkalinity (as CaCo3)	mg/l			106	104	163.6	157
18	Conductivity	µs/cm			238	247	348	353
19	Organic Carbons	mg/l			6.6	5.7	10.49	8.73

Note: PWQS – Potable Water Quality Standards of Cambodia, MIH, Sept. 2015
Source : Battambang Waterworks, June 2016

60. The data in Table 24. indicates that turbidity, color, iron, and E.Coli are well treated by the existing conventional treatment plant and the results meet both the National and WHO drinking water standards. The chemical parameters in the raw water can be removed by the conventional treatment process - coagulation/sedimentation to remove turbidity, color and iron; rapid sand filtration with air/water backwash, and chlorination are used to oxidize hydrogen sulfide and disinfect bacteria. Chlorine is also beneficial for the distribution system: the residual chlorine protects the water against bacterial pollution from leaking pipes up to the distribution tap.

61. Based on the JICA Phase II water quality results, toxic substances such as heavy metals and cyanide were either not reported or below the water quality limits, and the Sangker River water was considered safe for the raw water source to JICA's Phase II WTP. It is also considered safe for the proposed new WTP and system.

62. Thus the conventional treatment process (coagulation – flocculation - sedimentation - rapid sand filtration – chlorination – clear water storage), the same as the process used in the vast majority of plants in Cambodia, is deemed most appropriate and this was agreed by the MIH/DIH. The conventional treatment process is also widely utilized in Laos and Vietnam and is relatively simple to operate and maintain.

63. Sodium hypochlorite powder is selected for disinfection as it is available locally, and safer to handle than chlorine gas.

64. The proposed treatment process includes the drying beds for treatment of sludge.

65. The proposed WTP system will distribute the water from the clear water reservoir to the cusumers from the WTP booster pump station located at the WTP site.

6.4.3 WTP process design criteria

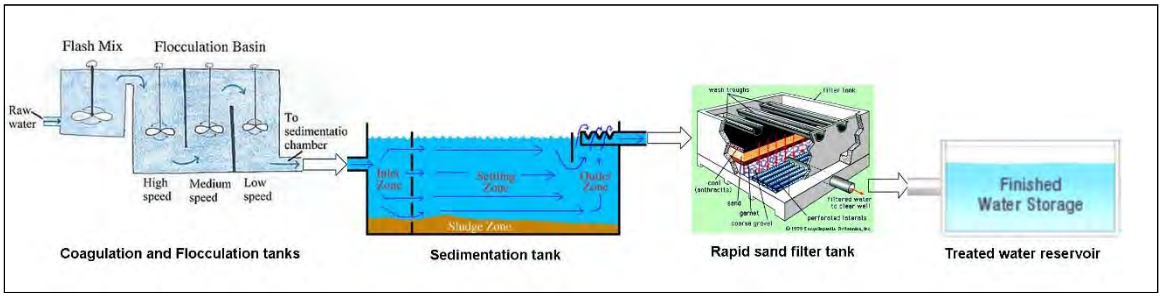
66. For economic and operational reasons, the WTP will initially be installed for 2025 capacity only, and space will be reserved for additional units to meet 2040 water demand.

67. The design criteria for the WTP are presented in the table below and a basic process diagram shown in the following figure:

Table 25. Design criteria for WTP preliminary design

Facility	Component
Design capacity	50,000 m^3/h
Running time	24 hours per day
Receiving Well	Retention Time: T=2.8 min in dry season (T≥1.5 min)
Rapid Mixing Tank	Retention Time: T=2 min (2 <T< 5 min)
Flocculation Tank	Contact Time: 30 min
Sedimentation Tank	Horizontal-Flow Sedimentation Basin Surface Loading: Q/A=20.0 mm/min (15-30 mm/min) Mean Velocity: V=0.14 m/min (below 0.40 m/min)
Rapid Sand Filter	Filtration Speed: V=5 m/hr Maximum filtration speed (N-1): 10m/h Backwash Method: Simultaneous Backwash Method by Air and Water
Backwash	Backwash pumps and blowers
Chlorination	Sodium hypochlorite powder Contact time: 30 min Treatment rate: 5 mg/l
Clearwater Reservoir	Retention Time: T= 3 hours
Sludge Drying Bed (Lagoon)	Sludge quantity production: To be determined
Chemical Feeding Facility	Alum: To be determined Lime: To be determined

Figure 13. WTP Process diagram



6.5 Distribution Network

6.5.1 Network Philosophy

68. Piped water supply shall be installed along roads in populated areas and areas where there is a potential for further housing development. The pipe distribution network is designed to supply the proposed extended service areas and also to complement the existing service areas during the period 2019 to 2040.

69. Two options for the distribution network have been modelled and analysed:

Option 1 uses a booster pump at the clearwater reservoir within the WTP grounds and two elevated water towers within the service areas to maintain pressure during peak hour and emergency water needs, see Figure 14.

Option 2 uses only a booster pump from the clearwater reservoir, see Figure 15. Treated water from the WTP will be pumped directly to the consumers without intermediate elevated water towers. This will minimize the land acquisition required for the elevated water towers.

Figure 14. Network arrangements Option 1: booster pump with water tower

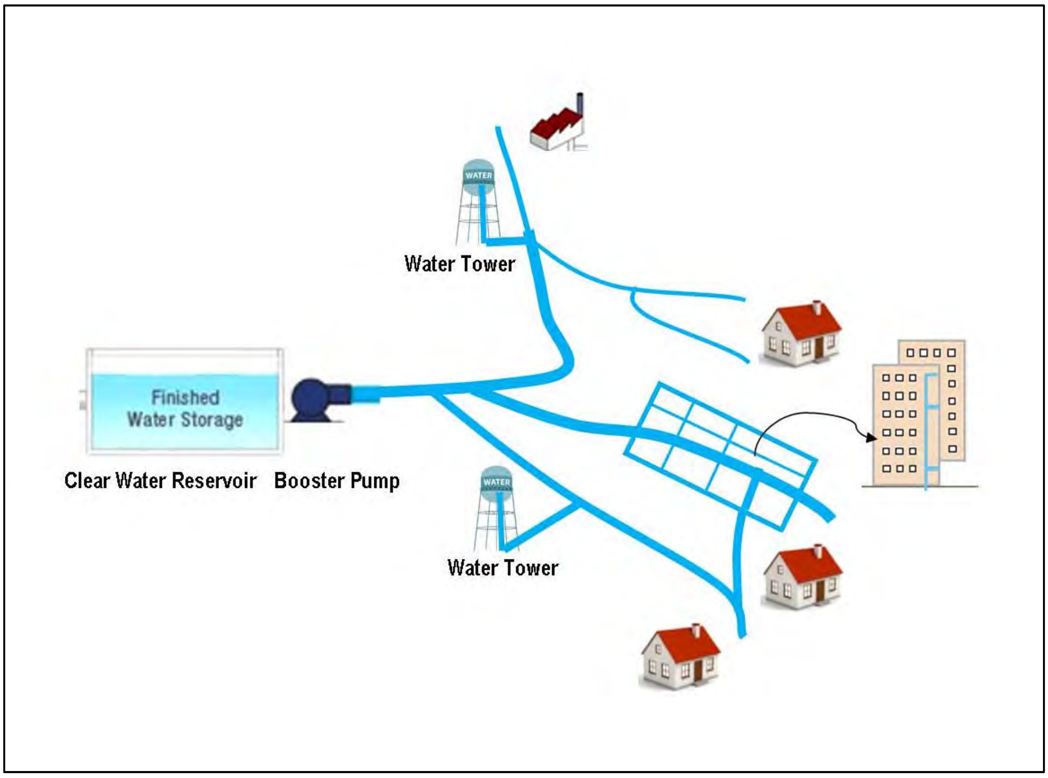
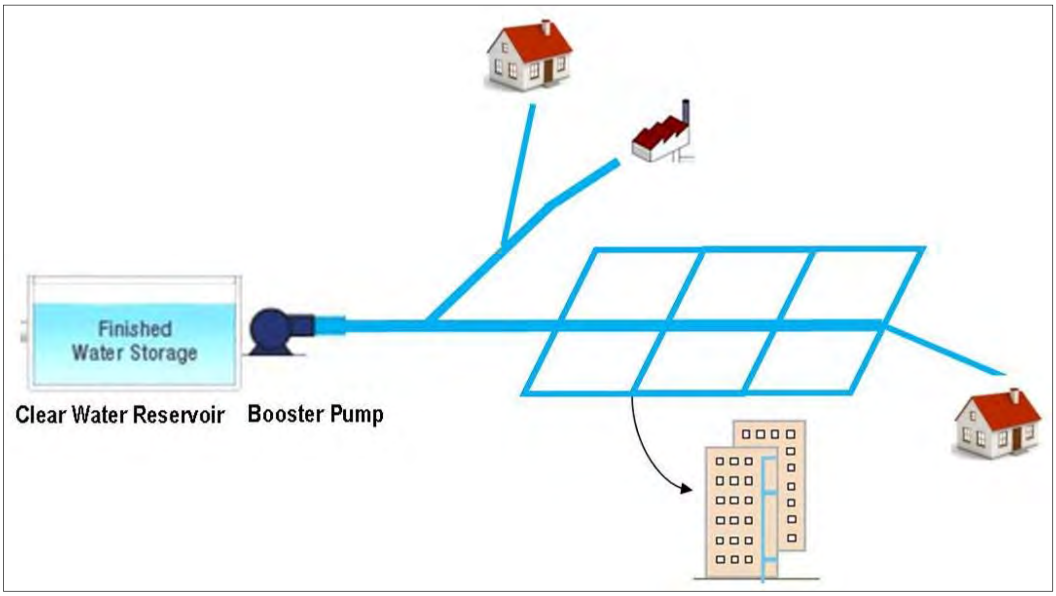


Figure 15. Network arrangements Option 2: booster pump only


Finished
Water Storage
Clear Water Reservoir
Booster Pump

6.5.2 Hourly peak factor

70. The proposed expanded coverage areas are towards the outer suburbs of Battambang. In general, the value of the hourly peak factor is related to number of water consumers or amount of total water demand, the larger the number of consumers or larger water demand giving a smaller hourly peak factor. An hourly peak factor in the range 1.5-3.0 is typically used on urban water supply projects. In the document from JICA Phase II[14], the peak hourly factor was determined by the following equation and is detailed in Table 26.

$$K = 2.6002 \times (Q/24)^{-0.0628}$$

Where *K – hourly factor*

Q – Daily supply amount

Table 26. Hourly factor for Battambang

Description	Phase I (2020-2025)	Phase II (2026-2040)
Total daily water demand	47,585 m^3/d	78,865 m^3/d
Hourly factor: K	1.61	1.56

6.5.3 Pipe design criteria

71. The pipe design is based on a 20 year project design period, providing pipes of sufficient diameter to convey 2040 water demand. Two hydraulic modelling scenarios (EPAnet) have been developed: Scenario 1 from year 2020 to 2025 (Phase I), and Scenario 2 from year 2026 to 2040 (Phase II).

72. Pipelines can be divided into raw water pipe, clear water main, main distribution pipeline, secondary distribution pipeline, and house connections. Under the preliminary design, only layouts of raw water pipeline and main distribution pipelines are considered. The secondary distribution and house connection pipelines will be addressed in the detailed design stage following full topo survey and detailed resettlement studies.

73. The most popular pipe material used in urban water supply in Cambodia is the cost-effective high density polyethylene (HDPE) for all distribution networks including trunk lines. HDPE is rigid and tough and it has better stress crack resistance and higher impact resistance than Polyvinyl Chloride (PVC). Fewer joints are required for HDPE pipe, so longer lengths can be laid at one time as compared to uPVC. For the pumped raw water pipeline, ductile cast iron (DCI) is more durable and practical under higher-pressure situations, particularly for the above-ground sections near the intake and WTP outlet.

[14] "Design Criteria for Water Supply Facilities" published by the Japan Waterworks Association

6.5.4 Network Design Criteria

74. The following design criteria are considered for the network:

- Distribution system fed by booster pump.
- The modelled water velocity ranges from 0.3 to 1.6 m/s to avoid settlement of particles inside the pipe.
- The minimum and maximum pressure in the network is 50m and 8m respectively.
- The roughness calculation is based on the Hazen-William formula:

$$\boldsymbol{V = kCAR^{0.63}S^{0.54}}$$

Where $V: discharge\ in\ section\ (m/s)$

$C: roughness\ coefficient$

$R: hydaulic\ radius\ (m)$

$S: friction\ slope\ (m/m)$

$k: factor\ dependent\ on\ units$

- The selected pipe diameters follow HDPE catalogue, with coefficient roughness C=150.
- Water demand is estimated on average water consumption. It is increased depending on the period of the day, and an hourly peak factor of 1.61 and 1.56 was adopted for determining pipe size in 2025 and 2040 respectively.
- Two pipelines along the main roads have been considered, to avoid damaging the road with multiple crossings.
- Air valves and washout valves will be provided at high and low points.
- Ultimately it is recommended that rider mains[15] will be provided both sides of road from which domestic connections will be made.

6.6 Pump station design criteria

75. Two pumping stations are proposed, a raw water pumping station at the intake, and a clear water pumping station inside the WTP grounds.

6.6.1 Intake pump station

76. The intake pump should have sufficient capacity and provide sufficient head to meet the requirement of treatment plant, and the pump type should preferably be of the same type as the pumps that have been used at the intake of the JICA Phase II project, to be compatible with existing operation and maintenance requirements (operator training, spares).

77. Two pipelines will be installed in parallel as raw water mains: one for the 2025 flow and one for the 2040 flow, installed at a later stage in Phase 2.

6.6.2 Clearwater pump station

78. The pumps of the clear water pumping station at the WTP should have the same head as the pumps at the JICA Phase II WTP, to maintain equal water pressure in the network supplied from the two separate WTP's. Pumps are to be installed with capacity to serve year 2025 initially, with floor space reserved for additional pumps to meet further capacity for year 2040 demands.

[15] A small (usually 50mm) pipe from which domestic connections are made. It is usually capped every 500m or so and has one cross connnection to the distribution main. The purpose is to minimise road crossings and minimize fittings on the main.

7 Preliminary Design

7.1 Intake

7.1.1 Intake Structure

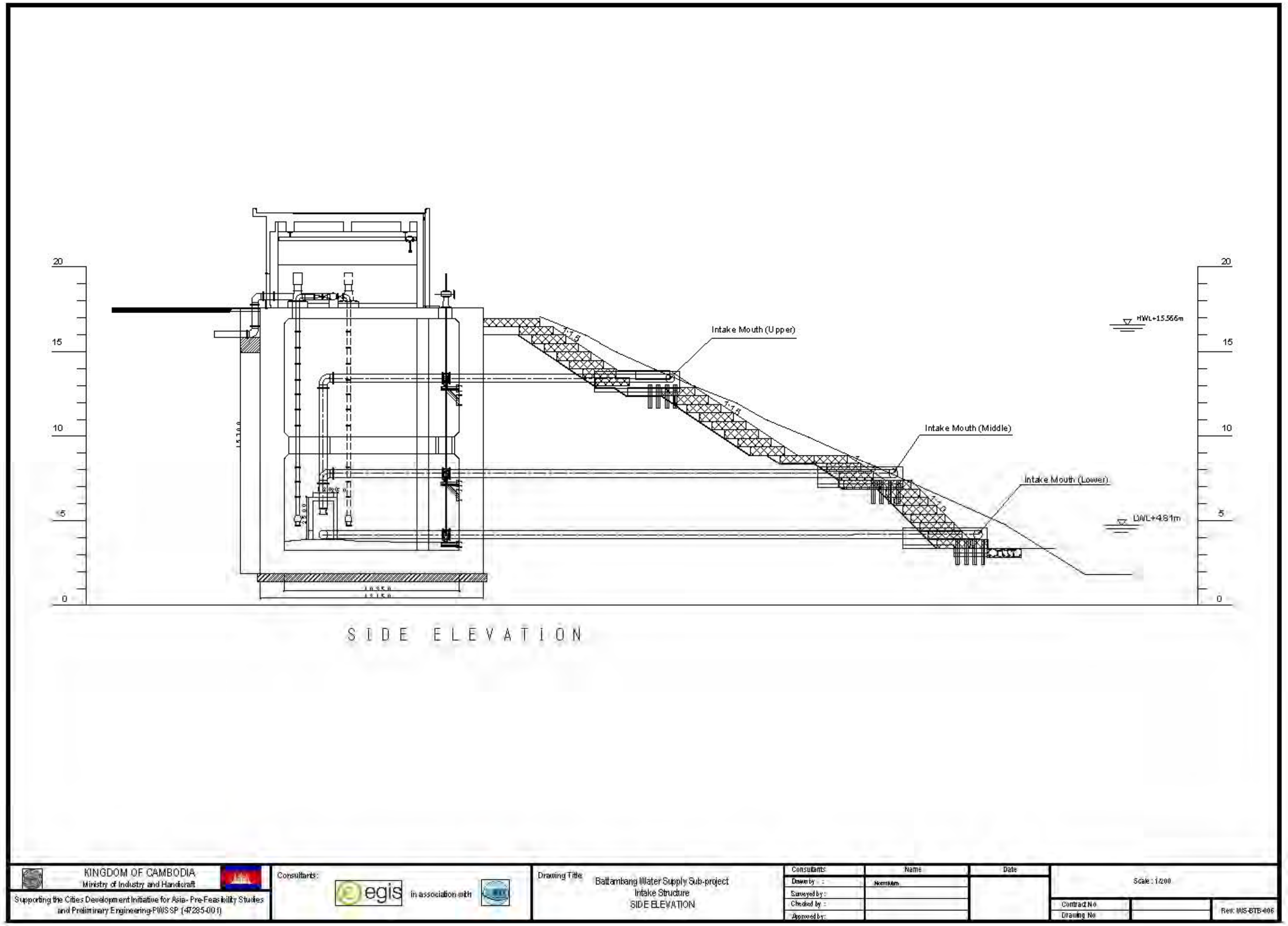
79. The structural design of the proposed intake works is similar to the existing facilities (JICA II), and follows international best practice and standards for reinforced concrete. The intake structure is to be constructed in the river bank, and the structural design should be considered for stability in sliding, overturning, uplifting and slope protection (gabion mattress, concrete or stone pavement, rip-rap, wooden stakes or sheet piles).

80. The intake structure has been sized for the full 2040 capacity, and reserved space for additional future pumps and associated equipment has been included. The main design elements of the intake are ;

- The intake structure dimensions are 13.40m x 12.95m x 15.70m deep
- The pump room dimensions (W*L*H) are 9.60m x 13.30m x 6.00m.
- RC roof slab shall be provided.
- Travelling crane shall be installed for pump lifting.
- A full geotechnical soil investigation is required around the intake site.
- Support for the pipe entry and multi-level access platforms with access ladders are to be installed.
- Screens for the inlet pipe shall be installed.
- A generator room is to be constructed adjacent to the pump room.
- A RC concrete ramp and steps should be provided if necessary.
- A drainage sump is required.

81. A typical section of the proposed intake structure is shown in the figure below.

Figure 16. Typical intake structure



7.1.2 Intake Pumps

7.1.2.1 Intake Pumps for 2025, (Q=50,000m3/d)

82. The total capacity of pump intake is Q_p = 50,000 m^3/d, ie 2,083m^3/hr or about 579 l/s with 24h operation. The number of pump units was determined as 3 duty pumps plus 1 standby pump, each with capacity Qp = 193 l/s.

Diameter of suction pipe

83. From pump calculations included in Annex 2, the diameter of the suction pipe is approximately 400mm with a water velocity of 1.54m/s.

Diameter of discharge pipe

84. The diameter of the discharge pipe (short length of pump outlet inside pump chamber) is approximately 350mm with water velocity 2.01m/s. The downstream raw water transmission main pipe is calculated as having diameter 710mm with velocity 1.79m/s.

Pumping Head

85. The pumping head is the sum of the static head, entry losses, friction loss in the suction pipe, discharge losses, and other loss from fittings as described in equation below:

$$H_t = H_{stat} + h_{ent} + h_{fs} + h_{fd} + \sum h_m \, H_t = H_{stat} + h_{ent} + h_{fs} + h_{fd} + \sum h_m$$

H_t = 16.68 + 0.006 + 0.082 + 15.887 + 0.944 = 33.6m.

The total dynamic head (H_t) has been rounded to 35m; see detailed calculation in Annex 2.

Pump type selection

86. The total dynamic head required is over 10m (Ht = 35m). A vertical turbine pump is designed for this higher suction lift and an example layout is shown in Figure 16. The motor sits on top of the well, with the impellers at the bottom connected by a long vertical shaft.

87. Both vertical turbine pumps and submersible pumps were presented as options for the intake pumps at the DFR Workshop. MIH commented that submersible pumps are not to be considered due to previous problems experienced by various waterworks (spare parts not available easily). The existing water intake pumps under the JICA 2 project are vertical turbine pumps, and waterworks officials are hesitant to use different types of pumps both to maintain familiarity and for ease of sourcing spares and maintenance.

88. Horizontal split-casing pumps have been considered as an alternative. They may be easier to maintain, but do not currently exist in the Cambodian water sector and are not suitable for pumping with high suction lift.

89. JICA Phase II booster pump systems are already utilizing a Frequency Inverter system for energy savings and to improve efficiency. Under this subproject the variable frequency inverter (VFI) system will also be adopted in all of the pumping systems.

Figure 17. Typical Vertical Turbine Pump arrangement

7.1.2.2 Intake Pumps for 2040, (Q=30,000m^3/d)

Diameter of suction pipe

90. From report pump calculations in Annex 2, the diameter of the suction pipe (DCI) is approximately 400mm with water velocity 1.38 m/s.

Diameter of discharge pipe

91. The discharge pipe (short length of DCI pump outlet inside pump chamber) diameter is approximately 350mm with velocity 2.01 m/s. The downstream raw water transmission main pipe (DCI) has diameter calculated at 560mm with velocity 1.72m/s.

Pumping Head

92. The pumping head is the sum of the static head, entry losses, friction loss in the suction pipe, discharge losses, and other loss from fittings as described in equation below:

$$H_t = H_{stat} + h_{ent} + h_{fs} + h_{fd} + \sum h_m$$

The total dynamic head (H_t) has been derived and H_t = 38m.

See detail calculations in Annex 2

Pump type selection

93. For the same reasons described before, the vertical turbine pump is selected. Table 27. summarizes the intake pump characteristics for 2025 and 2040.

Table 27. Summary of intake pump capacity and characteristic

Description	Year 2025	Year 2040
Total pump capacity	579 l/s	347 l/s
Total pump head	Ht = 35 m	Ht = 38 m
Pump type	Vertical Turbine with variable speed drive	Vertical Turbine with variable speed drive
Number of pump	3 duty and 1 standby; capacity 193 l/s each pump	2 duty; capacity 231 l/s each pump
Total power required	270kw	180kw
Column/suction pipe (DCI)	Φ400 mm	Φ400 mm
Discharge pipe (DCI)	Φ350mm	Φ350mm
Main pipe (DCI)	Φ710mm, L = 5000m	Φ560mm, L = 5000m
Transformer 22KV/0.4KV, 3 phases 4 wire, Outdoor Type	500kVA	315kVA
Generator 400 V/230 V, 3 phases 4 wire, 50Hz, Silent Type	380kVA	275kVA
MV Pole Concrete for Incoming Overhead Line 22KV	2-pcs ; Voltage=22KV ; AL-150mm2	

7.1.2.3 Sump Drainage pump for 2025 & 2040

94. The capacity required for sump drainage, during intake sump maintenance, is approximately 43.6 l/s (157m^3/h) for both 2025 and 2040. The number of pumps selected is 1 duty only. The total pump head is about Ht = 18m. The diameter of discharge pipe is calculated as 150mm. The power required is 11kw. The sump pump will be utilized only during maintenance/ dewatering of the intake chamber. Therefore the required capacity will not change over time.

7.2 Water Treatment Plant

95. The required design capacity of the proposed WTP has to provide both sufficient water to serve the JICA Phase II coverage area after year 2019, and to provide additional capacity for the proposed extended coverage area under this project to 2025. However, the full required “Phase 2” WTP capacity to 2040 has also been calculated in order to reserve enough land for 2040 water demand horizon.

96. The WTP has 2 trains and is proposed to normally run 24 hours. As its is currently the practice in Cambodia, during times of maintenance that require a partial shutdown, for example when desludging a sedimentation tank, one train can be temporarily shut down, and either half of the peak WTP capacity delivered within expected water quality standards, or full flow delivered to a lower quality. Media messages would need to be used to inform the public during these temporary shutdowns, which typically take less then 48 hours[16]. In addition, the maximum WTP flow (which includes a 1.2 peak day factor) is 47,858m^3/d for 2025, and has been rounded up to 50,000m^3/d. This allows for an additional 2,142m^3/d buffer during maintenance shutdowns.

[16] This is commonly done in Phnom Penh and Provincial towns when temporary works that disrupt the water supply service are being carried out.

97. Table 28. below summarizes the preliminary design dimensions of the different structures proposed for detailed design and construction at the WTP site.

98. The chemical dosing system uses aluminum sulfate for coagulant, lime for pH and alkalinity adjustment, and chlorine (powdered or granule sodium hypochlorite) for disinfection. Figure 18. presents the proposed WTP layout.

99. Powdered sodium or calcium hypochlorite are proposed for chlorination. The existing older central WTP already uses this. The newer WTP recently constructed under JICA Phase II utilises a gaseous chlorine system under a negative pressure, but this carries a safety risk for the WTP and surrounding community, and is not available in Cambodia. All gas refills and spare parts must be imported.

100. Replacement of gaseous chlorine systems in WTP's across Cambodia was a key recommendation of the 2013 PPTA for the Urban Water Supply & Sanitation project[17] , due to the failure of all gas chlorine system installed under a previous 2006 ADB implementation project[18]. It is not known why it has been installed under JICA projects.

101. There have been some reported difficulties with using powdered source chlorine – dust and blocking of dosing lines – but these can be easily mitigated by enforcement of good operating practices like wearing masks, eye protection and gloves, and by having outlet pipes from mixing and constant head tanks at least 20cm above the tank base, to ensure settled insoluble residual matter does not enter the dosing pipelines.

102. The total land requirement is approximately 1.31 ha for WTP Phase-I (including service building and workshop), with a further 0.51 ha required for WTP Phase-II. A 2.4 ha parcel of land has been committed.

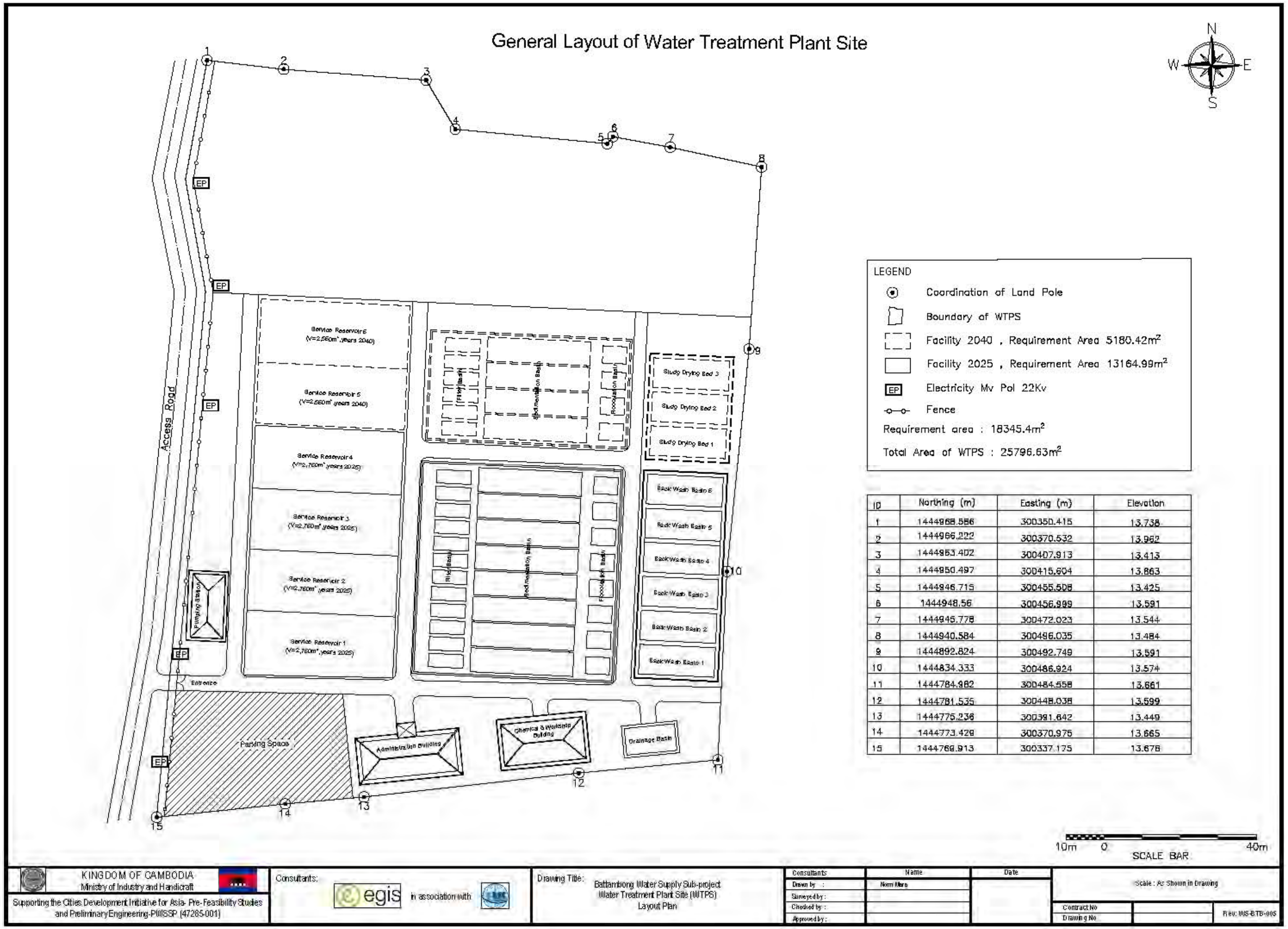
103. The preliminary layout of the proposed WTP is shown on Figure 18. The two proposed phases of the WTP are shown in the layout: Phase 1 is drawn with solid-line and Phase 2 in dash-line. The service building, pump station, chemical/workshop building, and other items common to both phases are included in Phase 1 construction.

[17] ADB PPTA: TA-8125-CAM
[18] Metering & dosing equipment corroding and being repaired with hazardous "home made" temporary fixes.

Table 28. Preliminary design of WTP Phase 1

Facility		Component
Water Treatment Plant	Receiving Well	Reinforced Concrete Structure Internal Dimension: (2.00 m width x 5.25 m length x 4.45 m depth) x 2 units Volume: (V=46.7 m^3) x 2 unit,
	Rapid Mixing Tank	Reinforced Concrete Structure Gravitational force mixing using a weir Internal Dimension: (2.00 m width X 3.00 m length X 3.81 m depth) x 2 units Volume: (V=22.9 m^3) x 2 units
	Flocculation Basin	Reinforced Concrete Structure Slow Mixing Method: Vertical channel bands flocculator Internal Dimension: (7.10 m width x 5.90 m length x 4.70 m height) x 8 units
	Sedimentation Basin	Reinforced Concrete Structure Horizontal-Flow Sedimentation Basin Supernatant Collecting System: Collecting Trough + Submerged Orifice Internal Dimension: (7.10 m width X 30.00 m length x 4.12m average depth) x 8 units
	Rapid Sand Filter	Reinforced Concrete Structure Internal Dimension: (3.50 m width X 9.10 m length) x 12 units Filter Sand Thickness: 100-120 cm Underdrain System: Porous Filter Bed Method Backwash: 20m/h with 5 min with air + 10 min with air and water + 10 min with water)
	Clearwater Reservoir	Reinforced Concrete Structure using Flat Slab Structure Volume: V= 12,800 m^3 Water Depth: H= 4.30 Internal Dimension: (16m width x 40m length x 5m height) x 4 units Chlorine dosing weir at inlet
	Backwash Wastewater Basin	Reinforced Concrete Structure Volume: 230 m^3 Internal Dimension: (4m width x 11m length x 5.5m height + 2.70m depth) x 6 units
	Sludge Drying Bed (Lagoon)	Reinforced Concrete Structure Area: 1,950 m^2 Quantity: 3
	Chemical Feeding Facility	Chemical storage room, mixing tanks and constant head tanks.
	Standby Generator	600 KVA (Soundproof type, Equipped with Internal Water Tank)
	Administration and service Building	Reinforced Concrete Structure, Total Floor Area: 590 m^2 Customer service and laboratory, administration and technical services, meeting and staff rooms

Figure 18. Preliminary layout of the proposed WTP site (Phase 1 & 2)



ID	Northing (m)	Easting (m)	Elevation
1	1444968.586	300350.415	13.738
2	1444966.222	300370.532	13.962
3	1444963.402	300407.913	13.413
4	1444950.497	300415.604	13.863
5	1444946.715	300455.506	13.425
6	1444948.56	300456.999	13.591
7	1444945.778	300472.023	13.544
8	1444940.584	300496.035	13.484
9	1444892.824	300492.749	13.591
10	1444834.333	300486.924	13.574
11	1444784.982	300484.558	13.661
12	1444781.535	300448.038	13.599
13	1444775.236	300391.642	13.449
14	1444773.429	300370.975	13.665
15	1444769.913	300337.175	13.678

7.3 Distribution System

7.3.1 Clear water Reservoir Preliminary Design

104. The clear water reservoir capacity was calculated considering the hourly peak demand, which is 1.62 and 1.56 in year 2025 and 2040 respectively. Usually the peak hours occur between 6:00-8:00AM and 5:00-8:00PM, coinciding with beginning and end of school hours and working hours in Cambodia. Three hours' storage for the clear water reservoir is proposed for each phase. The volume requirements of the clear water reservoir are described in Table 29.

Table 29. Sizing of the clear water reservoir

Specifications	Phase I (2020-2025)	Phase II (2026-2040): Additional storage to Phase I
Hourly water demand (m^3)	50,000/24=2,083	30,000/24=1,250
Hourly peak	1.61	1.56
Retention time (hour)	3	3
Stored water volume when WTP not operating (m^3)	6,249	3,750
Absorb peak hourly flow (m^3/d)	3 x (3,354 - 2,083) = 3,813	3 x (1,950 - 1,250) = 2,100
Total storage required (m^3/d)	10,062	5,850

Note: The clearwater reservoir is an underground reinforced concrete structure

7.3.2 Pump characteristics for Clearwater pumping

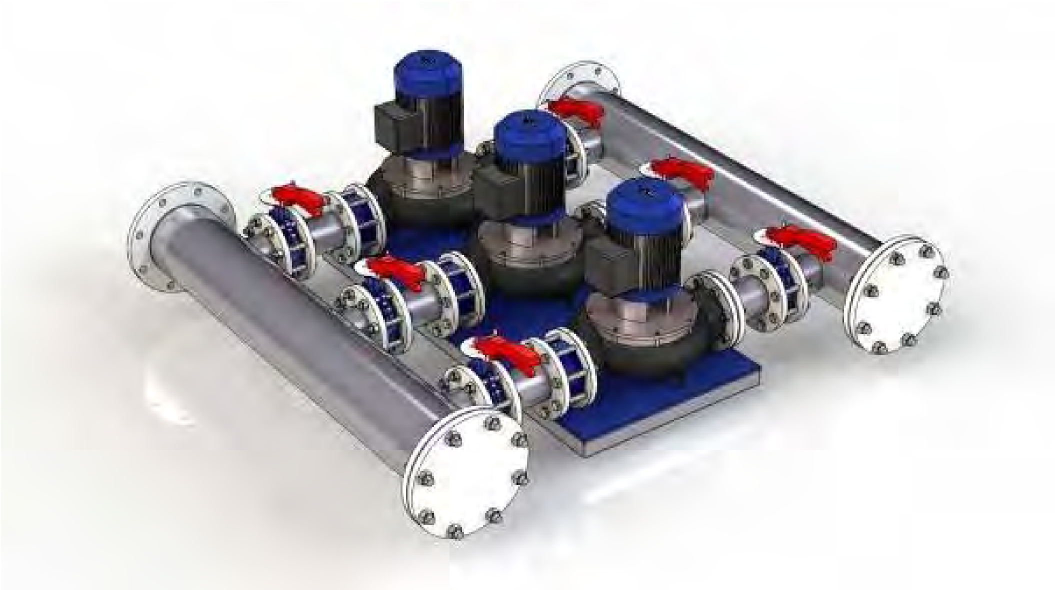
105. The capacity of the required booster pumps need to be designed to meet peak day and peak hourly demand. Two phases of the project have been planned and the two scenarios hydraulically modelled. Based on the EPAnet model results, the capacity of the booster pump can be described as follows:

- **Phase 1** (Year 2020-2025) :
  Booster pump at WTP requires capacity of 3,192 m³/h or 887 l/s, and total head 25m (26m for pump selection)
- **Phase 2** (Year 2026-2040):
  Booster pump at WTP requires total capacity of 5,159 m³/h or 1,433 l/s, and total head 50m (51m for the pump selection)

106. Vertical in-line centrifugal pumps are the type of pump commonly used for distributing potable water to consumers in Cambodia. A packaged booster system is proposed for use at the WTP. The package booster system is factory designed for optimized pumping and simplified installation (smaller footprint, easy to install, lower installation cost, one electrical connection, no shaft/coupling alignments/adjustments, and integrated variable speed control). Figure 19. shows a typical arrangement of such booster system. The benefits of a packaged booster system are :

- Saves Energy Costs: Efficient cascade control, application optimized software and pumps in the industry
- Single Source Responsibility: One manufacturer for pumps, motors, drives & control
- Plug-and-Pump: Easy to install and commission
- Easy to Operate: Large, clear, user friendly & advanced controls interface
- Reduce floor space: Space-saving complete solution

Figure 19. Typical arrangement of Packaged Booster system



107. Packaged booster systems usually consist of two to six identical vertical in-line multi stage pumps connected in parallel and mounted on a common base frame, provided with a control cabinet and all the necessary fittings. The pumps of the booster system can be removed without interfering with the pipework on either side of the manifolds.

108. It is possible to let one or more pumps function as standby pumps. A booster system with, for example, four pumps - one having the status of standby pump, will run like a booster system with three pumps. If a pump is stopped due to a fault, the standby pump will be cut in. This function ensures that the system can maintain the rated performance even if one of the pumps is stopped due to a fault. The status as standby pump alternates between all pumps of the same type, to ensure equal usage.

109. The proposed packaged booster system for pumping clearwater from the reservoir to distribution will have one common controller for all of the pumps. Many manufacturers make vertical inline centrifugal pumps and can provide packages.

110. Horizontal split case centrifugal pump has been considered for use in pumping clearwater to the network. However they require a larger footprint for installation and require regular inspection of the alignment of the horizontal shafts and associated bearings. Vertical in-line centrifugal pumps require a smaller footprint for installation and do not require constant inspection of alignment, having its drive shaft from the motor to the pump directly coupled removing the risk of mis-alignment even after a long period of operation.

7.3.2.1 Pumping Clearwater to the Network for 2025

Pump capacity

111. The capacity of the pump selected should consider both the peak day factor (for seasonal demand variation) and the peak hourly factor, which varies depending on the type of service area (for example, peak hourly factor for urban area is normally less than peak factor for rural area). For Battambang the daily peak factor selected is 1.2 and peak hourly factor has been calculated as 1.61 for 2025. Therefore, the combined required capacity of the booster pumps

for 2025 is 887 l/s. Vertical in-line single stage pumps are selected, with 3 duty pumps plus 1 standby, each with capacity 296 l/s.

Pumping head

112. The total head depends on design criteria such as head loss along pipe, elevation difference from WTP to end of pipe network, and the target pressure requirement. The total head for the booster pump at the WTP is estimated at 26m.

7.3.2.2 Pumping Clearwater to the Network for 2040

Pump capacity

113. As above, the selected daily peak factor is 1.2 and peak hourly factor calculated as 1.56. Therefore, the total required capacity of the booster pumps for 2040 is 1,433 l/s. In 2025, all 4 pumps will be replaced by 6 pumps (5 duty, 1 standby, 287 l/s each) to meet the 2040 demand. Table 30. summarises clear water pumping requirements for 2025 and 2040.

Pumping head

114. The total head depends on design criteria such as head loss along pipe, elevation difference from WTP to end of pipe network, and the target pressure requirement. The total head for booster pump at the WTP is estimated at 51m.

7.3.2.3 Summary of pumps for clearwater pumping station

Table 30. Summary of pump parameters at WTP

Description	Year 2025	Year 2040
Total pump capacity	887 l/s	1,433 l/s
Total pump head	Ht = 26 m	Ht = 51 m
Pump type	Vertical in-line single stage	Vertical in-line single stage
Number of pump	3 duty and 1 standby, with capacity 296 l/s each pump	5 duty and 1 standby, and capacity 287 l/s each pump
Total power required	330kw	1125kw
Transformer 22KV/0.4KV, 3 phases 4 wire, Outdoor Type	800kVA	1250kVA
Generator 400 V/230 V, 3 phases 4 wire, 50Hz, Silent Type	600kVA	1250kVA (Replace 4 sets of pump year 2025)
MV Pole Concrete for Incoming Overhead Line 22KV	2-pcs ; Voltage=22KV ; AL-150mm2	

Figure 20. and Figure 21. below provide a synoptic view of required flows in different stages of the proposed system, both in 2025 and 2040.

Figure 20. Synoptic of flows in 2025

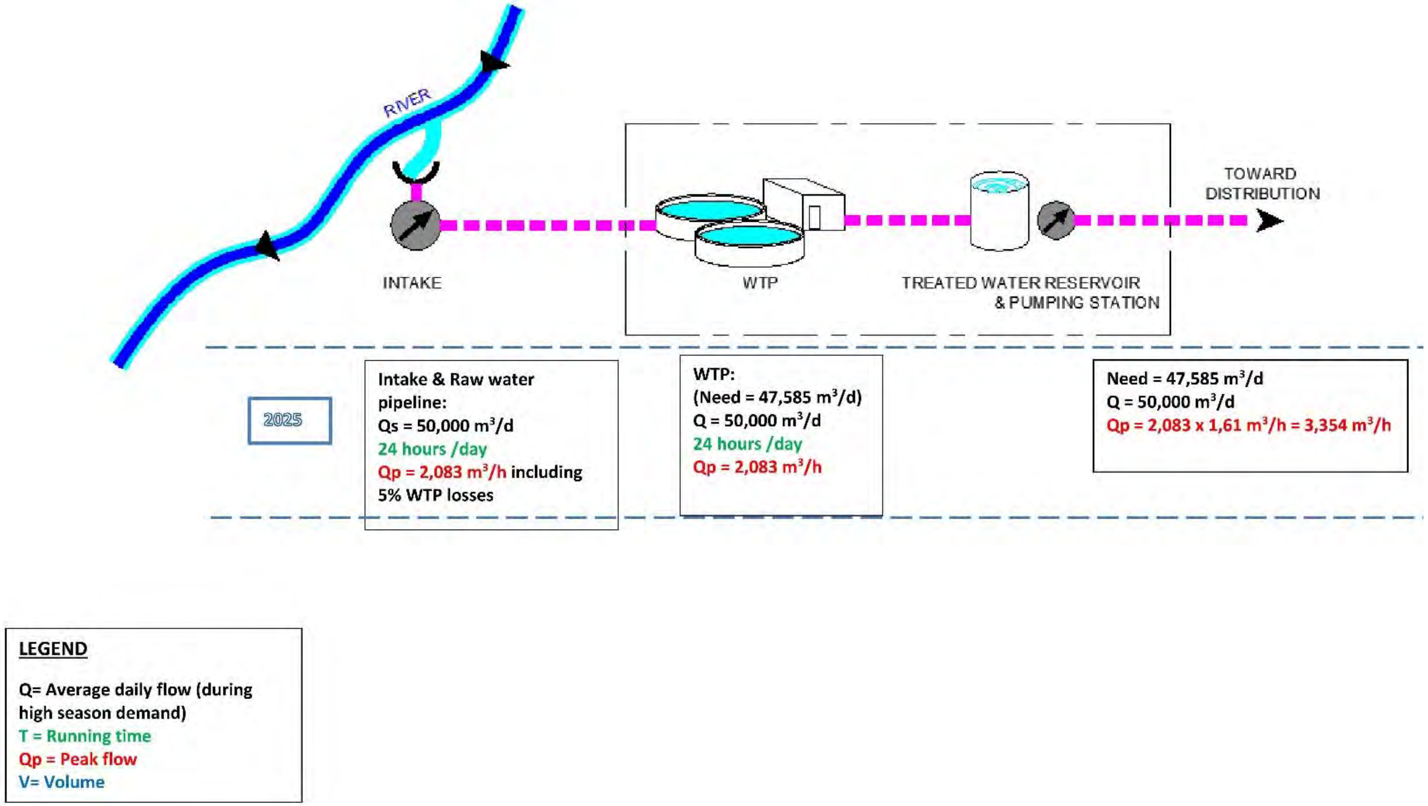
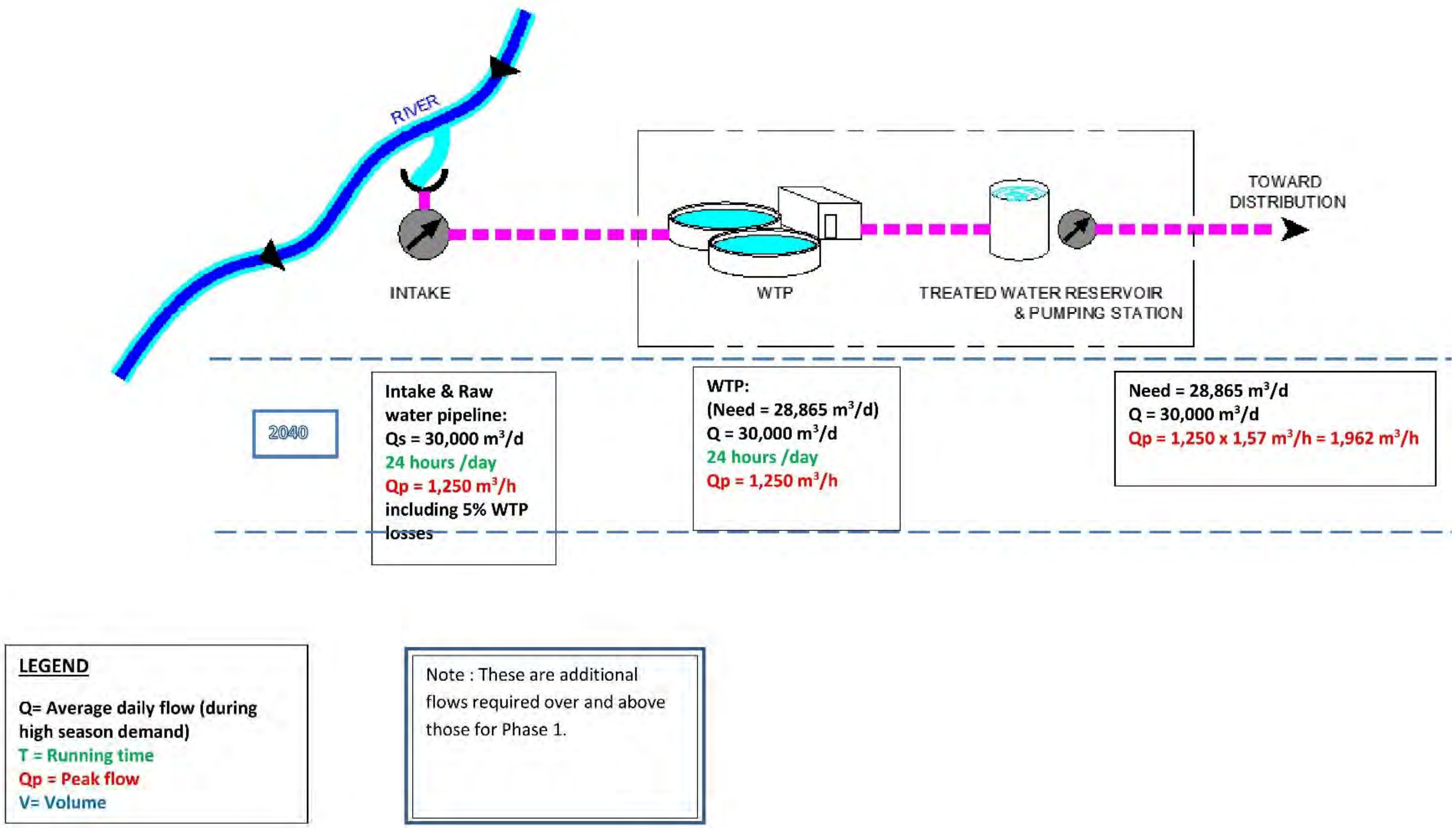


Figure 21. Synoptic of flows in 2040



7.3.3 Distribution network preliminary design

115. EPAnet software has been used to design diameters of pipelines. Two distribution zones ("blocks") have been modelled for the distribution, as shown in Figure 22.

116. Block "A" covers the main expansion area in north-west part of the town which includes five sangkat (Ou Mal, Ou Cha, Chamkar Samroung, Slakaet, Kdol Donteav and two communes (Chrey and Ou Taki) in Thmakoul district.

117. Block "B" covers the expansion area in south-east of the Battambang town which includes one Sangkat (Vaot Kor) in Battambang town and four communes (Vaot Tamuem, Ou Dambang Mouy, Ou Dambang Pir, Anlong Vil) in Sangker district. Block "A" is more densely populated than block "B".

118. As described in Section 6.5.1, two options for the distribution arrangement have been modelled, to investigate the optimization of the capital and operation costs. The results of the model runs are described below, and the comparisons of both options summarized.

Model Option 1- Distribution by Booster Pump and Elevated Water Tower;

- Total flow from WTP in 2025 is 3,192 m^3/h
- Total flow from WTP in 2040 is 5,159 m^3/h

Pipeline sizes and supply pressure by block as determined by the model run are described in Table 31. and Table 32. The results are summarized in Table 33. The detailed results of running EPAnet are enclosed in Annex 3.

Table 31. Pipeline requirement in block "A" by Option 1

No.	Results			
I.	**Pipe Diameter (mm) (*1)**		**Pressure Nominal (PN)**	**Pipe Length (m)**
1	63		PE100, PN 10	6,118
2	75		PE100, PN 10	2,041
3	90		PE100, PN 10	4,050
4	110		PE100, PN 10	1,950
5	125		PE100, PN 8	2,548
6	160		PE100, PN 8	8,429
7	200		PE100, PN 8	8,282
8	225		PE100, PN 8	442
9	250		PE100, PN 8	3,616
10	280		PE100, PN 8	7,640
11	400		PE100, PN 8	6,730
12	450		PE100, PN 8	5,990
13	500		PE100, PN 8	20,938
14	560		PE100, PN 8	264
15	630		PE100, PN 8	23
16	710		PE100, PN 8	420
Total pipe length (m)				**79,481**
II. Total Pump Capacity in 2025			**Flow (m^3/h)**	**Head (m)**
18	At Water Treatment Plant		3,200	33
III. Total Pump Capacity in 2040			**Flow (m^3/h)**	**Head (m)**
19	At Water Treatment Plant		5,138	50
III. Water Tower			**Volumne (m^3)**	**Height bottom of the Tank (m)**
20	Water Tower in Block A (Supply for 1 hour with hourly peak) (*2)		350	29

Note: (*) Pipe diameters follow pipe catalogue
(**) The proposed water tower in block A is located at the end of Ka Kou Village, where the ground level is 13.3masl

Table 32. Pipeline requirement in block "B" by Option 1

No.	Results		
I.	**Pipe Diameter (mm) (*1)**	**Pressure Nominal (PN)**	**Pipe Length (m)**
1	63	PE100, PN 10	2,479
2	75	PE100, PN 10	1,013
3	90	PE100, PN 10	2,973
4	110	PE100, PN 10	2,504
5	125	PE100, PN 8	499
6	160	PE100, PN 8	483
7	225	PE100, PN 8	1,756
8	560	PE100, PN 8	9,346
9	630	PE100, PN 8	8,538
Total pipe length (m)			**29,591**
II. Water Tower		**Volumne (m^3)**	**Height bottom of the Tank (m)**
10	water Tower In Block B (Supply for 1 hour with hourly peak) (*2)	1,100	29

Note: (1) Pipe diameter here follow pipe catalogue

(2) The water tower in block B is located in Ou Khcheay village, where the ground level is 15.53masl

Table 33. Summary for Pipeline Network - Option 1

No.	Results		
I.	Pipe Diameter (mm) (*1)	Pressure Nominal (PN)	Pipe Length (m)
1	63	PE100, PN 10	8,597
2	75	PE100, PN 10	3,054
3	90	PE100, PN 10	7,023
4	110	PE100, PN 10	4,454
5	125	PE100, PN 8	3,047
6	160	PE100, PN 8	8,912
7	200	PE100, PN 8	8,282
8	225	PE100, PN 8	2,198
9	250	PE100, PN 8	3,616
10	280	PE100, PN 8	7,640
11	400	PE100, PN 8	6,730
12	450	PE100, PN 8	5,990
13	500	PE100, PN 8	20,938
14	560	PE100, PN 8	9,610
15	630	PE100, PN 8	8,561
16	710	PE100, PN 8	420
Total pipe length (m)			**109,072**
II. Total Pump Capacity in 2025		**Flow (m^3/h)**	**Head (m)**
18	At Water Treatment Plant	2,667	33
III. Total Pump Capacity in 2040		**Flow (m^3/h)**	**Head (m)**
19	At Water Treatment Plant	5,159	50
III. Water Tower		**Volumne (m^3)**	**Height bottom of the Tank (m)**
20	Water Tower in Block A (Supply for 1 hour with hourly peak) (*2)	350	29
21	water Tower In Block B (Supply for 1 hour with hourly peak) (*3)	1,100	29

Note: (1) Pipe diameter follows pipe catalogue
(2) The water tower in block A is located in end of Ka Kou Village, where the ground level is 13.3masl
(3) The water tower in block B is located in Ou Khcheay village, where the ground level is 15.53masl

119. The total pipe length in the model run for year 2025 is approximately 109km, along the existing road sides in the proposed expansion area. In addition to the 109km, supply to houses 2 or 3 rows back from the main roads with smaller diameter pipe (63-90mm) account for 11km. In Year 2040 further pipelines with total length 120km (pipe diameter 63-560mm) will be needed to increase the served population up to 90-100% of the coverage area.

120. On main roads with professional concrete or tarmac seal, mains have been modelled on both sides of the road to minimize road crossings. On ordinary roads, there is a distribution main on one side only.

Figure 22. Layout of proposed pipeline by block

Figure 23. Layout of pipeline, and location of water tower, and air valve for Option 1 year 2025

Figure 24. Layout of pipeline, and location of water tower, and air valve for Option 1 year 2040

Model Option 2- Distributed by Booster Pump alone (no water tower):

- Total flow from WTP in 2025 is 3,192 m^3/h
- Total flow from WTP in 2040 is 5,159 m^3/h

Pipeline sizes and supply pressure by block as determined by the model run are described in Table 34. and Table 35. The results are summarized in Table 36. All networks are to be constructed after 2026 to maintain proper water pressure and supply enough water to the downtown areas and the expanded areas as planned by the Battambang Waterworks. The detailed results of running EPAnet are enclosed in Annex 3.

Table 34. Pipeline requirement in block "A" by Option 2

No .	Results		
I.	Pipe Diameter (mm) (*1)	Pressure Nominal (PN)	Pipe Length (m)
1	63	PE100, PN 10	4,910
2	75	PE100, PN 10	1,325
3	90	PE100, PN 10	4,763
4	110	PE100, PN 10	1,438
5	125	PE100, PN 8	3,126
6	160	PE100, PN 8	11,003
7	180	PE100, PN 8	889
8	200	PE100, PN 8	6,495
9	225	PE100, PN 8	2,609
10	250	PE100, PN 8	3,829
11	315	PE100, PN 8	4,061
12	400	PE100, PN 8	3,087
13	450	PE100, PN 8	9,564
14	500	PE100, PN 8	12,641
15	560	PE100, PN 8	8,508
16	710	PE100, PN 8	441
Total pipe length (m)			**78,689**

Note: (*) Pipe diameter here follow pipe catalogue

Table 35. Pipeline requirement in block "B" by Option 2

No.	Results		
I.	Pipe Diameter (mm) (*1)	Pressure Nominal (PN)	Pipe Length (m)
1	63	PE100, PN 10	3,192
2	75	PE100, PN 10	513
3	90	PE100, PN 10	1,975
4	110	PE100, PN 10	4,002
5	125	PE100, PN 8	499
6	225	PE100, PN 8	2,239
7	560	PE100, PN 8	9,917
8	630	PE100, PN 8	7,955
Total pipe length (m)			**30,292**

Note: (*) Pipe diameter here follow pipe catalogue

Table 36. Summaries for Pipeline Network by Option 2

No.	Summaries Results for Pipe Network		
I.	Pipe Diameter (mm) (*1)	Pressure Nominal (PN)	Pipe Length (m)
1	63	PE100, PN 10	8,102
2	75	PE100, PN 10	1,838
3	90	PE100, PN 10	6,738
4	110	PE100, PN 10	5,440
5	125	PE100, PN 8	3,625
6	160	PE100, PN 8	11,003
7	180	PE100, PN 8	889
8	200	PE100, PN 8	6,495
9	225	PE100, PN 8	4,848
10	250	PE100, PN 8	3,829
11	315	PE100, PN 8	4,061
12	400	PE100, PN 8	3,087
13	450	PE100, PN 8	9,564
14	500	PE100, PN 8	12,641
15	560	PE100, PN 8	18,425
16	630	PE100, PN 8	7,955
17	710	PE100, PN 8	441
Total pipe length (m)			**108,981**
II. Total Pump Capacity in 2025		**Flow (m^3/h)**	**Head (m)**
18	At Water Treatment Plant	3,200	25
III. Total Pump Capacity in 2040		**Flow (m^3/h)**	**Head (m)**
19	At Water Treatment Plant	5,159	50

Note: (*) Pipe diameter here follow pipe catalogue

Figure 25. Layout of pipeline, and location of water tower, and air valve for Option 2 year 2025

Figure 26. Layout of pipeline, and location of water tower, and air valve for Option 2 year 2040

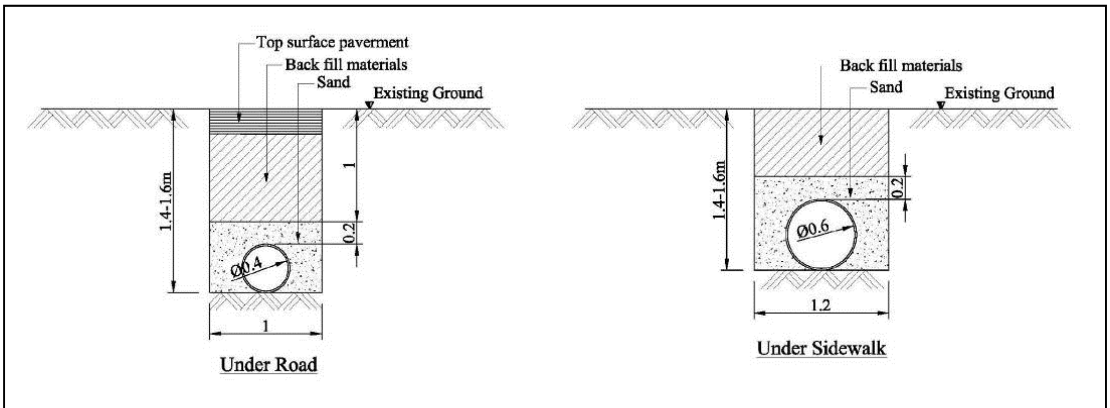
121. The comparison of the results of Option 1 and Option 2 in Table 37. below reveals that Option 2 is more economical than Option 1. The Option one cost more with two water towers of US$ 1.7 million and more electricity to maintain higher water head at the WTP in Phase 1. Therefore, for the purposes of cost estimation for the proposed subproject in 2025 and 2040, Option 2 has been considered (direct pumping from the booster pump at WTP).

Table 37. Summary of modelling results of 2 Options

Items	Option 1 with Water Towers	Option 2 without Water Towers	Remarks on Option 1
Pipe size			
Block A	63-630	63-630	
Block B	63-560	63-560	
Pipe length			
Block A	80.12 Km	80.12 Km	
Block B	28.90 Km	28.90 Km	
Booster Pump	24 hours running	24 hours running	
Water Tower (WT)			
Block A	V= 300 m^3 h= 25 m	no	US$ 0.5 Million for 300m^3 WT
Block B	V= 910 m^3 h= 25 m	no	US$ 1.2 Million for 910m^3 WT
Total head at WTP			
In 2025	33 m	25 m	More electricity consumption for the Phase 1
In 2040	50 m	50 m	
Pressure at end pipeline			
In 2025	17 – 18 m	8m	
In 2040	7 – 9 m	8m	

122. The typical detail of pipe trench and backfill material for the water transmission pipeline and main distribution line is shown in Figure 27.

Figure 27. Typical detail for pipe trench and backfill



7.4 Summary of the Proposed Facilities

123. The project is proposed to be implemented in 2 Phases: Phase 1 shall be implemented in 2018-2020, with an estimated construction period of 2-2.5 years. The facilities in Phase 1, described in Table 38. will supply safe water with sufficient capacity to serve 85-100% of total people within the expansion area on the edges of Battambang, plus some communes in Sangker and Thmakol districts. Supplementary water will also be available to meet the demand in the downtown area from 2020 to 2025 to complement the existing WTPs.

124. Phase 2 is planned to provide safe water for the whole service area for the target period 2026-2040. The facilities in Phase 2 are described in 0and will serve 90-100% of total population over the whole service area as planned by Battambang Waterworks.

Table 38. Summary of Proposed Facilities in Phase 1 (Year 2020-2025)

1) Water source	Sangker River; HWL=15.57 m, LWL=4.81 m
2) Intake Capacity	50,000 m^3/d
Pump at Intake	• Total pump capacity: 2,083 m^3/h or 579 l/s • Four pumps (vertical turbine pump) will be installed: 3 duty and 1 standby • Total head 35 m
3) Raw water transmission pipe (DCI)	Diameter : 710 mm Length : 5 km
4) WTP Capacity	50,000 m^3/d
Treatment facilities	• Receiving well: Reinforced concrete with internal dimensions: (2.00 m width x 5.25 m length x 4.45 m depth) x 2 units • Rapid Mixing: Reinforced Concrete Structure with gravitational force mixing using a weir with internal dimensions: (2.00 m width X 3.00 m length X 3.81 m depth) x 2 units • Flocculation: Vertical channel baffled flocculator with internal dimensions: (7.10 m width x 5.90 m length x 4.70 m height) x 8 units • Sedimentation: Reinforced Concrete Structure, Horizontal-Flow Sedimentation Basin with internal dimension: (7.10 m width X 30.00 m length x 4.12m average depth) x 8 units; Surface Loading: Q/A=20.0 mm/min (15-30 mm/min); Mean Velocity V=0.14 m/min (below 0.40 m/min) • Filtration: - Reinforced Concrete Structure - Internal Dimension: (3.50 m width X 9.10 m length) x 12 units - Underdrain System: Porous Filter Bed Method - Filtration Rate: V=126.6 m/day (120-150 m/day) - Flow Control: Lower Part Control Method - Backwash Method: Simultaneous Backwash Method by Air and Water • Clearwater Reservoir: Reinforced Concrete Structure using Flat Slab Structure; Effective Volume: 12,800 m3, Effective Water Depth: H= 4.30, Detention Time: T= 3 hours; and Internal Dimension: (16m width x 40m length x 5m height) x 4 units
Booster pump at WTP	• Total pump capacity: 3,192 m3/h or 887 l/s • Four booster pump will be installed: 3 on-duty and 1 standby • Total head 26m (confirmed following pump selection)
5) Distribution Networks	
Pipe network	120 km including 109 km of mains and 11km for connection of secondary and tertiary road houses
6) Construction plan	2018-2020

Table 39. Summary of Proposed Facilities in Phase 2 (Year 2026-2040)

1) Water source	Sangker river; HWL=15.57 m, LWL=4.81 m
2) Intake Capacity	30,000 m^3/d
Pump at Intake	• Total pump capacity: 1,250 m3/h or 347 l/s • Five pumps will be installed: 4 in duty and 1 standby (including pumps for 2025) • Total head : 38 m
3) Raw water transmission pipe (DCI)	Diameter : 560 mm Length : 5 km (in parallel of 2025 pipe)
4) WTP Capacity	30,000 m^3/d
Treatment facilities	• Rapid Mixing • Flocculation • Sedimentation • Filtration • Clearwater reservoir • Disinfection
Booster pump at WTP	• Total pump capacity: 5,159 m3/h or 1,433 l/s • Two further duty pumps to be installed at WTP (6 total: 5 duty and 1 standby • Total head 51m (following pump selection)
5) Distribution Network	
Pipe networks	120 km of additional network to cover infill areas
6) Construction plan	2028-2030

8 Preliminary Cost Estimates

125. The preliminary cost estimates for Phases 1 and 2 of the proposed subproject are presented in Table 41. and Table 42. , respectively. The unit rate for the WTP has been selected from available cost data from previous projects in Cambodia. Capital costs for WTP's rise with capacity with a logarithmic function. Ideally we would have ten or more recent completed WTP costs, but much data from the pre-2010 period has been lost. Recent previous water supply projects in Cambodia are Battambang and Kampong Cham (JICA, estimated), Mondul Kiri (JIAF, US$ 600/m^3/d), Rattanakiri (private, US$ 215/m^3/d) and Siem Reap (private). Data from Mondulkiri was not relevant as it was an imported package plant, data from Ratanakiri not relevant as the private plant was to a much lower standard than GoC plants, and Siem Reap cost data is not yet available as the contract has not been awarded yet. This left Battambang and Kampong Cham for reference as shown in Table 40. below. Thus the conservative unit rate cost adopted is US$ 450/m^3/d for the WTP including pump, yard pipes and other associated facilities, including all labour.

Table 40. Historical cost data for WTP construction

Town	WTP capacity (m^3/d)	Capital Cost ($)	Unit cost ($/(m^3/d))
Battambang	22,000	10,000,000	456
Kampong Cham	11,500	8,104,000	705

126. Detailed engineering works and the project management and construction supervision (including O&M training) are not included in this unit rate as the engineering works are to be included in the Design-Build contract, and the project management and construction supervision works depend on the scope of works[19] and are proposed under the project implementation and assistant (PIAC) contract.

[19] approximately 8% of the project cost according to ADB at the DFR Workshop)

Table 41. Preliminary Cost Estimates of Phase 1 (Year 2020-2025)

No.	Descripton	Unit	Quantity	Unit rate	Cost
1	New WTP[20]	m^3/d	50,000	450	22,500,000
	Pumps	ea	4	25,000	100,000
	Electrical & controls	LS	1	463,000	463,000
2	New intake[21]:				
	Structure	LS	1	730,000	730,000
	Pumps	ea	4	25,000	100,000
	Electrical	LS	1	151,000	151,000
3	Raw water pipeline 710mm DCI	km	5	284,000	1,420,000
4	Distribution Pipelines	km	120	50,108	6,013,000
6	Office, pump station, chemical building		1	250,000	250,000
	Subtotal				**31,727,000**
7	Contingency 10%				3,172,700
	Total				**34,899,700**

Table 42. Preliminary Cost estimates of Phase 2 (Year 2026-2040)

No.	Descripton	Unit	Qty	Unit rate	Cost
1	New WTP extension	m^3/d	30,000	450	13,500,000
	Pumps	ea	2	25,000	50,000
	Electrical & controls	LS	1	30,000	30,000
2	New intake				
	Pumps	ea	2	25,000	50,000
	Electrical	LS	1	151,000	151,000
3	Raw water pipeline 560mm DCI	km	5	200,000	1,000,000
4	Distribution Pipelines	km	120	45,000	5,400,000
	Subtotal				**20,181,000**
7	Contingency 10%				2,018,100
	Total				**22,199,100**

[20] WTP structure will be constructed for capacity in 2025 only, and reserved space for additional WTP capacity in 2040.
[21] Intake structure will be constructed for 2040, but pumps installed for 2025 capacity only and space reserved for further pumps to 2040.

127. Based on the current facility operations, preliminarily estimates for monthly O&M costs are presented in the Table 43. for Year 2025 and 2040.

Table 43. Estimation monthly O&M cost

No.	Description	unit	Production Cost/m^3		Estimated Operating Cost/month	
			Riel	US$	2025	2040
1	Electrical	Kwh	313.74	0.078	102,464	170,774
2	Chemical		139.03	0.035		
	Powder chlorine	kg	16.79	0.004	5,483	9,139
	Gas Chlorine	kg	12.11	0.003	3,955	6,591
	lime	kg	19.45	0.005	6,352	10,587
	Alum	kg	37.66	0.009	12,299	20,499
	PAC	kg	53.02	0.013	17,315	28,859
3	Labour	month	209.73	0.052	68,496	114,160
4	Maintenance	month	268.36	0.067	87,644	146,073
	Total Monthly Production Cost, US$				**304,011**	**506,685**

9 Conclusions

128. As summarized in Table 38. , the Battambang water supply subproject is proposed to be included under PWSSP with a conventional water treatment plant and a design capacity of 50,000m^3/d to serve 85-100% of urban population (275,453 people) in the expanded service area of 116km^2 by 2025.

129. A water intake with a capacity of 50,000m^3/d to withdraw raw water from the Sangker River and a 5 km transmission line to the WTP, with 120km distribution network are also included. The estimated cost is US$ 34.9 million including contingency.

130. There are no permits required, except a water source allocation MOU, to be secured from the MoWRM to have the raw water available from the Sangker River in an amount up to 50,000m^3/d for 2025.

131. Land commitment for the WTP is confirmed but commitment for the water intake is yet to be secured.

132. The construction works of the proposed project will cause various impacts on the surroundings. Most of these impacts are short-term, reversible, localized and are easily mitigated. Negative impacts during construction from pipelaying works will include possible siltation and increased turbidity of nearby receiving bodies of water, generation of construction waste, construction nuisances (i.e. noise and vibration), and traffic congestion. An environmental management plan is to be developed to guide the mitigation of the impacts. Coordination with the contractor and the local traffic management office should be done prior to start of construction, particularly on network extension. A set of environmental code of practices (ECOPs) may also be developed to address common negative impacts of construction works.

133. No specific construction problems with cost implications are expected from existing ground conditions or access.

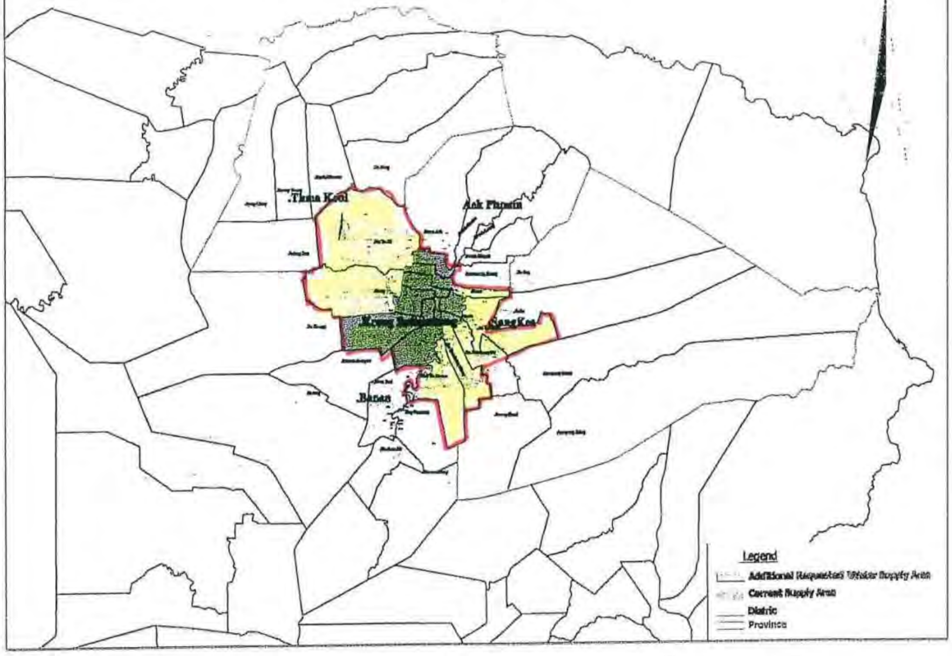
134. Additional studies required before launching a DB contract for the WTP include geotechnical, hydrology, allocation of supplementary water sources during drought periods, D&B technical performance specifications.

135. Detailed topography survey, and detailed design of the distribution network for the separate Network (Works) Contract will be required.

136. All additional surveys and studies mentioned above might be included under the PIAC scope.

Annex 1: Service area confirmation

ផ្ទៃតំបន់សេវាកម្មផ្គត់ផ្គង់ទឹកស្អាត អង្គភាពរដ្ឋាករទឹកខេត្តបាត់ដំបង



តំបន់ផ្ទៃសេវាកម្មផ្គត់ផ្គង់ទឹកស្អាត របស់អង្គភាពរដ្ឋាករទឹកបាត់ដំបង សម្រាប់ក្រុងបាត់ដំបង ខេត្តបាត់ដំបង ៖

មាន១០ សង្កាត់ ត្រូវជា ៦២ ភូមិ (យកទាំងអស់ លើកលែងតែភូមិខ្សាច់ពោយ នៃសង្កាត់វត្តគរ)៖

១/ សង្កាត់ទួលតាឯក , ២/សង្កាត់ព្រែកព្រះស្តេច, ៣/ សង្កាត់រតនៈ ,៤/ សង្កាត់ចំការសំរោង, ៥/ សង្កាត់ស្លាកែត, ៦/ សង្កាត់ក្តុលដូនទាវ , ៧ / សង្កាត់ឱម៉ាល់ , ៨/សង្កាត់វត្តគរ, ៩ /សង្កាត់អូរចារ និង១០/ សង្កាត់ស្វាយប៉ោ។

បាត់ដំបង,ថ្ងៃទី ... ខែ សីហា ឆ្នាំ២០១៦

អង្គភាពរដ្ឋាករទឹកបាត់ដំបង

ឡុង ឈុនសៅរិទ្ធ

លេខ ៥៣៥/១៦

បានឃើញ និងឯកភាព

បាត់ដំបង,ថ្ងៃទី២៥ ខែសីហា ឆ្នាំ២០១៦

អភិបាលក្រុងបាត់ដំបង

សឿងឯម_ហួនស្មី

លេខ ៩៥១ មឧស

បានឃើញ និងឯកភាព

បាត់ដំបង,ថ្ងៃទី ២៥ ខែ សីហា ឆ្នាំ២០១៦

មន្ទីរឧស្សាហកម្ម និងសិប្បកម្មខេត្ត

ប្រធាន

សំបូរ_ឌី

Annex 2: Pump and Electrical Calculations

1. Battambang Sub-project - Proposed Pump and Electrical Design Capacity

A. Intake Pump

i. Capacity of intake pump in 2025

The pump design should include water loss during water treatment plant processing that is assumed approximately 5% of the total water demand (Q_{total}). Therefore, the water flow rate of pump intake was determined as below:

$$Q_p = Q_{total} \times 1.05 \qquad \textbf{Eq. 1}$$

Where: Q_{total} = 47585m^3/day (***in Year 2025***)

Therefore, the total capacity of pump intake is **Qp = 49,964 say 50,000m^3/day**.

And the number of pump unit was determined (**3**) duty pumps and (**1**) pump stand by with capacity of each pump **q_p = 193 LPS**.

- **Diameter of suction pipe and discharge pipe**

We have the equation:

$$\phi = \sqrt{\frac{q_p \times 4}{v \times \pi}} \qquad \textbf{Eq. 2}$$

Where:
Φ : pipe diameter, mm
q_p : pump intake capacity of each pump, 0.193m^3/s
V : velocity in pipe (suction pipe and discharge pipe), m/s
π = 3.14

Note: The velocity of suction pipe varies within **1.0m/s** to **1.6m/s**, and the velocity of discharge pipe is between **1.5m/s** to **3m/s**.

- **Diameter of suction pipe**

According to the **Eq.2**, the diameter of suction pipe is approximately **$\Phi_{suction}$ = 400mm** with **V_s = 1.54m/s**.

- **Diameter of discharge pipe**

According to the **Eq.2**, the diameter of discharge pipe is approximately **Φ_d =350mm** with velocity **V_d = 2.01m/s**.

The main pipe of raw water transmission is proposed **Φ_{main}=630mm** of diameter with **V_{main} = 1.79m/s**.

- **Pump head (H_t)**

The pumping head is the sum of the static head, entrance head, friction loss in the suction pipe, discharge loss, and other band loss as emphasizing in equation below:

$$H_t = H_{static} + h_{ent} + h_{fs} + h_{fd} + \Sigma h_m \qquad \textbf{Eq. 3}$$

Where:

H_t : total pump head, m

H_{static} : static head, m

h_{ent} : head loss of entrance bell, m

h_{fs} : friction loss of suction pipe, m

h_{fd} : friction loss of discharge pipe, m

h_m : friction loss by minors, m

- **Friction loss of suction pipe**

We have:

$$h_{f,s} = 10.7 \times \left(\frac{q_p}{C}\right)^{1.85} \times D^{-4.87} \times L_s \qquad \textbf{Eq. 4}$$

Where:

q_p : the capacity of each pump, 0.193m³/s

C : the Hazen-Williams factor, **120**

D : the diameter of the suction pipe, 400mm

Ls : the length of the suction pipe (column pipe), 13m

Therefore, The friction suction loss is $\mathbf{h_{fs} = 0.082m}$.

- **Velocity suction head ($h_{v,s}$)**

We have:

$$h_{v,s} = \alpha \frac{v^2}{2g} \qquad \textbf{Eq. 5}$$

Where:

$h_{v,s}$: the velocity headloss, m
V : the velocity in suction pipe, 1.54m/s.
g : the earth's gravity, 9.81m2/s
α : velocity factor was assume 1

Therefore, the velocity suction head loss is $\mathbf{h_{vs} = 0.120m}$.

- **Entrance head loss (h_{ent})**

We have:

$$h_{ent} = k \times h_{v,s} \qquad \textbf{Eq. 6}$$

Where: For bell mouth, k = 0.05

Therefore, the entrance headloss is h_{ent} = 0.006m

- **Gate valve headloss** ($h_{g,v}$)

We have:

$$h_{g,v} = k \times h_{v,s} \qquad \textbf{Eq. 7}$$

Where: For the gate valve, **k = 0.2**

Therefore, the gate valve headloss is $h_{g,v}$ = 0.024m

- **Friction loss of discharge pipe (h_{fd})**

We have:

$$h_{f,d} = 10.7 \times \left(\frac{q_p}{C}\right)^{1.85} \times D^{-4.87} \times L_d \quad \textbf{Eq. 8}$$

Where:

q_p : the capacity of each pump, $0.193m^3/s$
C : the Hazen-Williams factor, 120
D : the diameter of the discharge pipe, 350mm
L_d : the length of discharge pipe (transmission pipe), 1.5m

Therefore, the friction discharge headloss is $h_{f,d}$ = 0.047m

- **Friction loss of transmission pipe line (h_{ft})**

We have:

$$h_{ft} = 10.7 \times \left(\frac{q_p}{C}\right)^{1.85} \times D^{-4.87} \times L_t \quad \textbf{Eq. 9}$$

Where:

q_p : the capacity of each pump, $0.193m^3/s$
C : the Hazen-Williams factor, 120
D : the diameter of the main pipe, DN710mm (642.2mm)
L_t : the length of discharge pipe (transmission pipe), 5,000m

Therefore, the friction discharge headloss is **$h_{f,d}$ = 15.856m**.

- **Velocity discharge head (h_{vd})**

We have:

$$h_{v,d} = \alpha \frac{V_d^2}{2g} \quad \textbf{Eq. 10}$$

Where:

$h_{v,d}$: the velocity headloss, m
V_d : the velocity in discharge pipe, 2.01m/s
g : the earth's gravity, $9.81m^2/s$
α : velocity factor was assume 1

Therefore, the velocity discharge headloss is **h_{vd} = 0.205m**.

- **Gate valve headloss at discharge**

We have:

$$h_{g,v} = k \times h_{v,d} \quad \textbf{Eq. 11}$$

Where: For the gate valve, **k = 0.2**

Therefore, the gate valve headloss is **h_{gv} = 0.041m**.

- **Bends headloss, h_b**

We have:

$$h_b = k \times h_{v,d} \quad \textbf{Eq. 12}$$

Where: We supposed that there are two bends with k = 0.25

Therefore, the bends headloss is h_b = 0.103m.

- **Minor loss check valve (h_{chv})**

We have:

$$h_{chv} = k \times h_{v,d} \qquad \textbf{Eq. 13}$$

Where: For check valve, k = 2.20

Therefore, the check valve headloss is **h_{chv} = 0.451m**.

- **The total dynamic head (Ht)**

We have:

$H_t = H_{static} + h_{ent} + h_{fs} + h_{fd} + \Sigma h_m$

Where: the static head is h_{static} = 16.68m.

⇨ H_t = 16.68 + 0.006 + 0.082 + 15.903 + 0.945 = 33.61m.

Note: there are more bend along main transmission pipe line, we also used the model to run and the excel sheet found that the total dynamic head is approximately 34.16m say 35m.

Therefore, the total dynamic head is H_t = 35.0m.

- **Type of pump**

A specialized centrifugal pumps designed to move water from a well or reservoir that is deep underground. Also known a deep well turbine pump or a line shaft turbine pump, it is one of two main types of turbine pumps, which are vertical turbine pumps and submersible turbine pumps.

- **Submersible pumps** have the electric motor located underwater at the bottom of the pump.
- **Vertical turbine pumps** have the motor located above ground, connected via a long vertical shaft to impellers at the bottom of the pump.

We have: H_t = 35m and the type of pump suitable for an installation in a sump with a vertical distance more than 10m just below the pump is a vertical turbine pump.

Therefore, the pump type proposed is vertical turbine pump.

ii. Capacity intake pump for 2040

From the Same formula of calculation mentioned above, the pump capacity in year 2040 can be determined as shown below with the Q_t = 28,865 m^3/day. Therefore, the actual pump intake capacity is **30,000m^3/day**. The number of pump unit determined is (**2**) duty pumps with capacity of the pump is **q_p = 174 LPS**.

- **Diameter of suction pipe**

According to the equation **Eq.2**, the diameter of suction pipe (DIP) is approximately $\Phi_{suction}$ = 400mm with Vs = 1.38m/s.

- **Diameter of discharge pipe**

According to the equation **Eq.3**, the diameter of discharge pipe (DIP) is approximately Φ_d = 350 with velocity V = 1.81m/s.

The main pipe (HDPE) of raw water transmission is supposed Φ_{main}=560mm of diameter with V_d = 1.72m/s.

- **Pumping Head**

The pumping head is the sum of the static head, entrance head, friction loss in the suction pipe, discharge loss, and other band loss as emphasized in equation **Eq.4** to **Eq.12**.

We have:

$H_t = H_{static} + h_{ent} + h_{fs} + h_{fd} + \Sigma h_m$

Where: the static head is h_{static} = 16.68m.

⇨ H_t = 16.68 + 0.005 + 0.067 + 19.615 + 1.323 = 37.69m.

Note: there are more bend along main transmission pipe line, we also used the model to run and the excel sheet found the total dynamic head is approximately 37.54m say 38m.

Therefore, the total dynamic head (H_t) could be derived and resulting H_t = 38m.

- **Determine the type of pump**

We have: H_t = 38m > 15m

Therefore, the type of pump was chosen as the same in 2025 that is the vertical turbine pump type.

- **Summary result of intake pump in BTB**

Description	Year 2025	Year 2040
Total pump capacity	579 LPS	347 LPS
Total pump head	Ht = 35 m	Ht = 38 m
Pump type	Vertical Turbine	Vertical Turbine
Number of pump	3 in duty and 1 stand by, capacity 193LPS/1pump	2 duty and, capacity 174 LPS/1 pump
Total power required	270kw	180kw
Column pipe (DIP)	Φ400 mm	Φ450 mm
Discharge pipe (DIP)	Φ350 mm	Φ350 mm
Main pipe (HDPE)	Φ710 mm, L = 5km	Φ560 mm, L = 5km

According to the results, both pump in 2025 and in 2040 have the same capacity of power consumption. Therefore, the all pumps will be used in the same model and capacity. Furthermore, the standby pump using in 2025 will be remain serving in 2040 as well. However, the flow and head are different, so that it will be introduced to use VFDs to control on such problems (see explanations in Annex below).

B. Booster pump at water treatment plant (WTP) in 2025

The booster pump is the type of pump commonly using for distributing the potable water to the consumers.

i. Capacity of Clearwater pumping station at WTP

The capacity of the pump should consider hourly peak factor, daily peak and 5% loss at WTP. In the project in Battambang focused only in the town, so that the daily peak factor is 1.2; and the hourly peak depends on net flow that in 2025 is 1.61. Therefore, the capacity of the booster pump for 2025 is **889LPS**. The proposed pump are the vertical pump, the number of pump selected is (**3)** pumps and (**1)** stand by with the capacity of each pump being **296LPS**.

ii. Pump head

The total head depends on criteria design such as head loss along pipe (friction loss), different level from WTP to end of pipe network, and pressure requirement. Therefore, the total head requirement for booster pump after modeling is resulted **H = 26.0m**.

C. Booster Pump at WTP in 2040

i. Capacity of Clearwater pumping station at WTP

The capacity of the booster pump in 2040 is **1428 LPS**. The number of pump selected is (**5)** pumps and (**1)** stand by with the capacity of each pump being **287LPS**. The 4 pumps in 2025 will be removed and replaced by the pumps in 2040 because the capacity for 2025 and for 2040 is too different.

ii. Pumping head

The total head depends on criteria design such as head loss along pipe (leaking and friction), different level from WTP to end of pipe network, and pressure requirement. Therefore, the total head for booster pump at the WTP is **H = 51m**.

- **Summary of the booster pump at the WTP in 2025 and 2040**

Description	Year 2025	Year 2040
Total pump capacity	889 LPS	1428 LPS
Total pump head	Ht = 26 m	Ht = 51 m
Pump type	Vertical in-line single stage	Vertical in-line single stage
Number of pump	3 in duty and 1 stand by, capacity 296 LPS/1 pump	5 in duty and 1 stand by, capacity 286 LPS/1 pump
Total power required	330kw	1125kw

D. Sump drainage pump for 2025 & 2040

The capacity of the drainage pump (dewatering pump) for both 2025 and 2040 was determined to be approximately **43.6 LPS**. The number of pump selected is (**1**) pump. The total pump head is about **Ht = 18m**. The diameter of discharge pipe was **Φ = 150mm**. The power required is **11kw**. Sump pump will be utilized only during maintenance/dewatering of the sump at the intake. Thus, the capacity will not change.

Annex:

The vibration frequencies drive (VFD):

A better approach for maintaining the desired water pressure is to regulate the speed of the pump motor with a variable frequency drive (VFD). VFDs control the speed of AC induction motors by controlling the frequency and voltage supplied to the motor. While mechanical devices can be installed at a pump's output to adjust flow, a VFD regulates flow by adjusting the motor/pump speed. This approach is much better for a number of reasons:

- Uses a minimum amount of energy
- Reduces motor starting current
- Provides a degree of motor protection
- Cuts the wear and tear on the motor
- Simplifies the design of the flow control system
- Provides extensive diagnostics
- Reduces the required maintenance

When a VFD is used, the controlled motor's efficiency is optimized and runs at maximum efficiency regardless of the required flow and corresponding motor/pump speed. The VFD input current rises linearly with respect to output power because the VFD can slowly ramp the pump up to speed. As a result, the typical six to seven times motor rated current seen with an across-the-line started motor is nonexistent with a VFD.

As a result, the negative impact of frequent start/stop cycles is greatly reduced because the VFD limits the motor's inrush current, which and prevents the motor's thermal rises that are inherent with across-the-line starting.

Basic VFDs provide phase loss detection and motor thermal overload protection. Advanced pump-specific VFDs offer features such as loss of prime detection, detection of a pump in a no-flow (deadhead) condition, low/high-pressure level detection, broken pipe protection and pump over cycle protection. These factors make VFDs superior to mechanical devices for regulating pump flow, and other features of VFDs allow direct implementation in complex pump control applications.

Annex 3: Pipe calculations

I. Piped Distribution Network

Pipe water supply shall be installed along the road where priority for potential household. We design pipe distribution network to supply in extension area and also to fill up the existing area for the project duration.

It base on:

1. Data of population in each area and population grow.
2. Elevation profile of existing ground.
3. Water demand that include water consumption from domestic, commercial place, institutional place etc.

- Criteria design
  - Type of distribution system by using booster pump and water tower.
  - Water consumption in 2025 is 140 lpcpd and in 2040 is 155 lpcpd
  - The minimum residual pressure at the end of network is around 8 mH2O.
  - The water velocity range from 0.3 to 1.6 m/s to avoid particles sedimentation inside the pipe.
  - Assume Technical Water loss 15% of total demand.
  - Population grow base on each area

    Formula: $\boldsymbol{P_t = P_0 * (1 + e)^t}$

    Where: P_t: Future population for project duration

    P_o: Population in current year/ $starter\ year$

    e : Population grow (%)

    t : Project $duration\ (year)$
  - The roughness calculation is based on Hazen-William formula

    Formula $\boldsymbol{V = kCAR^{0.63}S^{0.54}}$

    Where $V: discharge\ in\ section\ (m/s)$

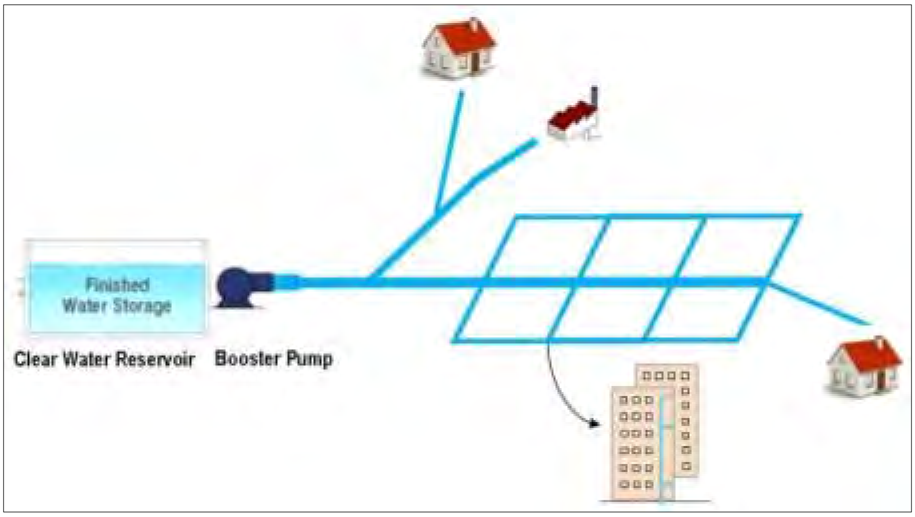
    $C: roughnedd\ coefficient$

    $R: hydaulic\ radius\ (m)$

    $S: friction\ slope\ (m/m)$

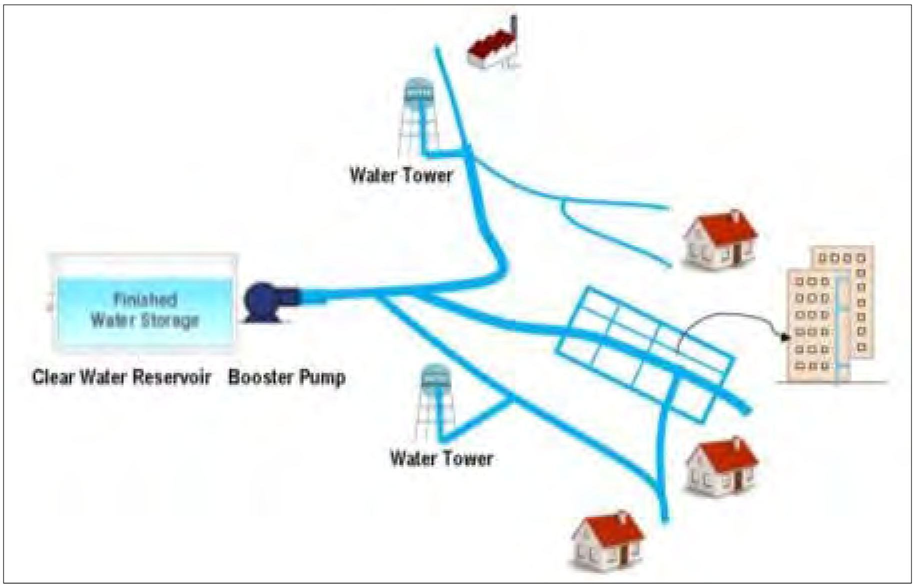
    $k: factor\ depent\ on\ unit$
  - The pipe diameter follow catalogue of HDPE which coefficient roughness C=150.
  - Water demand that estimate normally in average water consumption. In practice it will be increase whenever they use at the same time. Peak hour in year 2025 is 1.61 and in 2040 peak hour is 1.58.
  - Air valve and drained valve will be provided for the higher and lower of existing ground level.
  - EPANET software for design pipe network.
  - There have two options for pipe network design:
    - Option 1: We use booster from water treatment plant to distribute directly to consumption place (household, institutional building, commercial, industrial place…)
    - Option 2: We use booster from the Water treatment plant to distribute along the pipeline and install 2 water towers at the system.

Figure 1: Option 1_Schema of distribution pipe network



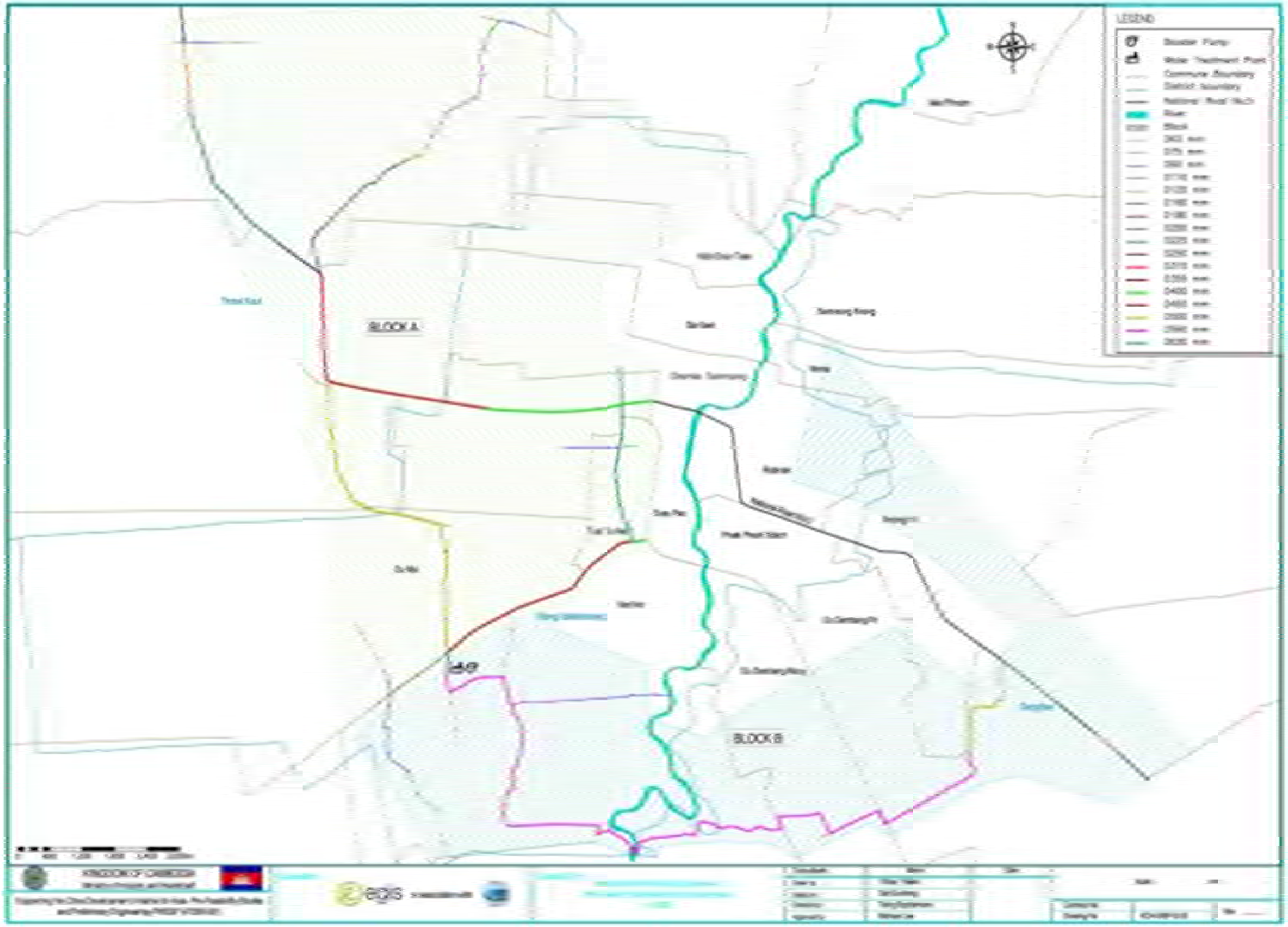
Option 1: we use booster pump to distribute the water for 24 hours and the total head will vary depend on the consumption of water's user.

Figure 2: Option 2_Schema of distribution pipe network



Option 2: we use the booster pump and two water towers for distribution 24 hours. In this option, we install the water tower to stock the water from the pipeline whenever the consumptions less than normal users or the residual pressure can allow the water into the tanks. And it will continue to distribute the water in the downstream area.

Figure 3: Block for pipe network in Battambang water supply



II. Data input

Table 1: Estimation of Water Demand in 2025

No.	Commune	Village	Pop_Total	Coverage (%)		Est_pop	Pop_growt	Pop_design in	Unit Water cons.	Water deamand (L/d)		Water loss	Total_water deamand		
				Dosmectic	Big Cons.		(%)	2025	lpcpd	Dosmectic	Big Cons.	(%)	(L/d)	(m3/d)	(m3/hour)
1	Tuol Ta Aek	Dang Kou Teab	2,033	100%	19.0%	2,033	1.91	2,456	140	343,900.99	65,341.19	15%	470,629	470.63	19.61
2		Ou Tar Kam 1	4,547	100%	5.0%	4,547	1.91	5,494	140	769,168	38,458	15%	928,770	928.77	38.70
3		Ou Tar Kam 2	5,417	24%	3.0%	1,300	1.91	1,571	140	219,921	6,597.6	15%	260,496	260.50	10.85
4	Chamkar samraong	Chamkar Samraong 2	4,613	95%	10.0%	4,382	2.33	5,517	140	772,440	77,244	15%	977,137	977.14	40.71
5	Ou Mal	Andoung Pring	1,531	90%	15.0%	1,378	1.34	1,574	140	220,371	33,055.7	15%	291,441	291.44	12.14
6		Beong Reang	1,638	90%	21.0%	1,474	1.34	1,684	140	235,773	49,512	15%	328,078	328.08	13.67
7		Dak Sasar	1,136	90%	9.0%	1,022	1.34	1,168	140	163,515	14,716.4	15%	204,966	204.97	8.54
8		Koun Sek	650	90%	18.0%	585	1.34	668	140	93,561	16,841	15%	126,962	126.96	5.29
9		Ou Mal	854	90%	7.0%	769	1.34	878	140	122,924	8,604.7	15%	151,258	151.26	6.30
10		Prey Dach	977	90%	-	879	1.34	1,004	140	140,628.75	-	15%	161,723	161.72	6.74
11		Prey Roka	895	90%	-	806	1.34	920	140	128,826	-	15%	148,150	148.15	6.17
12		Sala Balat	1,218	90%	9.0%	1,096	1.34	1,252	140	175,318	15,779	15%	219,761	219.76	9.16
13		Vaot Roka	1,460	90%	-	1,314	1.34	1,501	140	210,151	-	15%	241,674	241.67	10.07
14	Voat Kor	Ballang	2,346	90%	5.0%	2,111	1.18	2,374	140	332,388	16,619	15%	401,358	401.36	16.72
15		Chrab Krasang	3,762	100%	12.2%	3,762	1.18	4,230	140	592,234	71,956.5	15%	763,819	763.82	31.83
16		Damnak Luong	2,659	90%	9.0%	2,393	1.18	2,691	140	376,734.71	33,906	15%	472,237	472.24	19.68
17		Kampong Seima	2,064	90%	-	1,858	1.18	2,089	140	292,433	-	15%	336,298	336.30	14.01
18	Ou Char	Anchanh	2,785	95%	20.3%	2,646	0.85	2,879	140	403,121	81,632	15%	557,467	557.47	23.23
19		Andoung Chenh	2,260	100%	24.0%	2,260	0.85	2,460	140	344,346	82,643.1	15%	491,038	491.04	20.46
20		Ang	2,996	85%	24.3%	2,547	0.85	2,772	140	388,014	94,093	15%	554,424	554.42	23.10
21		Kab Kou Thmei	4,423	95%	16.0%	4,202	0.85	4,573	140	640,218	102,434.8	15%	854,050	854.05	35.59
22		Ou Char	4,209	95%	12.0%	3,999	0.85	4,352	140	609,242	72,804	15%	784,353	784.35	32.68
23		Prey Koun Sek	1,245	90%	-	1,121	0.85	1,219	140	170,726	-	15%	196,335	196.33	8.18
24	Anlong Vil	Puk Chhma	894	90%	5.0%	805	1.02	891	140	124,676	6,234	15%	150,546	150.55	6.27
25		Svay Kang	2,873	90%	5.0%	2,586	1.02	2,862	140	400,664	20,033.2	15%	483,801	483.80	20.16
26	Ou Dambang 1	Ou Khcheay	2,026	90%	9.0%	1,823	1.6	2,137	140	299,190	26,927	15%	375,035	375.03	15.63
27	Ou Dambang 2	Dambouk Khpos	2,380	90%	18.0%	2,142	1.72	2,540	140	355,640	64,015.3	15%	482,604	482.60	20.11
28		Svay Thum	2,129	90%	11.0%	1,916	1.72	2,272	140	318,134	34,995	15%	406,098	406.10	16.92
29	Voat Ta Muem	Ou Khcheay	2,219	95%	-	2,108	2.16	2,610	140	365,441	-	15%	420,258	420.26	17.51
30	Chrey	Chrey Thmei	1,220	90%	5.0%	1,098	2.08	1,349	140	188,859	9,443	15%	228,047	228.05	9.50
31		Hai San	2,757	100%	10.0%	2,757	2.08	3,387	140	474,211	47,421.1	15%	599,877	599.88	24.99
32		Ka Kou	1,901	90%	8.0%	1,711	2.08	2,102	140	294,279	23,542	15%	365,495	365.49	15.23
33		Kbal Khmaoch	1,818	90%	5.0%	1,636	2.08	2,010	140	281,430	14,071.5	15%	339,827	339.83	14.16
34		Popeal Khae	2,397	100%	12.0%	2,397	2.08	2,945	140	412,290	49,475	15%	531,029	531.03	22.13
35		Prey Totueng	1,895	90%	-	1,706	2.08	2,095	140	293,350	-	15%	337,353	337.35	14.06
36		Svay Chrum	1,548	90%	5.0%	1,393	2.08	1,712	140	239,634	11,982	15%	289,358	289.36	12.06
37	Ou Ta Ki	Kakaoh	1,313	90%	6.0%	1,182	1.94	1,432	140	200,485	12,029.1	15%	244,391	244.39	10.18
38		Ou Ta Ki	3,806	96%	3.0%	3,654	1.94	4,428	140	619,890	18,597	15%	734,259	734.26	30.59
39		Popeal Khae	2,874	85%	6.0%	2,443	1.94	2,960	140	414,458	24,867.5	15%	505,224	505.22	21.05
40		Prey Dach	1,531	90%	-	1,378	1.94	1,670	140	233,772	-	15%	268,838	268.84	11.20
41		Prey Totueng	1,596	90%	-	1,436	1.94	1,741	140	243,697	-	15%	280,251	280.25	11.68
42		Trang	1,950	90%	-	1,755	1.94	2,127	140	297,750	-	15%	342,412	342.41	14.27
43		Tras	1,837	100%	-	1,837	1.94	2,226	140	311,662	-	15%	358,411	358.41	14.93
Total			**96,682**			**86,245**		**100,825**	**6,020**	**14,115,464**	**1,245,871**		**17,665,536**	**17,666**	**736.06**

Table 2: Estimation of Water Demand in 2040

No.	Commune	Village	Pop_Total	Coverage (%)		Est_pop	Pop_growt	Pop_design in	Unit Water cons.	Water deamand (Ltr/d)		Water loss	Total_water deamand		
				Dosmectic	Big Cons.		(%)	2040	(Ltr/Pop/d)	Dosmectic	Big Cons.	(%)	(Ltr/d)	(m3/d)	(m3/hour)
1	Tuol Ta Aek	Dang Kou Teab	2,033	100%	19.0%	2,033	1.91	3,263	155	505,695.53	96,082.15	15%	692,044	692.04	28.84
2		Ou Tar Kam 1	4,547	100%	5.0%	4,547	1.91	7,297	155	1,131,037	56,552	15%	1,365,727	1,365.73	56.91
3		Ou Tar Kam 2	5,417	24%	3.0%	1,300	1.91	2,086	155	323,386	9,701.6	15%	383,051	383.05	15.96
4	Chamkar samraong	Chamkar Samraong 2	4,613	95%	10.0%	4,382	2.33	7,794	155	1,208,128	120,813	15%	1,528,282	1,528.28	63.68
5	Ou Mal	Andoung Pring	1,531	90%	15.0%	1,378	1.34	1,922	155	297,901	44,685.1	15%	393,974	393.97	16.42
6		Beong Reang	1,638	90%	21.0%	1,474	1.34	2,056	155	318,721	66,931	15%	443,500	443.50	18.48
7		Dak Sasar	1,136	90%	9.0%	1,022	1.34	1,426	155	221,042	19,893.8	15%	277,076	277.08	11.54
8		Koun Sek	650	90%	18.0%	585	1.34	816	155	126,477	22,766	15%	171,629	171.63	7.15
9		Ou Mal	854	90%	7.0%	769	1.34	1,072	155	166,171	11,632.0	15%	204,473	204.47	8.52
10		Prey Dach	977	90%	-	879	1.34	1,226	155	190,104.01	-	15%	218,620	218.62	9.11
11		Prey Roka	895	90%	-	806	1.34	1,124	155	174,149	-	15%	200,271	200.27	8.34
12		Sala Balat	1,218	90%	9.0%	1,096	1.34	1,529	155	236,998	21,330	15%	297,077	297.08	12.38
13		Vaot Roka	1,460	90%	-	1,314	1.34	1,833	155	284,086	-	15%	326,699	326.70	13.61
14	Voat Kor	Ballang	2,346	90%	5.0%	2,111	1.18	2,831	155	438,802	21,940	15%	529,854	529.85	22.08
15		Chrab Krasang	3,762	100%	12.2%	3,762	1.18	5,044	155	781,839	94,993.4	15%	1,008,357	1,008.36	42.01
16		Damnak Luong	2,659	90%	9.0%	2,393	1.18	3,209	155	497,346.91	44,761	15%	623,424	623.42	25.98
17		Kampong Seima	2,064	90%	-	1,858	1.18	2,491	155	386,056	-	15%	443,965	443.96	18.50
18	Ou Char	Anchanh	2,785	95%	20.3%	2,646	0.85	3,269	155	506,732	102,613	15%	700,747	700.75	29.20
19		Andoung Chenh	2,260	100%	24.0%	2,260	0.85	2,793	155	432,850	103,884.1	15%	617,244	617.24	25.72
20		Ang	2,996	85%	24.3%	2,547	0.85	3,147	155	487,742	118,277	15%	696,922	696.92	29.04
21		Kab Kou Thmei	4,423	95%	16.0%	4,202	0.85	5,192	155	804,766	128,762.6	15%	1,073,558	1,073.56	44.73
22		Ou Char	4,209	95%	12.0%	3,999	0.85	4,941	155	765,829	91,517	15%	985,947	985.95	41.08
23		Prey Koun Sek	1,245	90%	-	1,121	0.85	1,385	155	214,606	-	15%	246,796	246.80	10.28
24	Anlong Vil	Puk Chhma	894	90%	5.0%	805	1.02	1,037	155	160,730	8,036	15%	194,081	194.08	8.09
25		Svay Kang	2,873	90%	5.0%	2,586	1.02	3,332	155	516,528	25,826.4	15%	623,708	623.71	25.99
26	Ou Dambang 1	Ou Khcheay	2,026	90%	9.0%	1,823	1.6	2,712	155	420,297	37,827	15%	526,842	526.84	21.95
27	Ou Dambang 2	Dambouk Khpos	2,380	90%	18.0%	2,142	1.72	3,281	155	508,522	91,534.0	15%	690,065	690.06	28.75
28		Svay Thum	2,129	90%	11.0%	1,916	1.72	2,935	155	454,892	50,038	15%	580,670	580.67	24.19
29	Voat Ta Muem	Ou Khcheay	2,219	95%	-	2,108	2.16	3,597	155	557,487	-	15%	641,110	641.11	26.71
30	Chrey	Chrey Thmei	1,220	90%	5.0%	1,098	2.08	1,837	155	284,741	14,237	15%	343,825	343.83	14.33
31		Hai San	2,757	100%	10.0%	2,757	2.08	4,613	155	714,965	71,496.5	15%	904,431	904.43	37.68
32		Ka Kou	1,901	90%	8.0%	1,711	2.08	2,862	155	443,683	35,495	15%	551,054	551.05	22.96
33		Kbal Khmaoch	1,818	90%	5.0%	1,636	2.08	2,737	155	424,311	21,215.6	15%	512,356	512.36	21.35
34		Popeal Khae	2,397	100%	12.0%	2,397	2.08	4,010	155	621,608	74,593	15%	800,630	800.63	33.36
35		Prey Totueng	1,895	90%	-	1,706	2.08	2,853	155	442,283	-	15%	508,625	508.63	21.19
36		Svay Chrum	1,548	90%	5.0%	1,393	2.08	2,331	155	361,295	18,065	15%	436,263	436.26	18.18
37	Ou Ta Ki	Kakaoh	1,313	90%	6.0%	1,182	1.94	1,910	155	296,111	17,766.7	15%	360,959	360.96	15.04
38		Ou Ta Ki	3,806	96%	3.0%	3,654	1.94	5,907	155	915,561	27,467	15%	1,084,482	1,084.48	45.19
39		Popeal Khae	2,874	85%	6.0%	2,443	1.94	3,949	155	612,143	36,728.6	15%	746,203	746.20	31.09
40		Prey Dach	1,531	90%	-	1,378	1.94	2,228	155	345,275	-	15%	397,066	397.07	16.54
41		Prey Totueng	1,596	90%	-	1,436	1.94	2,322	155	359,934	-	15%	413,924	413.92	17.25
42		Trang	1,950	90%	-	1,755	1.94	2,837	155	439,769	-	15%	505,734	505.73	21.07
43		Tras	1,837	100%	-	1,837	1.94	2,970	155	460,317	-	15%	529,364	529.36	22.06
Total			**96,682**			**86,245**		**128,006**	**6,665**	**19,840,917**	**1,707,460**		**24,780,633**	**24,781**	**1032.53**

III. Result

We use Epanet software to design diameter of pipeline which follow to design criteria.

III.1. Result of option 1 (Use Only Booster Pump)

Figure 4: Layout Plant-Piped Network of Battambang In Extension Area in 2025 (Option 1)

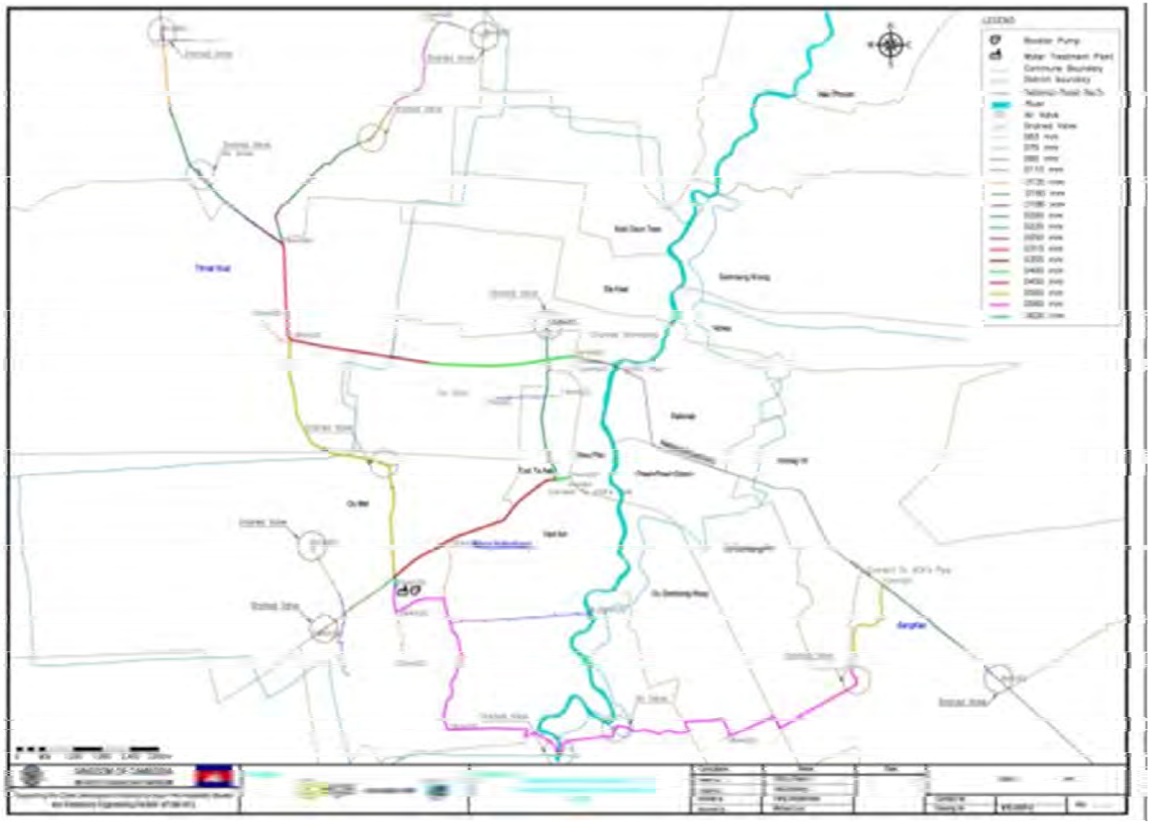


Figure 5: Layout plant- Piped Network of Battambang In Extension Area In 2040 (Option 1)

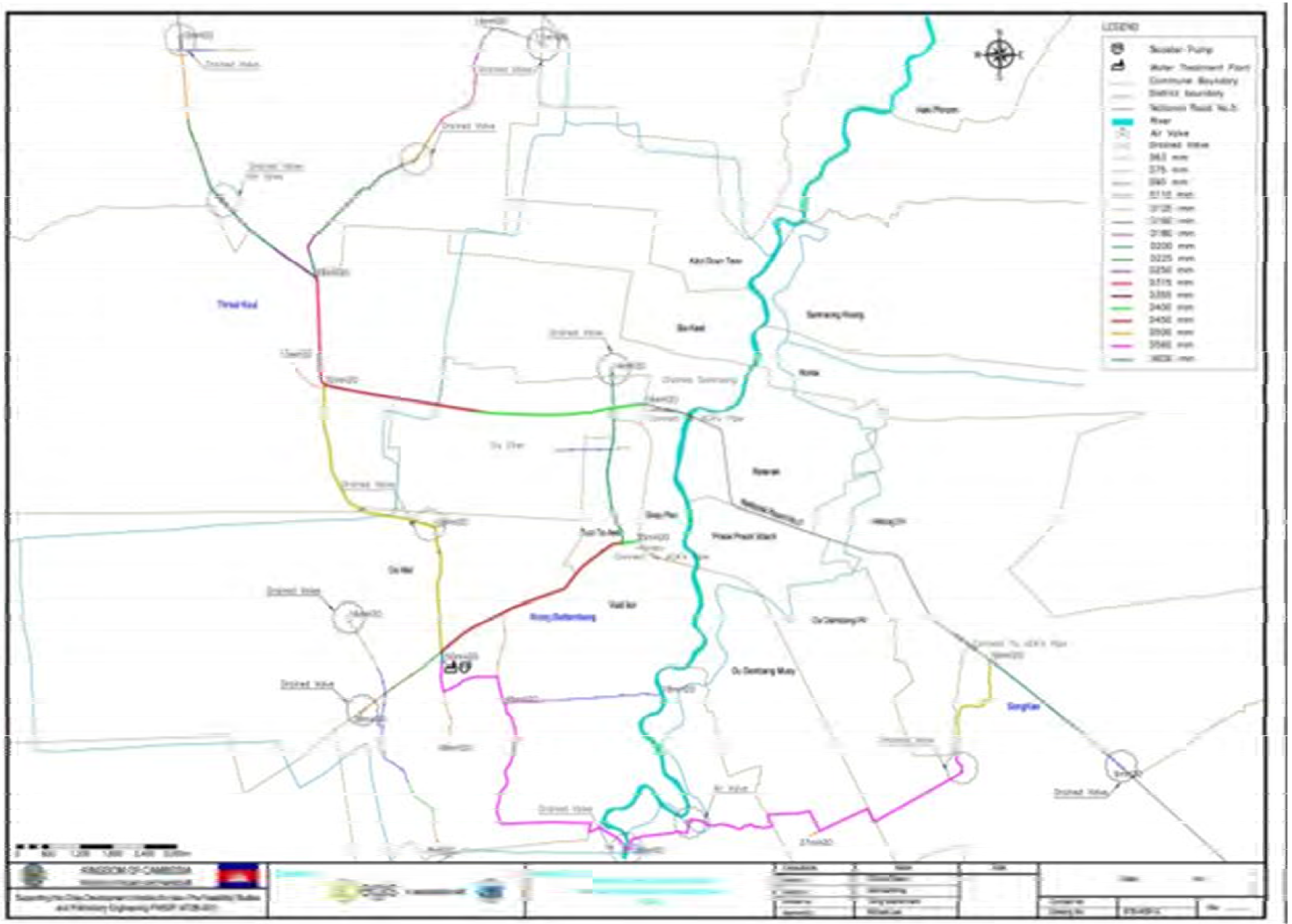


Table 3: Summaries data for facility of Pipe Network in Block A

Summaries for Battambang pipe water supply in Block A			
No.	**I. Pipe Diameter (mm) (*1)**	**Pressure Normal (PN)**	**Pipe Length (m)**
1	63	PE100, PN 10	6,618
2	75	PE100, PN 10	1,534
3	90	PE100, PN 10	3,057
4	110	PE100, PN 10	3,024
5	125	PE100, PN 8	4,668
6	160	PE100, PN 8	8,377
7	180	PE100, PN 8	889
8	200	PE100, PN 8	8,906
9	225	PE100, PN 8	2,017
10	250	PE100, PN 8	2,020
11	315	PE100, PN 8	4,061
12	400	PE100, PN 8	6,671
13	450	PE100, PN 8	14,102
14	500	PE100, PN 8	12,798
15	560	PE100, PN 8	898
16	630	PE100, PN 8	442
Total pipe length (m)			**80,082**

Note: (*1) Pipe diameter here follow pipe catalogue

Table 4: Summaries data for facility of Pipe Network In Block B

Summaries for Battambang pipe water supply in Block B			
No.	**I. Pipe Diameter (mm) (*1)**	**Pressure Normal (PN)**	**Pipe Length (m)**
1	63	PE100, PN 10	2,479
2	75	PE100, PN 10	1,000
3	90	PE100, PN 10	4,488
4	110	PE100, PN 10	1,002
5	125	PE100, PN 8	499
6	160	PE100, PN 8	483
7	200	PE100, PN 8	1,756
8	500	PE100, PN 8	2,187
9	560	PE100, PN 8	15,005
Total pipe length (m)			**28,899**

Note: (*1) Pipe diameter here follow pipe catalogue

Table 5: Summaries data for facility of Pipe Network Extension and Fill up Area

No.	**Summaries for Battambang pipe water supply**		
I. Pipe Diameter (mm) (*1)		**Pressure Normal (PN)**	**Pipe Length (m)**
1	63	PE100, PN 10	9,097
2	75	PE100, PN 10	2,534
3	90	PE100, PN 10	7,545
4	110	PE100, PN 10	4,026
5	125	PE100, PN 8	5,167
6	160	PE100, PN 8	8,860
7	180	PE100, PN 8	889
8	200	PE100, PN 8	10,662
9	225	PE100, PN 8	2,017
10	250	PE100, PN 8	2,020
11	315	PE100, PN 8	4,061
12	400	PE100, PN 8	6,671

13	450	PE100, PN 8	14,102
14	500	PE100, PN 8	14,985
15	560	PE100, PN 8	15,903
16	630	PE100, PN 8	442
Total pipe length (m)			**108,981**
II. Total Pump Capacity in 2025		**Flow (m3/h)**	**Head (m)**
17	At Water Treatment Plant	2,667	25
III. Total Pump Capacity in 2040		**Flow (m3/h)**	**Head (m)**
18	At Water Treatment Plant	4,331	50

Note: (*1) Pipe diameter here follow pipe catalogue

According to Table result above:

- Diameter of pipeline was design for demand in 2040.
- Pipe diameters design to supply for new extension area and also to fill up the demand in the existing area. It needs to connect with JICA's pipe in order to respond the pressure at the end of pipe network enough.
- The capacity of booster pump need to be design in peak demand for whole scheme (include block A and B) which device in 2 phases:
  - **Phase 1**: From 2015 to 2025, the total capacity of booster pump at water treatment plant is 2,667 m^3/h; total head 25 mH2O in peak hour.
  - **Phase 2**: After 2025 to 2040, the total capacity of booster pump at water treatment plant is 4,331 m^3/h; Total head 50 mH2O.

III.2. Result of option 2 (Use Booster Pump with Water Tower)

Figure 6: Layout Plant-Piped Network of Battambang In Extension Area in 2025 (Option 2)

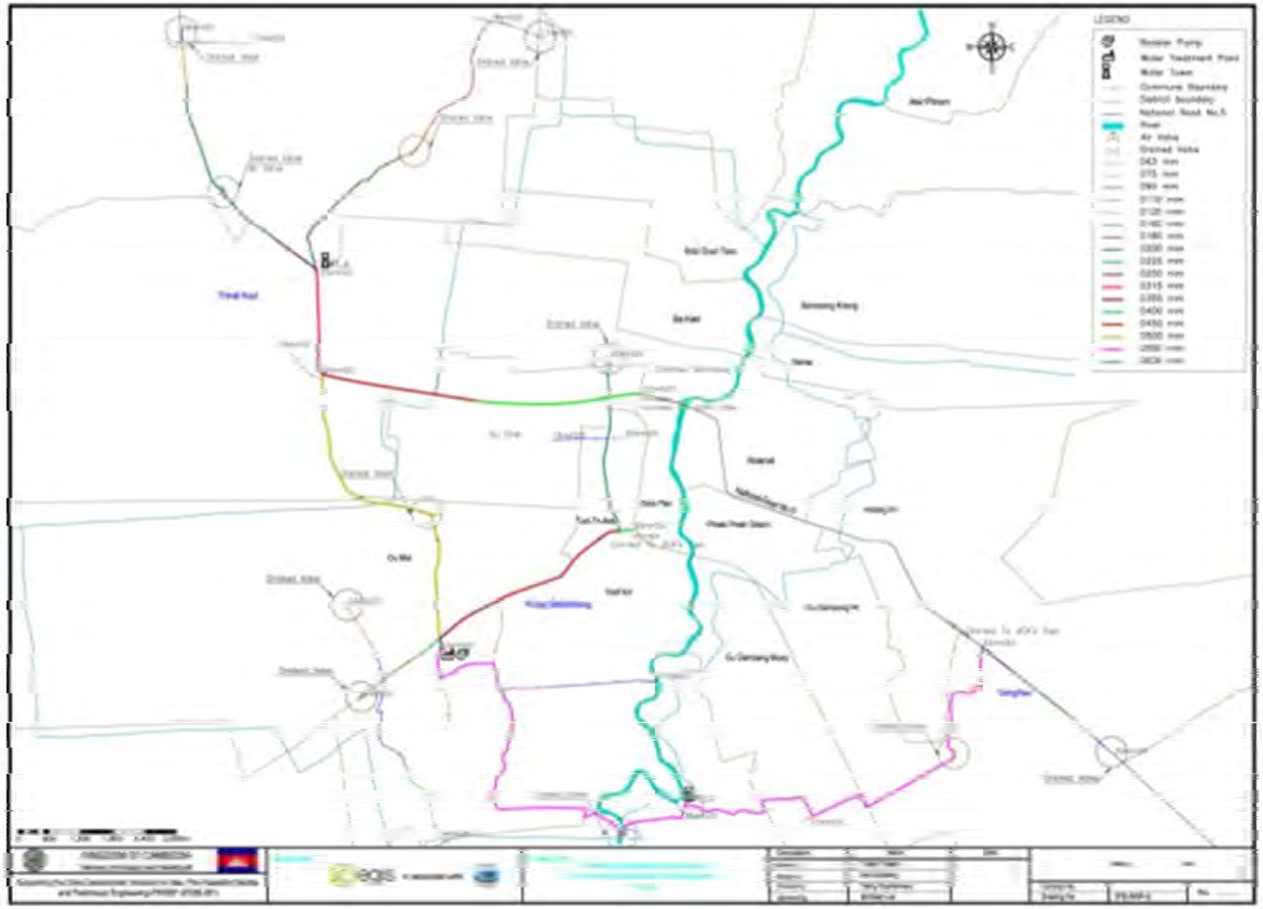


Figure 7: Layout Plant-Piped Network of Battambang In Extension Area in 2040 (Option 2)

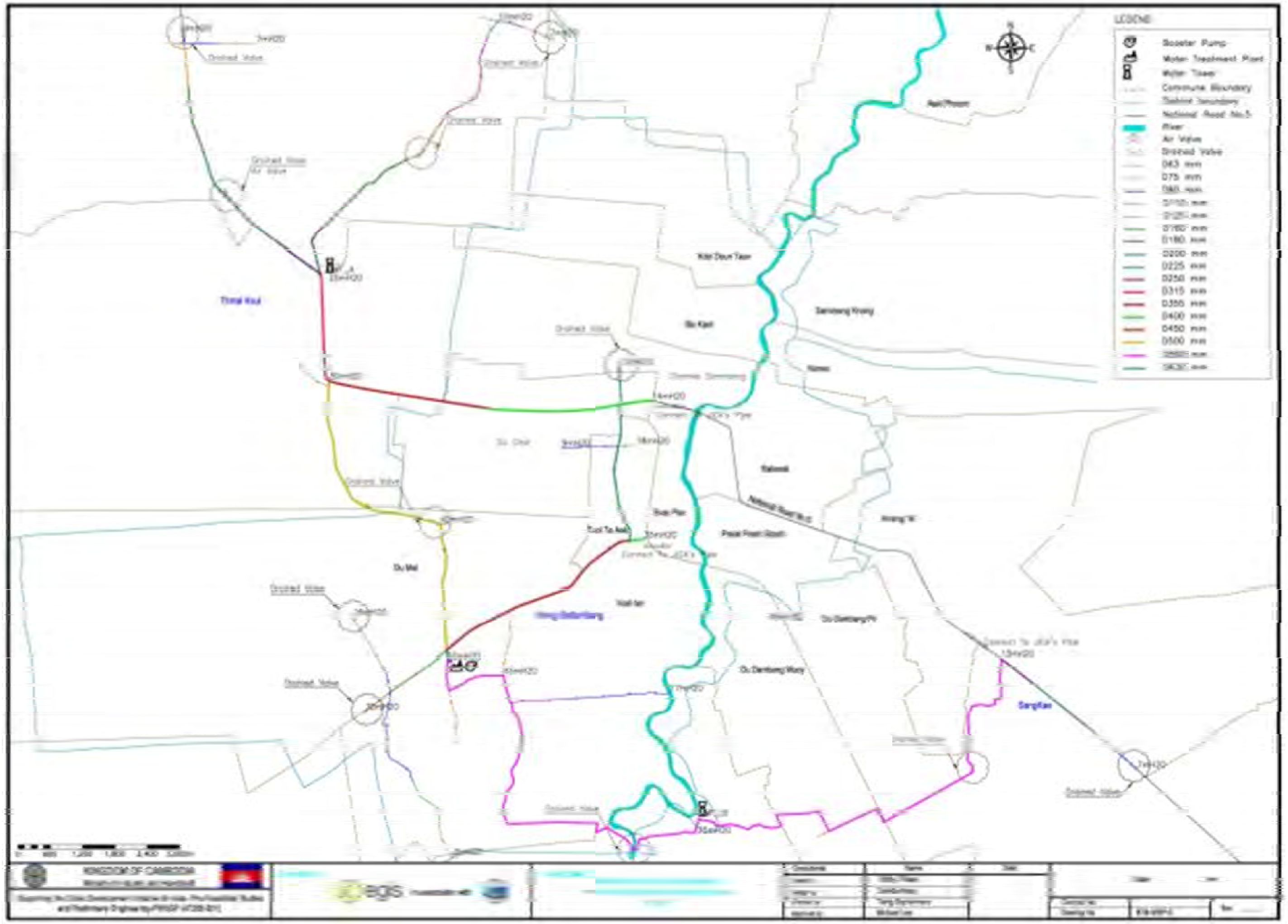


Table 6: Summaries data for facility of Pipe Network in Block A

Summaries for Battambang pipe water supply			
No.	**I. Pipe Diameter (mm) (*1)**	**Pressure Normal (PN)**	**Pipe Length (m)**
1	63	PE100, PN 10	6,618
2	75	PE100, PN 10	1,534
3	90	PE100, PN 10	3,057
4	110	PE100, PN 10	3,024
5	125	PE100, PN 8	3,671
6	160	PE100, PN 8	9,374
7	180	PE100, PN 8	889
8	200	PE100, PN 8	8,906
9	225	PE100, PN 8	2,017
10	250	PE100, PN 8	2,020
11	315	PE100, PN 8	4,093
12	400	PE100, PN 8	6,671
13	450	PE100, PN 8	14,102
14	500	PE100, PN 10	12,798
15	560	PE100, PN 11	912
16	630	PE100, PN 11	442
Total pipe length (m)			**80,128**
II. Total Pump Capacity in 2025		**Flow (m3/h)**	**Head (m)**
18	At Water Treatment Plant	2,667	25
III. Total Pump Capacity in 2040		**Flow (m3/h)**	**Head (m)**
19	At Water Treatment Plant	4,331	50
III. Water Tower		**Volume (m3)**	**Height bottom of the Tank (m)**
20	Water Tower in Block A (Supply for 1 hour with hourly peak) (*2)	300	25

Note: (*1) Pipe diameter here follow pipe catalogue
(*2) The location of water tower in block A will be built in end of Ka Kou Village. Where the nature ground is 13.3m (m.s.l)

Table 7: Summaries data for facility of Pipe Network In Block B

Summaries for Battambang pipe water supply			
No.	**I. Pipe Diameter (mm) (*1)**	**Pressure Normal (PN)**	**Pipe Length (m)**
1	63	PE100, PN 10	2,479
2	75	PE100, PN 10	1,000
3	90	PE100, PN 10	4,488
4	110	PE100, PN 10	1,002
5	125	PE100, PN 8	499
6	160	PE100, PN 8	483
7	200	PE100, PN 8	982
8	250	PE100, PN 8	774
9	560	PE100, PN 8	17,192
Total pipe length (m)			**28,899**
II. Water Tower		**Volume (m3)**	**Height bottom of the Tank (m)**
21	water Tower In Block B (Supply for 1 hour with hourly peak) (*2)	910	25

Note: (*1) Pipe diameter here follow pipe catalogue
(*2) The location of water tower in block B will be built in Ou Khcheay village. Where the nature ground is 15.53m (m.s.l)

Table 8: Summaries data for facility of Pipe Network Extension and Fill up Area

Summaries for Battambang pipe water supply			
No.	**I. Pipe Diameter (mm) (*1)**	**Pressure Normal (PN)**	**Pipe Length (m)**
1	63	PE100, PN 10	9,097
2	75	PE100, PN 10	2,534
3	90	PE100, PN 10	7,545
4	110	PE100, PN 10	4,026
5	125	PE100, PN 8	4,170
6	160	PE100, PN 8	9,857
7	180	PE100, PN 8	889
8	200	PE100, PN 8	9,888
9	225	PE100, PN 8	2,017
10	250	PE100, PN 8	2,794
11	315	PE100, PN 8	4,093
13	400	PE100, PN 8	6,671
14	450	PE100, PN 8	14,102
15	500	PE100, PN 8	12,798
16	560	PE100, PN 8	18,104
17	630	PE100, PN 8	442
Total pipe length (m)			**109,027**
II. Total Pump Capacity in 2025		**Flow (m3/h)**	**Head (m)**
18	At Water Treatment Plant	2,667	33
III. Total Pump Capacity in 2040		**Flow (m3/h)**	**Head (m)**
19	At Water Treatment Plant	4,331	50
III. Water Tower		**Volume (m3)**	**Height bottom of the Tank (m)**
20	Water Tower in Block A (Supply for 1 hour with hourly peak) (*2)	300	25
21	water Tower In Block B (Supply for 1 hour with hourly peak) (*3)	910	25

Note: (*1) Pipe diameter here follow pipe catalogue
(*2) The location of water tower in block A will be built in end of Ka Kou Village.
Where the nature ground is 13.3m (m.s.l)
(*3) The location of water tower in block B will be built in Ou Khcheay village.
Where the nature ground is 15.53m (m.s.l)

According to Table result above:

- Diameter of pipeline was design for demand in 2040.
- Pipe diameters design to supply for new extension area and also to fill up the demand in the existing area. It needs to connect with JICA's pipe in order to respond the pressure at the end of pipe network enough.
- The capacity of booster pump need to be design in peak demand for whole scheme (include block A and B) which device in 2 phases:
  - **Phase 1**: From 2015 to 2025, the total capacity of booster pump at water treatment plant is 2,667 m^3/h; total head 33 mH2O in peak hour in order to allow convey the water into water tower.
  - **Phase 2**: After 2025 to 2040, the total capacity of booster pump at water treatment plant is 4,331 m^3/h; Total head 50 mH2O.
- The volume of water tower in block A is 300 m^3 with the height bottom of the tanks is 25 m. It is supply for 24 hours.
- The volume of water tower in block B is 910 m^3 with the height bottom of the tanks is 25 m, and also supply for 24 hours.

IV. Data from EPANET

IV.1. Option 1: Use Only Booster Pump

Figure 8: Network map

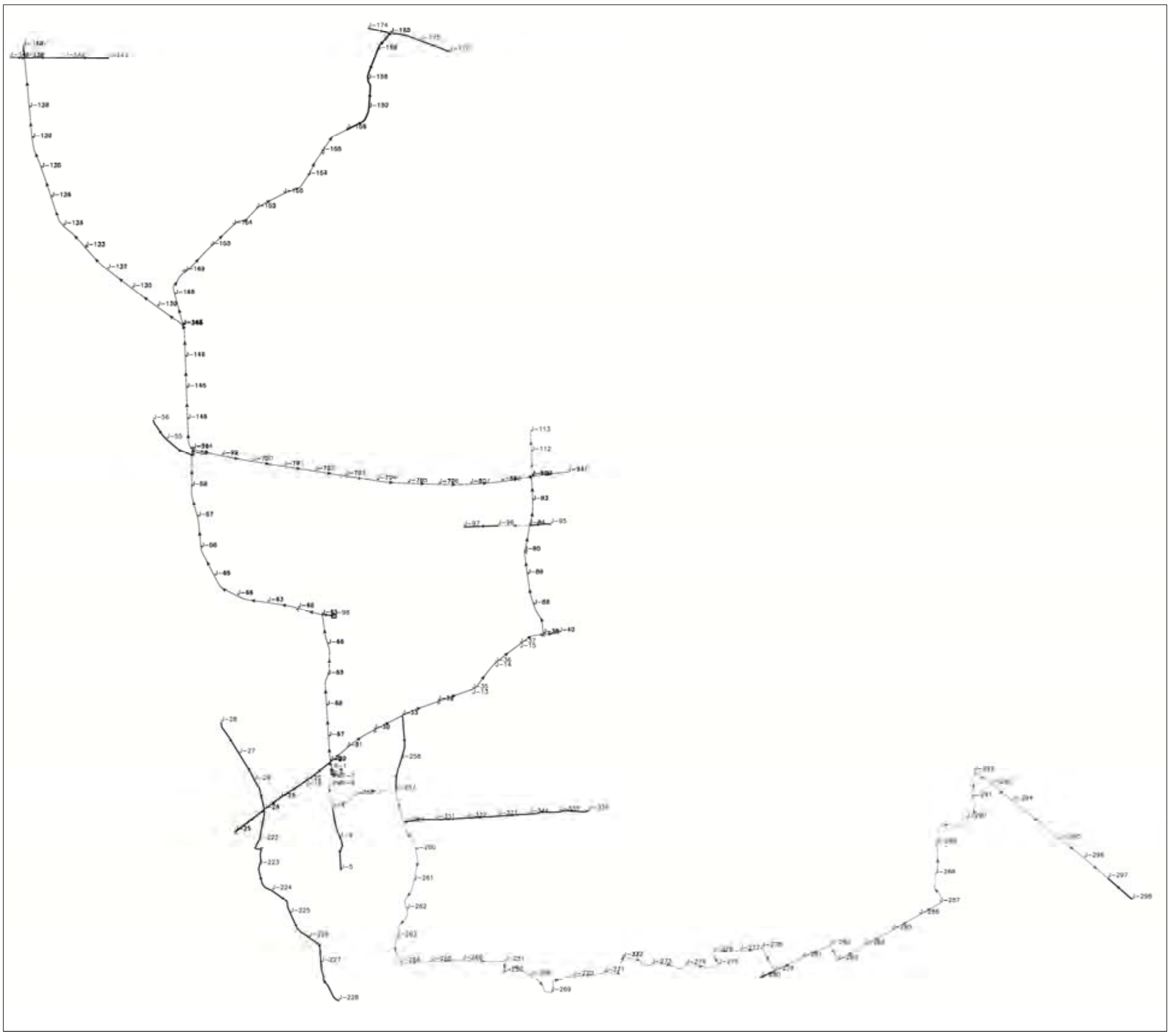


Table 9: Node data in 2025

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-1	13.99	6.81	25.00	18.00
J-2	14.01	20.43	24.99	17.95
J-3	14.84	34.05	23.64	16.91
J-4	14.71	13.62	23.49	16.92
J-5	14.69	6.81	23.48	16.93
J-6	15.01	52.78	23.73	16.85
J-7	15.01	27.24	23.74	16.85
J-8	15.11	87.96	23.63	16.74
J-9	14.70	73.21	23.54	16.95
J-10	14.80	76.28	22.71	16.54
J-11	14.89	84.73	21.93	16.17
J-12	14.65	36.10	21.35	16.07
J-13	15.36	48.14	19.89	15.05
J-14	12.63	71.00	21.88	17.47
J-15	12.45	83.04	21.43	17.39
J-16	13.75	83.04	19.61	15.88
J-17	12.47	4031.81	20.63	17.05
J-18	14.89	27.24	21.47	15.98
J-19	15.29	33.20	18.80	14.64
J-20	15.14	56.84	17.60	14.23
J-21	14.16	10.22	17.68	14.84
J-22	14.89	21.34	21.75	16.10
J-23	15.29	31.70	18.20	14.39
J-24	15.05	167.05	17.68	14.32
J-25	14.16	63.27	17.67	14.83
J-26	13.75	135.31	13.60	13.39
J-27	12.84	96.88	6.18	10.85
J-28	12.82	34.47	5.37	10.53
J-29	15.15	99.49	23.58	16.70
J-30	15.15	28.16	23.57	16.69
J-31	14.70	61.95	23.54	16.95
J-32	14.80	65.47	22.71	16.54
J-33	15.14	44.16	21.68	15.92
J-34	14.65	77.29	21.33	16.06
J-35	15.36	99.04	19.86	15.04
J-36	12.63	158.18	21.86	17.46
J-37	12.45	193.79	21.43	17.39
J-38	12.60	110.74	20.82	17.05
J-39	12.46	66.65	20.88	17.16
J-40	12.47	3990.94	20.61	17.04
J-41	14.39	69.15	23.83	17.25
J-42	14.06	54.32	23.51	17.31
J-43	14.10	44.22	22.83	17.01
J-44	14.01	35.22	22.29	16.83
J-45	13.25	44.77	22.44	17.34
J-46	13.49	56.03	21.72	16.90
J-47	13.26	53.91	21.35	16.88
J-48	13.22	64.05	20.79	16.67
J-49	13.54	65.60	19.86	16.10
J-50	13.58	74.97	19.23	15.82
J-51	13.25	65.21	18.98	15.91
J-52	13.25	118.34	18.41	15.67
J-53	12.88	161.07	18.22	15.81

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m^3/day)	Peak Demand	Average Demand
J-54	12.04	44.95	18.95	16.60
J-55	11.64	104.62	10.09	13.17
J-56	11.30	35.96	9.85	13.27
J-57	14.39	62.81	23.82	17.25
J-58	14.06	53.48	23.51	17.31
J-59	14.10	36.56	22.84	17.01
J-60	14.01	65.06	22.29	16.83
J-61	13.26	82.83	22.44	17.33
J-62	13.49	40.33	21.72	16.90
J-63	13.26	4.49	21.35	16.88
J-64	13.22	1.87	20.79	16.67
J-65	13.54	20.62	19.87	16.10
J-66	13.58	44.98	19.24	15.82
J-67	13.25	56.23	18.99	15.91
J-68	13.25	101.57	18.41	15.67
J-69	12.88	82.48	18.22	15.81
J-70	13.06	76.35	17.93	15.58
J-71	12.85	58.93	18.14	15.79
J-72	12.78	128.16	17.73	15.67
J-73	12.28	89.98	17.76	15.97
J-74	12.46	27.27	17.12	15.60
J-75	12.45	16.36	16.67	15.42
J-76	11.26	38.18	17.41	16.42
J-77	12.20	65.44	16.02	15.30
J-78	11.28	125.84	16.16	15.89
J-79	11.70	209.92	15.00	15.17
J-80	11.24	150.76	14.78	15.34
J-81	11.33	155.80	14.06	14.99
J-82	11.58	169.63	13.21	14.50
J-83	11.19	66.36	14.45	15.24
J-84	11.62	90.08	14.97	15.20
J-85	11.44	95.09	16.97	16.13
J-86	11.61	141.14	18.53	16.68
J-87	11.69	116.69	19.98	17.23
J-88	11.69	206.53	19.78	17.15
J-89	11.61	168.05	18.39	16.62
J-90	11.44	51.97	16.98	16.14
J-91	11.62	131.58	15.20	15.30
J-92	11.19	55.66	14.57	15.29
J-93	11.48	203.91	13.31	14.60
J-94	10.49	4155.24	13.66	15.32
J-95	11.70	79.61	14.16	14.82
J-96	11.00	146.22	9.54	13.31
J-97	11.38	136.49	7.44	12.22
J-98	12.74	6.71	22.94	17.84
J-99	12.78	120.57	17.69	15.65
J-100	12.28	109.05	17.68	15.94
J-101	12.46	106.68	17.01	15.56
J-102	12.45	161.00	16.53	15.36
J-103	11.26	126.48	17.28	16.37
J-104	12.20	118.30	15.91	15.25
J-105	11.28	141.07	16.10	15.87
J-106	11.70	93.27	14.99	15.16
J-107	11.24	104.31	14.78	15.34
J-108	11.33	153.72	14.06	15.00

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m³/day)	Peak Demand	Average Demand
J-109	11.52	95.16	13.28	14.56
J-110	11.44	428.12	13.34	14.63
J-111	10.49	4263.88	13.62	15.31
J-112	10.82	721.43	12.50	14.64
J-113	10.87	572.31	12.19	14.49
J-114	12.83	36.73	18.17	15.82
J-115	11.97	75.11	18.67	16.53
J-116	12.91	74.06	17.40	15.45
J-117	13.40	93.81	16.61	14.84
J-118	13.33	90.54	16.41	14.80
J-119	13.07	54.05	16.34	14.92
J-120	12.80	31.02	16.32	15.07
J-121	12.48	46.53	15.86	15.06
J-122	12.44	55.69	15.19	14.81
J-123	12.14	74.26	14.84	14.84
J-124	11.81	92.82	14.64	14.95
J-125	11.77	95.63	13.51	14.51
J-126	11.77	72.52	12.67	14.16
J-127	11.63	42.06	10.79	13.46
J-128	11.40	134.46	8.35	12.59
J-129	11.85	100.82	7.57	12.00
J-130	13.06	25.62	16.37	14.94
J-131	12.80	25.62	16.35	15.08
J-132	12.48	13.64	15.90	15.09
J-133	12.44	30.63	15.19	14.82
J-134	12.14	81.87	14.82	14.83
J-135	11.81	112.02	14.62	14.95
J-136	11.77	76.76	13.54	14.52
J-137	11.77	45.65	12.68	14.16
J-138	11.63	95.33	10.49	13.34
J-139	11.35	226.98	8.40	12.64
J-140	11.85	98.01	7.59	12.01
J-141	10.96	5.61	8.78	13.03
J-142	10.30	84.11	8.04	13.11
J-143	10.23	28.04	7.47	12.91
J-144	12.99	82.66	17.65	15.51
J-145	12.91	78.12	17.40	15.45
J-146	13.18	91.42	16.84	15.06
J-147	13.30	85.96	16.43	14.82
J-148	12.63	51.97	16.73	15.34
J-149	12.44	41.85	16.48	15.35
J-150	11.64	15.92	16.37	15.77
J-151	12.57	24.73	14.73	14.55
J-152	11.73	38.05	14.99	15.14
J-153	11.47	50.81	14.75	15.20
J-154	11.44	37.76	14.21	14.99
J-155	11.43	13.44	13.81	14.84
J-156	11.71	29.87	12.44	14.10
J-157	11.71	37.66	11.92	13.89
J-158	12.03	26.15	10.99	13.31
J-159	11.14	18.68	11.43	14.01
J-160	11.34	38.92	9.76	13.21
J-161	12.64	25.62	16.75	15.34
J-162	12.38	41.53	16.53	15.40
J-163	11.62	37.54	16.35	15.77

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m^3/day)	Peak Demand	Average Demand
J-164	13.01	34.24	14.30	14.11
J-165	11.75	47.56	14.96	15.12
J-166	11.54	64.54	14.68	15.13
J-167	11.44	53.92	14.13	14.96
J-168	11.40	37.34	13.83	14.86
J-169	11.72	41.82	12.43	14.09
J-170	11.73	42.01	11.28	13.61
J-171	12.05	42.96	10.32	13.02
J-172	11.14	28.03	10.86	13.78
J-173	11.30	20.24	9.80	13.25
J-174	11.07	15.57	9.90	13.42
J-175	11.09	65.39	9.13	13.09
J-176	11.14	34.25	8.38	12.75
J-222	14.11	32.35	13.96	13.33
J-223	14.29	20.43	10.16	11.65
J-224	14.57	27.24	6.90	10.14
J-225	15.25	37.86	3.99	8.53
J-226	15.73	52.69	0.15	6.66
J-227	15.78	53.09	-1.14	6.10
J-228	15.78	21.24	-1.55	5.93
J-250	15.28	0.00	18.15	14.38
J-251	15.55	0.00	18.02	14.16
J-256	14.08	23.84	24.01	17.51
J-257	14.01	70.41	23.44	17.31
J-258	13.87	54.54	22.70	17.08
J-259	14.59	60.95	22.36	16.53
J-260	15.08	51.80	21.43	15.85
J-261	15.27	44.40	20.82	15.49
J-262	15.21	84.54	20.46	15.37
J-263	15.77	124.76	19.48	14.64
J-264	15.69	61.05	19.22	14.58
J-265	15.51	0.00	18.99	14.59
J-266	15.70	0.00	18.41	14.24
J-268	14.13	0.00	18.89	15.36
J-269	15.24	0.00	17.42	14.10
J-270	15.32	0.00	16.91	13.84
J-271	15.27	9.56	16.55	13.72
J-272	15.54	93.66	15.90	13.30
J-273	15.28	150.05	15.77	13.39
J-274	14.88	94.62	15.72	13.61
J-275	14.29	72.91	15.90	14.03
J-276	14.60	88.48	15.42	13.64
J-277	14.93	88.48	14.79	13.19
J-278	13.56	77.42	15.91	14.46
J-279	11.43	77.42	17.74	16.46
J-280	12.42	44.24	16.22	15.25
J-281	12.42	0.00	16.42	15.33
J-282	11.42	0.00	17.09	16.19
J-283	11.99	0.00	16.34	15.55
J-284	11.38	0.00	16.62	16.02
J-285	11.37	0.00	16.29	15.90
J-286	11.20	0.00	16.13	15.92
J-287	11.15	1.53	15.93	15.87
J-288	12.47	1.53	14.27	14.41
J-289	12.44	30.49	13.73	14.21

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-290	12.11	70.12	13.12	14.15
J-291	12.89	112.80	11.84	13.17
J-292	12.46	6271.29	11.81	13.40
J-293	13.08	207.34	10.19	12.36
J-294	12.63	206.32	9.75	12.45
J-295	12.65	228.44	8.47	11.91
J-296	12.92	252.98	7.29	11.26
J-297	11.29	176.85	8.08	12.54
J-298	11.04	65.36	7.91	12.62
J-331	14.60	0.00	20.36	15.69
J-332	13.75	0.00	19.21	15.71
J-333	13.57	0.00	14.15	13.72
J-334	13.63	18.68	8.83	11.48
J-335	14.14	130.78	3.74	9.08
J-336	8.55	112.10	8.28	14.23
J-337	15.53	0.00	15.91	13.30
J-343	13.30	0.00	16.44	14.83
J-344	13.33	0.00	16.41	14.80
J-345	13.30	0.00	16.45	14.83

Table 10: Pipe data in 2025

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-2	J-1	J-2	5	582	21,319.00	0.93	13,241.00	0.58
P-3	J-2	J-3	544	517	14,971.00	0.82	9,299.00	0.51
P-4	J-3	J-4	500	57	33.00	0.15	21.00	0.09
P-5	J-4	J-5	527	57	11.00	0.05	7.00	0.03
P-6	J-1	J-6	200	582	24,484.00	1.07	15,208.00	0.66
P-7	J-2	J-7	195	582	24,509.00	1.07	15,223.00	0.66
P-8	J-7	J-8	6	582	13,592.00	0.59	8,442.00	0.37
P-9	J-7	J-6	6	582	9,545.00	0.42	5,929.00	0.26
P-10	J-6	J-9	354	416	10,408.00	0.89	6,465.00	0.55
P-11	J-9	J-10	525	416	10,290.00	0.88	6,391.00	0.55
P-12	J-10	J-11	506	416	10,167.00	0.87	6,315.00	0.54
P-13	J-11	J-12	615	416	10,005.00	0.85	6,214.00	0.53
P-14	J-12	J-13	582	416	9,947.00	0.85	6,178.00	0.53
P-15	J-13	J-14	569	416	9,869.00	0.84	6,130.00	0.52
P-16	J-14	J-15	499	416	9,755.00	0.83	6,059.00	0.52
P-17	J-15	J-16	416	416	9,359.00	0.80	5,813.00	0.50
P-18	J-16	J-17	251	369	6,491.00	0.70	4,031.00	0.44
P-19	J-7	J-18	491	148	1,327.00	0.90	824.00	0.56
P-20	J-18	J-19	500	148	1,282.00	0.87	797.00	0.54
P-21	J-19	J-20	320	148	1,230.00	0.83	764.00	0.52
P-23	J-8	J-22	492	68	160.00	0.51	99.00	0.32
P-24	J-22	J-23	500	57	125.00	0.57	78.00	0.35
P-25	J-23	J-24	315	57	74.00	0.34	46.00	0.21
P-26	J-24	J-25	576	57	59.00	0.27	37.00	0.17
P-27	J-21	J-20	572	57	(59.00)	0.27	(37.00)	0.17
P-28	J-24	J-26	494	81	430.00	0.96	267.00	0.59
P-29	J-26	J-27	500	57	211.00	0.96	131.00	0.60
P-30	J-27	J-28	591	57	56.00	0.25	35.00	0.16
P-31	J-8	J-29	9	582	13,290.00	0.58	8,255.00	0.36
P-33	J-30	J-31	342	416	10,393.00	0.89	6,455.00	0.55
P-34	J-31	J-32	527	416	10,293.00	0.88	6,393.00	0.55
P-35	J-32	J-33	506	416	10,187.00	0.87	6,327.00	0.54
P-36	J-33	J-34	616	416	10,116.00	0.86	6,283.00	0.54
P-37	J-34	J-35	582	416	9,992.00	0.85	6,206.00	0.53
P-38	J-35	J-36	567	416	9,832.00	0.84	6,107.00	0.52
P-39	J-36	J-37	500	416	9,578.00	0.82	5,949.00	0.51
P-40	J-37	J-38	413	416	9,528.00	0.81	5,918.00	0.50
P-41	J-16	J-39	5	208	2,735.00	0.93	1,699.00	0.58
P-42	J-39	J-40	251	369	6,425.00	0.69	3,991.00	0.43
P-43	J-29	J-41	398	462	13,130.00	0.91	8,155.00	0.56
P-44	J-41	J-42	500	462	13,019.00	0.90	8,086.00	0.56
P-45	J-42	J-43	502	462	12,931.00	0.89	8,032.00	0.56
P-46	J-43	J-44	497	462	12,860.00	0.89	7,988.00	0.55
P-47	J-44	J-45	491	462	12,774.00	0.88	7,934.00	0.55
P-48	J-45	J-46	395	462	12,702.00	0.88	7,889.00	0.55
P-49	J-46	J-47	499	462	12,608.00	0.87	7,831.00	0.54
P-50	J-47	J-48	502	462	12,522.00	0.87	7,778.00	0.54
P-51	J-48	J-49	504	462	12,530.00	0.87	7,783.00	0.54
P-52	J-49	J-50	501	462	12,424.00	0.86	7,717.00	0.53
P-53	J-50	J-51	500	462	12,303.00	0.85	7,642.00	0.53

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-54	J-51	J-52	502	462	12,199.00	0.84	7,577.00	0.52
P-55	J-52	J-53	502	462	12,008.00	0.83	7,459.00	0.52
P-56	J-53	J-54	103	462	11,523.00	0.80	7,157.00	0.49
P-57	J-53	J-55	495	57	227.00	1.03	141.00	0.64
P-58	J-55	J-56	377	57	58.00	0.26	36.00	0.16
P-59	J-30	J-57	393	462	13,098.00	0.91	8,135.00	0.56
P-60	J-57	J-58	500	462	12,996.00	0.90	8,072.00	0.56
P-61	J-58	J-59	500	462	12,910.00	0.89	8,019.00	0.55
P-62	J-59	J-60	501	462	12,852.00	0.89	7,982.00	0.55
P-63	J-60	J-61	493	462	12,777.00	0.88	7,936.00	0.55
P-64	J-61	J-62	397	462	12,632.00	0.87	7,846.00	0.54
P-65	J-62	J-63	500	462	12,570.00	0.87	7,807.00	0.54
P-66	J-63	J-64	501	462	12,563.00	0.87	7,803.00	0.54
P-67	J-64	J-65	500	462	12,449.00	0.86	7,732.00	0.53
P-68	J-65	J-66	500	462	12,416.00	0.86	7,712.00	0.53
P-69	J-66	J-67	500	462	12,343.00	0.85	7,667.00	0.53
P-70	J-67	J-68	500	462	12,253.00	0.85	7,611.00	0.53
P-71	J-68	J-69	500	462	12,089.00	0.84	7,509.00	0.52
P-72	J-69	J-70	104	462	11,957.00	0.83	7,426.00	0.51
P-73	J-70	J-71	5	462	3,247.00	0.22	2,016.00	0.14
P-74	J-70	J-72	480	416	8,588.00	0.73	5,334.00	0.46
P-75	J-72	J-73	499	416	8,382.00	0.72	5,206.00	0.44
P-76	J-73	J-74	500	416	8,237.00	0.70	5,116.00	0.44
P-77	J-74	J-75	501	416	8,193.00	0.70	5,089.00	0.43
P-78	J-75	J-76	499	416	8,166.00	0.70	5,072.00	0.43
P-79	J-76	J-77	501	416	8,105.00	0.69	5,034.00	0.43
P-80	J-77	J-78	500	369	7,999.00	0.86	4,968.00	0.54
P-81	J-78	J-79	500	369	7,796.00	0.84	4,842.00	0.52
P-82	J-79	J-80	501	369	7,458.00	0.81	4,632.00	0.50
P-83	J-80	J-81	500	369	7,216.00	0.78	4,482.00	0.48
P-84	J-81	J-82	493	369	6,966.00	0.75	4,327.00	0.47
P-85	J-82	J-83	400	185	(1,524.00)	0.66	(947.00)	0.41
P-86	J-83	J-84	397	185	(1,631.00)	0.71	(1,013.00)	0.44
P-87	J-84	J-85	429	185	(2,231.00)	0.96	(1,385.00)	0.60
P-88	J-85	J-86	378	185	(2,304.00)	1.00	(1,431.00)	0.62
P-89	J-86	J-87	500	208	(2,531.00)	0.86	(1,572.00)	0.54
P-90	J-87	J-38	503	208	(2,719.00)	0.93	(1,689.00)	0.58
P-91	J-39	J-88	504	208	2,834.00	0.97	1,760.00	0.60
P-92	J-88	J-89	500	208	2,501.00	0.85	1,554.00	0.53
P-93	J-89	J-90	374	185	2,232.00	0.97	1,386.00	0.60
P-94	J-90	J-91	431	185	2,069.00	0.89	1,285.00	0.56
P-95	J-91	J-92	397	185	1,729.00	0.75	1,074.00	0.46
P-96	J-92	J-93	402	185	1,640.00	0.71	1,019.00	0.44
P-97	J-93	J-94	579	369	6,690.00	0.72	4,155.00	0.45
P-98	J-91	J-95	338	68	128.00	0.41	79.00	0.25
P-99	J-84	J-96	498	81	455.00	1.01	283.00	0.63
P-100	J-96	J-97	548	81	220.00	0.49	137.00	0.30
P-102	J-61	J-98	190	57	11.00	0.05	7.00	0.03
P-103	J-71	J-99	480	416	8,937.00	0.76	5,551.00	0.47
P-104	J-99	J-100	500	416	8,742.00	0.75	5,430.00	0.46
P-105	J-100	J-101	500	416	8,567.00	0.73	5,321.00	0.45
P-106	J-101	J-102	500	416	8,396.00	0.72	5,215.00	0.44

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-107	J-102	J-103	499	416	8,137.00	0.69	5,054.00	0.43
P-108	J-103	J-104	503	416	7,934.00	0.68	4,928.00	0.42
P-109	J-104	J-105	498	369	7,743.00	0.84	4,810.00	0.52
P-110	J-105	J-106	499	369	7,517.00	0.81	4,669.00	0.50
P-111	J-106	J-107	500	369	7,366.00	0.80	4,575.00	0.49
P-112	J-107	J-108	500	369	7,198.00	0.78	4,471.00	0.48
P-113	J-108	J-109	500	369	6,950.00	0.75	4,317.00	0.47
P-114	J-109	J-82	8	369	(2,840.00)	0.31	(1,764.00)	0.19
P-115	J-109	J-110	5	369	9,637.00	1.04	5,986.00	0.65
P-116	J-110	J-111	577	369	6,865.00	0.74	4,264.00	0.46
P-118	J-110	J-112	389	185	2,082.00	0.90	1,293.00	0.56
P-119	J-112	J-113	316	185	921.00	0.40	572.00	0.25
P-120	J-54	J-114	8	462	8,649.00	0.60	5,372.00	0.37
P-122	J-54	J-115	503	291	2,801.00	0.49	1,740.00	0.30
P-123	J-115	J-116	501	291	2,680.00	0.47	1,665.00	0.29
P-124	J-116	J-117	500	291	2,560.00	0.45	1,590.00	0.28
P-126	J-118	J-119	507	231	1,442.00	0.40	895.00	0.25
P-127	J-119	J-120	501	231	1,354.00	0.37	841.00	0.23
P-128	J-120	J-121	497	185	1,304.00	0.56	810.00	0.35
P-129	J-121	J-122	503	185	1,229.00	0.53	763.00	0.33
P-130	J-122	J-123	497	185	1,170.00	0.51	727.00	0.31
P-131	J-123	J-124	499	185	1,051.00	0.45	653.00	0.28
P-132	J-124	J-125	495	148	902.00	0.61	560.00	0.38
P-133	J-125	J-126	504	148	747.00	0.51	464.00	0.31
P-134	J-126	J-127	498	115	631.00	0.70	392.00	0.43
P-135	J-127	J-128	812	115	562.00	0.62	349.00	0.39
P-136	J-128	J-129	183	81	163.00	0.36	101.00	0.22
P-137	J-118	J-130	511	231	1,377.00	0.38	855.00	0.24
P-138	J-130	J-131	501	231	1,335.00	0.37	829.00	0.23
P-139	J-131	J-132	498	185	1,293.00	0.56	803.00	0.35
P-140	J-132	J-133	501	185	1,271.00	0.55	789.00	0.34
P-141	J-133	J-134	496	185	1,192.00	0.52	740.00	0.32
P-142	J-134	J-135	498	185	1,059.00	0.46	658.00	0.28
P-143	J-135	J-136	495	148	879.00	0.59	546.00	0.37
P-144	J-136	J-137	502	148	755.00	0.51	469.00	0.32
P-145	J-137	J-138	499	115	681.00	0.75	423.00	0.47
P-146	J-138	J-139	812	115	528.00	0.58	328.00	0.36
P-147	J-139	J-140	183	81	157.00	0.35	98.00	0.22
P-148	J-139	J-128	5	115	(174.00)	0.19	(108.00)	0.12
P-149	J-128	J-141	230	57	8.00	0.04	5.00	0.02
P-150	J-139	J-142	651	81	179.00	0.40	111.00	0.25
P-151	J-142	J-143	689	57	45.00	0.20	28.00	0.13
P-152	J-114	J-144	494	291	2,805.00	0.49	1,742.00	0.30
P-153	J-144	J-145	500	291	2,671.00	0.47	1,659.00	0.29
P-154	J-145	J-146	500	291	2,546.00	0.44	1,581.00	0.28
P-156	J-147	J-148	496	185	866.00	0.37	538.00	0.23
P-157	J-148	J-149	442	166	783.00	0.42	486.00	0.26
P-158	J-149	J-150	581	148	716.00	0.48	445.00	0.30
P-159	J-150	J-151	490	148	691.00	0.47	429.00	0.29
P-160	J-151	J-152	481	148	630.00	0.43	391.00	0.26
P-161	J-152	J-153	499	148	569.00	0.38	353.00	0.24
P-162	J-153	J-154	502	148	602.00	0.41	374.00	0.25

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-163	J-154	J-155	447	148	541.00	0.37	336.00	0.23
P-164	J-155	J-156	555	115	426.00	0.47	264.00	0.29
P-165	J-156	J-157	531	115	292.00	0.32	181.00	0.20
P-166	J-157	J-158	470	99	231.00	0.34	143.00	0.21
P-167	J-158	J-159	499	99	189.00	0.28	117.00	0.18
P-168	J-159	J-160	353	68	159.00	0.51	98.00	0.32
P-169	J-147	J-161	498	185	839.00	0.36	521.00	0.23
P-170	J-161	J-162	447	166	798.00	0.43	495.00	0.26
P-171	J-162	J-163	581	148	731.00	0.49	454.00	0.31
P-172	J-163	J-164	490	148	671.00	0.45	417.00	0.28
P-173	J-164	J-165	483	148	636.00	0.43	395.00	0.27
P-174	J-165	J-166	499	148	560.00	0.38	348.00	0.24
P-175	J-166	J-167	499	115	339.00	0.38	211.00	0.23
P-176	J-167	J-168	447	115	253.00	0.28	157.00	0.17
P-177	J-168	J-169	558	99	287.00	0.43	178.00	0.27
P-178	J-169	J-170	526	99	305.00	0.45	189.00	0.28
P-179	J-170	J-171	470	99	237.00	0.35	147.00	0.22
P-180	J-171	J-172	501	99	167.00	0.25	104.00	0.15
P-181	J-172	J-173	351	68	122.00	0.39	76.00	0.24
P-182	J-173	J-174	365	57	25.00	0.11	16.00	0.07
P-183	J-173	J-160	7	81	65.00	0.15	41.00	0.09
P-184	J-160	J-175	493	81	161.00	0.36	100.00	0.22
P-185	J-175	J-176	500	57	56.00	0.25	35.00	0.16
P-231	J-24	J-20	7	148	(683.00)	0.46	(424.00)	0.29
P-232	J-20	J-222	501	81	395.00	0.88	245.00	0.55
P-233	J-222	J-223	501	81	344.00	0.76	213.00	0.47
P-234	J-223	J-224	500	81	310.00	0.69	193.00	0.43
P-235	J-224	J-225	500	81	266.00	0.59	165.00	0.37
P-236	J-225	J-226	500	68	204.00	0.66	127.00	0.41
P-237	J-226	J-227	500	68	120.00	0.38	74.00	0.24
P-238	J-227	J-228	701	57	35.00	0.16	22.00	0.10
P-261	J-250	J-251	174	517	(13,560.00)	0.75	(8,422.00)	0.46
P-266	J-3	J-256	411	517	14,883.00	0.82	9,244.00	0.51
P-267	J-256	J-257	681	517	14,844.00	0.82	9,220.00	0.51
P-268	J-11	J-258	713	57	26.00	0.12	16.00	0.07
P-269	J-258	J-257	509	57	(62.00)	0.28	(38.00)	0.17
P-270	J-257	J-259	539	517	14,670.00	0.81	9,112.00	0.50
P-271	J-259	J-260	501	517	14,150.00	0.78	8,789.00	0.48
P-272	J-260	J-261	500	517	14,066.00	0.77	8,737.00	0.48
P-273	J-261	J-262	501	517	13,995.00	0.77	8,693.00	0.48
P-274	J-262	J-263	501	517	13,859.00	0.76	8,608.00	0.47
P-275	J-263	J-264	427	517	13,659.00	0.75	8,484.00	0.47
P-276	J-264	J-265	505	517	13,560.00	0.75	8,422.00	0.46
P-277	J-265	J-266	496	517	13,560.00	0.75	8,422.00	0.46
P-279	J-266	J-251	675	517	13,560.00	0.75	8,422.00	0.46
P-280	J-250	J-268	504	517	13,560.00	0.75	8,422.00	0.46
P-281	J-268	J-269	458	517	13,560.00	0.75	8,422.00	0.46
P-282	J-269	J-270	539	517	13,560.00	0.75	8,422.00	0.46
P-283	J-270	J-271	505	517	13,560.00	0.75	8,422.00	0.46
P-285	J-272	J-273	500	517	13,394.00	0.74	8,319.00	0.46

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-286	J-273	J-274	587	517	13,152.00	0.72	8,169.00	0.45
P-287	J-274	J-275	558	517	12,999.00	0.72	8,074.00	0.44
P-288	J-275	J-276	227	517	12,882.00	0.71	8,002.00	0.44
P-289	J-276	J-277	432	517	12,741.00	0.70	7,913.00	0.44
P-290	J-277	J-278	358	517	12,599.00	0.69	7,825.00	0.43
P-291	J-278	J-279	427	517	12,473.00	0.69	7,747.00	0.43
P-292	J-280	J-279	242	57	(71.00)	0.32	(44.00)	0.20
P-293	J-279	J-281	500	517	12,277.00	0.68	7,626.00	0.42
P-294	J-281	J-282	494	517	12,277.00	0.68	7,626.00	0.42
P-295	J-282	J-283	276	517	12,277.00	0.68	7,626.00	0.42
P-296	J-283	J-284	498	517	12,277.00	0.68	7,626.00	0.42
P-297	J-284	J-285	500	517	12,277.00	0.68	7,626.00	0.42
P-298	J-285	J-286	503	517	12,277.00	0.68	7,626.00	0.42
P-299	J-286	J-287	378	517	12,277.00	0.68	7,626.00	0.42
P-300	J-287	J-288	502	517	12,275.00	0.68	7,624.00	0.42
P-301	J-288	J-289	500	462	12,272.00	0.85	7,622.00	0.53
P-302	J-289	J-290	822	462	12,223.00	0.84	7,592.00	0.52
P-303	J-290	J-291	447	462	12,110.00	0.84	7,522.00	0.52
P-304	J-291	J-292	418	462	11,928.00	0.82	7,409.00	0.51
P-305	J-292	J-293	338	185	1,832.00	0.79	1,138.00	0.49
P-306	J-293	J-294	436	185	1,498.00	0.65	931.00	0.40
P-307	J-294	J-295	982	185	1,166.00	0.50	724.00	0.31
P-308	J-295	J-296	483	148	798.00	0.54	496.00	0.34
P-309	J-296	J-297	499	115	391.00	0.43	243.00	0.27
P-310	J-297	J-298	513	81	106.00	0.24	66.00	0.15
P-345	J-46	J-62	5	416	3.00	0.00	2.00	0.00
P-350	J-85	J-90	5	185	(79.00)	0.03	(49.00)	0.02
P-354	J-38	J-39	5	416	6,631.00	0.57	4,119.00	0.35
P-356	J-118	J-147	17	291	(555.00)	0.10	(345.00)	0.06
P-358	J-122	J-133	5	185	(31.00)	0.01	(19.00)	0.01
P-365	J-259	J-331	501	99	421.00	0.63	262.00	0.39
P-366	J-331	J-332	501	99	421.00	0.63	262.00	0.39
P-367	J-332	J-333	498	81	421.00	0.94	262.00	0.58
P-368	J-333	J-334	500	81	421.00	0.94	262.00	0.58
P-369	J-334	J-335	501	81	391.00	0.87	243.00	0.54
P-370	J-335	J-336	474	81	181.00	0.40	112.00	0.25
P-372	J-271	J-337	483	517	13,545.00	0.75	8,413.00	0.46
P-373	J-337	J-272	2	517	13,545.00	0.75	8,413.00	0.46
P-388	R-1	PMP-7	16	517	12,802.00	0.71	13,252.00	0.73
P-389	PMP-7	J-1	27	517	12,802.00	0.71	13,252.00	0.73
P-416	J-343	J-147	14	291	2,399.00	0.42	1,490.00	0.26
P-418	J-117	J-344	511	291	2,409.00	0.42	1,496.00	0.26
P-419	J-344	J-118	5	291	2,409.00	0.42	1,496.00	0.26
P-431	J-146	J-345	512	291	2,399.00	0.42	1,490.00	0.26
P-434	J-345	J-343	4	291	2,399.00	0.42	1,490.00	0.26
P-439	J-25	J-21	5	57	(43.00)	0.19	(26.00)	0.12
P-440	J-114	J-71	6	416	5,785.00	0.49	3,593.00	0.31
P-441	R-1	PMP-8	16	517	12,802.00	0.71	13,252.00	0.73
P-442	PMP-8	J-1	27	517	12,802.00	0.71	13,252.00	0.73
P-444	J-1	J-2	7	582	18,195.00	0.79	11,301.00	0.49
P-445	J-6	J-30	14	582	23,537.00	1.02	14,619.00	0.64
P-450	J-82	J-93	9	369	5,377.00	0.58	3,340.00	0.36

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-454	R-1	PMP-9	16	517	12,802.00	0.71	13,252.00	0.73
P-455	PMP-9	J-1	27	517	12,802.00	0.71	13,252.00	0.73
P-457	J-168	J-155	5	115	(93.00)	0.1	(58.00)	0.06
P-459	R-1	PMP-10	17	517	12802.00	0.71	-	-
P-460	PMP-10	J-1	27	517	12802.00	0.71	-	-
P-461	R-1	PMP-11	16	517	12802.00	0.71	-	-
P-462	PMP-11	J-1	28	517	12802.00	0.71	-	-

Table 11: Node data in 2040

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-1	13.99	8.99	50.0	25.0
J-2	14.01	26.97	50.0	25.0
J-3	14.84	44.96	48.0	24.0
J-4	14.71	17.98	48.0	24.0
J-5	14.69	8.99	48.0	24.0
J-6	15.01	69.68	48.0	24.0
J-7	15.01	35.96	48.0	24.0
J-8	15.11	118.90	48.0	24.0
J-9	14.70	96.65	47.0	23.0
J-10	14.80	100.70	45.0	23.0
J-11	14.89	111.86	43.0	22.0
J-12	14.65	47.66	41.0	21.0
J-13	15.36	63.55	38.0	19.0
J-14	12.63	93.74	39.0	21.0
J-15	12.45	109.62	38.0	21.0
J-16	13.75	109.62	35.0	19.0
J-17	12.47	7664.51	35.0	20.0
J-18	14.89	35.96	45.0	22.0
J-19	15.29	43.83	40.0	20.0
J-20	15.14	75.09	38.0	19.0
J-21	14.16	13.49	38.0	20.0
J-22	14.89	28.85	45.0	22.0
J-23	15.29	42.86	39.0	20.0
J-24	15.05	225.82	38.0	19.0
J-25	14.16	85.52	38.0	20.0
J-26	13.75	182.92	31.0	17.0
J-27	12.84	130.96	17.0	12.0
J-28	12.82	46.60	16.0	11.0
J-29	15.15	134.49	48.0	24.0
J-30	15.15	38.07	48.0	24.0
J-31	14.70	83.74	47.0	23.0
J-32	14.80	88.50	45.0	23.0
J-33	15.14	59.70	43.0	21.0
J-34	14.65	104.48	41.0	21.0
J-35	15.36	136.46	38.0	19.0
J-36	12.63	228.61	39.0	21.0
J-37	12.45	284.96	38.0	21.0
J-38	12.60	162.83	36.0	20.0
J-39	12.46	98.01	36.0	20.0
J-40	12.47	7611.45	35.0	20.0
J-41	14.39	93.48	48.0	24.0
J-42	14.06	73.42	47.0	24.0
J-43	14.10	59.77	45.0	23.0
J-44	14.01	47.61	44.0	22.0
J-45	13.25	60.52	43.0	22.0
J-46	13.49	75.74	42.0	22.0
J-47	13.26	72.87	40.0	21.0
J-48	13.22	90.97	39.0	21.0
J-49	13.54	98.90	37.0	20.0
J-50	13.58	113.03	36.0	19.0
J-51	13.25	96.92	35.0	19.0
J-52	13.25	177.02	33.0	18.0
J-53	12.88	242.84	32.0	18.0

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-54	12.04	67.77	33.0	19.0
J-55	11.64	157.73	14.0	11.0
J-56	11.30	54.22	13.0	11.0
J-57	14.39	84.91	48.0	24.0
J-58	14.06	72.29	47.0	24.0
J-59	14.10	49.42	45.0	23.0
J-60	14.01	87.94	44.0	22.0
J-61	13.26	111.97	43.0	22.0
J-62	13.49	54.52	42.0	22.0
J-63	13.26	6.07	40.0	21.0
J-64	13.22	2.83	39.0	21.0
J-65	13.54	31.08	37.0	20.0
J-66	13.58	67.82	36.0	19.0
J-67	13.25	84.78	35.0	19.0
J-68	13.25	153.14	33.0	18.0
J-69	12.88	124.36	32.0	18.0
J-70	13.06	115.11	32.0	18.0
J-71	12.85	88.86	32.0	18.0
J-72	12.78	193.22	31.0	18.0
J-73	12.28	135.66	30.0	18.0
J-74	12.46	41.11	29.0	17.0
J-75	12.45	20.57	28.0	16.0
J-76	11.26	47.99	28.0	17.0
J-77	12.20	82.27	26.0	16.0
J-78	11.28	158.18	25.0	16.0
J-79	11.70	263.87	22.0	14.0
J-80	11.24	189.50	21.0	14.0
J-81	11.33	195.84	19.0	13.0
J-82	11.58	213.22	17.0	12.0
J-83	11.19	88.14	20.0	14.0
J-84	11.62	130.89	22.0	14.0
J-85	11.44	139.82	26.0	16.0
J-86	11.61	207.55	30.0	18.0
J-87	11.69	171.59	33.0	19.0
J-88	11.69	303.69	33.0	19.0
J-89	11.61	247.11	29.0	18.0
J-90	11.44	76.42	26.0	16.0
J-91	11.62	193.48	22.0	14.0
J-92	11.19	77.52	20.0	14.0
J-93	11.48	259.15	17.0	12.0
J-94	10.49	7816.69	16.0	13.0
J-95	11.70	117.06	20.0	13.0
J-96	11.00	185.88	13.0	11.0
J-97	11.38	171.57	10.0	9.0
J-98	12.74	9.07	43.0	23.0
J-99	12.78	181.79	31.0	18.0
J-100	12.28	164.42	30.0	17.0
J-101	12.46	160.84	29.0	17.0
J-102	12.45	232.69	27.0	16.0
J-103	11.26	174.44	27.0	17.0
J-104	12.20	148.71	25.0	16.0
J-105	11.28	177.33	24.0	16.0
J-106	11.70	117.24	22.0	14.0
J-107	11.24	131.12	21.0	14.0
J-108	11.33	193.22	19.0	13.0

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-109	11.52	119.62	17.0	12.0
J-110	11.44	538.15	17.0	12.0
J-111	10.49	7953.25	16.0	12.0
J-112	10.82	1056.85	15.0	12.0
J-113	10.87	869.41	14.0	11.0
J-114	12.83	55.37	32.0	18.0
J-115	11.97	113.24	32.0	19.0
J-116	12.91	111.66	31.0	17.0
J-117	13.40	141.43	30.0	17.0
J-118	13.33	136.31	29.0	16.0
J-119	13.07	81.50	29.0	16.0
J-120	12.80	46.77	28.0	16.0
J-121	12.48	69.01	27.0	16.0
J-122	12.44	82.26	26.0	16.0
J-123	12.14	109.68	25.0	15.0
J-124	11.81	137.10	24.0	15.0
J-125	11.77	141.24	22.0	14.0
J-126	11.77	107.10	20.0	14.0
J-127	11.63	62.11	16.0	12.0
J-128	11.40	198.60	11.0	10.0
J-129	11.85	148.90	10.0	9.0
J-130	13.06	37.84	29.0	16.0
J-131	12.80	37.84	28.0	17.0
J-132	12.48	20.14	27.0	16.0
J-133	12.44	45.23	26.0	16.0
J-134	12.14	120.92	25.0	15.0
J-135	11.81	165.45	24.0	15.0
J-136	11.77	113.38	22.0	14.0
J-137	11.77	67.43	20.0	14.0
J-138	11.63	140.79	16.0	12.0
J-139	11.35	335.25	11.0	10.0
J-140	11.85	144.76	10.0	9.0
J-141	10.96	8.28	12.0	10.0
J-142	10.30	124.23	9.0	10.0
J-143	10.23	41.41	8.0	9.0
J-144	12.99	124.62	31.0	18.0
J-145	12.91	117.79	31.0	17.0
J-146	13.18	137.83	30.0	17.0
J-147	13.30	129.01	29.0	17.0
J-148	12.63	76.98	29.0	17.0
J-149	12.44	61.81	28.0	17.0
J-150	11.64	23.51	27.0	17.0
J-151	12.57	36.53	25.0	15.0
J-152	11.73	56.19	25.0	16.0
J-153	11.47	75.05	24.0	15.0
J-154	11.44	55.77	23.0	15.0
J-155	11.43	19.85	22.0	15.0
J-156	11.71	44.12	20.0	13.0
J-157	11.71	55.62	19.0	13.0
J-158	12.03	38.62	17.0	12.0
J-159	11.14	27.59	17.0	13.0
J-160	11.34	57.49	14.0	11.0
J-161	12.64	37.84	29.0	17.0
J-162	12.38	61.34	28.0	17.0
J-163	11.62	55.44	27.0	17.0

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-164	13.01	50.57	25.0	15.0
J-165	11.75	70.24	25.0	15.0
J-166	11.54	95.32	24.0	15.0
J-167	11.44	79.65	23.0	15.0
J-168	11.40	55.15	22.0	15.0
J-169	11.72	61.77	20.0	13.0
J-170	11.73	62.05	17.0	12.0
J-171	12.05	63.45	16.0	11.0
J-172	11.14	41.39	16.0	12.0
J-173	11.30	29.89	14.0	11.0
J-174	11.07	23.00	14.0	11.0
J-175	11.09	96.58	12.0	11.0
J-176	11.14	50.59	11.0	10.0
J-222	14.11	42.71	32.0	17.0
J-223	14.29	26.97	26.0	15.0
J-224	14.57	35.96	21.0	12.0
J-225	15.25	49.98	16.0	10.0
J-226	15.73	69.56	11.0	7.0
J-227	15.78	70.09	8.0	6.0
J-228	15.78	28.03	8.0	6.0
J-250	15.28	0.00	36.0	18.0
J-251	15.55	0.00	36.0	18.0
J-256	14.08	31.47	48.0	24.0
J-257	14.01	92.95	46.0	23.0
J-258	13.87	72.00	44.0	23.0
J-259	14.59	80.47	45.0	22.0
J-260	15.08	68.39	43.0	21.0
J-261	15.27	58.61	42.0	21.0
J-262	15.21	111.61	41.0	21.0
J-263	15.77	164.70	39.0	20.0
J-264	15.69	80.60	39.0	19.0
J-265	15.51	0.00	38.0	19.0
J-266	15.70	0.00	37.0	18.0
J-268	14.13	0.00	36.0	19.0
J-269	15.24	0.00	34.0	17.0
J-270	15.32	0.00	33.0	17.0
J-271	15.27	13.43	32.0	17.0
J-272	15.54	131.58	31.0	16.0
J-273	15.28	210.79	30.0	16.0
J-274	14.88	132.92	29.0	16.0
J-275	14.29	107.76	29.0	16.0
J-276	14.60	134.97	28.0	15.0
J-277	14.93	134.97	27.0	15.0
J-278	13.56	118.10	28.0	16.0
J-279	11.43	118.10	29.0	18.0
J-280	12.42	67.49	27.0	16.0
J-281	12.42	0.00	28.0	16.0
J-282	11.42	0.00	28.0	17.0
J-283	11.99	0.00	27.0	16.0
J-284	11.38	0.00	27.0	16.0
J-285	11.37	0.00	26.0	16.0
J-286	11.20	0.00	25.0	16.0
J-287	11.15	2.31	25.0	16.0
J-288	12.47	2.31	22.0	14.0
J-289	12.44	43.59	21.0	14.0

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-290	12.11	100.27	19.0	13.0
J-291	12.89	161.30	17.0	12.0
J-292	12.46	10761.81	16.0	12.0
J-293	13.08	274.59	14.0	10.0
J-294	12.63	273.13	13.0	10.0
J-295	12.65	311.92	11.0	9.0
J-296	12.92	354.11	9.0	8.0
J-297	11.29	246.37	9.0	9.0
J-298	11.04	90.65	9.0	9.0
J-331	14.60	0.00	41.0	21.0
J-332	13.75	0.00	39.0	21.0
J-333	13.57	0.00	31.0	17.0
J-334	13.63	24.66	22.0	13.0
J-335	14.14	172.65	14.0	10.0
J-336	8.55	147.99	18.0	15.0
J-337	15.53	0.00	31.0	16.0
J-343	13.30	0.00	29.0	17.0
J-344	13.33	0.00	29.0	16.0
J-345	13.30	0.00	29.0	17.0

Table 12: Pipe data in 2040

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-2	J-1	J-2	5	581.8	34,216.0	1.49	21,656.0	0.94
P-3	J-2	J-3	544	517.2	23,478.0	1.29	14,860.0	0.82
P-4	J-3	J-4	500	57.0	42.0	0.19	27.0	0.12
P-5	J-4	J-5	527	57.0	14.0	0.06	9.0	0.04
P-6	J-1	J-6	200	581.8	39,861.0	1.74	25,228.0	1.10
P-7	J-2	J-7	195	581.8	39,897.0	1.74	25,251.0	1.10
P-8	J-7	J-8	6	581.8	21,424.0	0.93	13,560.0	0.59
P-9	J-7	J-6	6	581.8	16,672.0	0.73	10,552.0	0.46
P-10	J-6	J-9	354	415.6	17,796.0	1.52	11,263.0	0.96
P-11	J-9	J-10	525	415.6	17,643.0	1.51	11,166.0	0.95
P-12	J-10	J-11	506	415.6	17,483.0	1.49	11,065.0	0.94
P-13	J-11	J-12	615	415.6	17,299.0	1.48	10,949.0	0.93
P-14	J-12	J-13	582	415.6	17,224.0	1.47	10,901.0	0.93
P-15	J-13	J-14	569	415.6	17,123.0	1.46	10,837.0	0.92
P-16	J-14	J-15	499	415.6	16,975.0	1.45	10,744.0	0.92
P-17	J-15	J-16	416	415.6	17,173.0	1.47	10,869.0	0.93
P-18	J-16	J-17	251	369.4	11,988.0	1.29	7,588.0	0.82
P-19	J-7	J-18	491	147.6	1,743.0	1.18	1,103.0	0.75
P-20	J-18	J-19	500	147.6	1,686.0	1.14	1,067.0	0.72
P-21	J-19	J-20	320	147.6	1,616.0	1.09	1,023.0	0.69
P-23	J-8	J-22	492	67.8	210.0	0.67	133.0	0.43
P-24	J-22	J-23	500	57.0	165.0	0.75	104.0	0.47
P-25	J-23	J-24	315	57.0	97.0	0.44	61.0	0.28
P-26	J-24	J-25	576	57.0	78.0	0.35	50.0	0.22
P-27	J-21	J-20	572	57.0	(79.0)	0.36	(50.0)	0.23
P-28	J-24	J-26	494	81.4	571.0	1.27	361.0	0.80
P-29	J-26	J-27	500	57.0	281.0	1.28	178.0	0.81
P-30	J-27	J-28	591	57.0	74.0	0.33	47.0	0.21
P-31	J-8	J-29	9	581.8	21,026.0	0.92	13,308.0	0.58
P-33	J-30	J-31	342	415.6	17,808.0	1.52	11,271.0	0.96
P-34	J-31	J-32	527	415.6	17,675.0	1.51	11,187.0	0.95
P-35	J-32	J-33	506	415.6	17,536.0	1.50	11,099.0	0.95
P-36	J-33	J-34	616	415.6	17,442.0	1.49	11,039.0	0.94
P-37	J-34	J-35	582	415.6	17,277.0	1.47	10,935.0	0.93
P-38	J-35	J-36	567	415.6	17,061.0	1.46	10,798.0	0.92
P-39	J-36	J-37	500	415.6	16,699.0	1.42	10,569.0	0.90
P-40	J-37	J-38	413	415.6	15,877.0	1.35	10,049.0	0.86
P-41	J-16	J-39	5	207.8	5,012.0	1.71	3,172.0	1.08
P-42	J-39	J-40	251	369.4	11,905.0	1.29	7,535.0	0.81
P-43	J-29	J-41	398	461.8	20,813.0	1.44	13,173.0	0.91
P-44	J-41	J-42	500	461.8	20,666.0	1.43	13,080.0	0.90
P-45	J-42	J-43	502	461.8	20,550.0	1.42	13,006.0	0.90
P-46	J-43	J-44	497	461.8	20,455.0	1.41	12,946.0	0.89
P-47	J-44	J-45	491	461.8	20,338.0	1.41	12,872.0	0.89
P-48	J-45	J-46	395	461.8	20,243.0	1.40	12,812.0	0.89
P-49	J-46	J-47	499	461.8	20,115.0	1.39	12,731.0	0.88
P-50	J-47	J-48	502	461.8	20,000.0	1.38	12,658.0	0.87
P-51	J-48	J-49	504	461.8	20,009.0	1.38	12,664.0	0.88
P-52	J-49	J-50	501	461.8	19,854.0	1.37	12,566.0	0.87
P-53	J-50	J-51	500	461.8	19,675.0	1.36	12,452.0	0.86
P-54	J-51	J-52	502	461.8	19,522.0	1.35	12,356.0	0.85

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-55	J-52	J-53	502	461.8	19,242.0	1.33	12,178.0	0.84
P-56	J-53	J-54	103	461.8	18,523.0	1.28	11,723.0	0.81
P-57	J-53	J-55	495	57.0	336.0	1.52	213.0	0.96
P-58	J-55	J-56	377	57.0	86.0	0.39	54.0	0.25
P-59	J-30	J-57	393	461.8	20,758.0	1.43	13,138.0	0.91
P-60	J-57	J-58	500	461.8	20,624.0	1.43	13,053.0	0.90
P-61	J-58	J-59	500	461.8	20,509.0	1.42	12,981.0	0.90
P-62	J-59	J-60	501	461.8	20,431.0	1.41	12,931.0	0.89
P-63	J-60	J-61	493	461.8	20,334.0	1.41	12,870.0	0.89
P-64	J-61	J-62	397	461.8	20,142.0	1.39	12,748.0	0.88
P-65	J-62	J-63	500	461.8	20,064.0	1.39	12,698.0	0.88
P-66	J-63	J-64	501	461.8	20,054.0	1.39	12,692.0	0.88
P-67	J-64	J-65	500	461.8	19,897.0	1.37	12,593.0	0.87
P-68	J-65	J-66	500	461.8	19,848.0	1.37	12,562.0	0.87
P-69	J-66	J-67	500	461.8	19,742.0	1.36	12,495.0	0.86
P-70	J-67	J-68	500	461.8	19,608.0	1.35	12,410.0	0.86
P-71	J-68	J-69	500	461.8	19,366.0	1.34	12,257.0	0.85
P-72	J-69	J-70	104	461.8	19,170.0	1.32	12,133.0	0.84
P-73	J-70	J-71	5	461.8	4,733.0	0.33	2,996.0	0.21
P-74	J-70	J-72	480	415.6	14,255.0	1.22	9,022.0	0.77
P-75	J-72	J-73	499	415.6	13,949.0	1.19	8,829.0	0.75
P-76	J-73	J-74	500	415.6	13,735.0	1.17	8,693.0	0.74
P-77	J-74	J-75	501	415.6	13,670.0	1.17	8,652.0	0.74
P-78	J-75	J-76	499	415.6	13,637.0	1.16	8,631.0	0.74
P-79	J-76	J-77	501	415.6	13,560.0	1.16	8,582.0	0.73
P-80	J-77	J-78	500	369.4	13,431.0	1.45	8,500.0	0.92
P-81	J-78	J-79	500	369.4	13,181.0	1.42	8,342.0	0.90
P-82	J-79	J-80	501	369.4	12,764.0	1.38	8,079.0	0.87
P-83	J-80	J-81	500	369.4	12,465.0	1.35	7,890.0	0.85
P-84	J-81	J-82	493	369.4	12,156.0	1.31	7,693.0	0.83
P-85	J-82	J-83	400	184.6	(2,559.0)	1.11	(1,620.0)	0.70
P-86	J-83	J-84	397	184.6	(2,698.0)	1.17	(1,708.0)	0.74
P-87	J-84	J-85	429	184.6	(3,470.0)	1.50	(2,196.0)	0.95
P-88	J-85	J-86	378	184.6	(3,628.0)	1.57	(2,296.0)	0.99
P-89	J-86	J-87	500	207.8	(3,955.0)	1.35	(2,503.0)	0.85
P-90	J-87	J-38	503	207.8	(4,227.0)	1.44	(2,675.0)	0.91
P-91	J-39	J-88	504	207.8	4,346.0	1.48	2,751.0	0.94
P-92	J-88	J-89	500	207.8	3,867.0	1.32	2,447.0	0.84
P-93	J-89	J-90	374	184.6	3,476.0	1.50	2,200.0	0.95
P-94	J-90	J-91	431	184.6	3,293.0	1.42	2,084.0	0.90
P-95	J-91	J-92	397	184.6	2,803.0	1.21	1,774.0	0.77
P-96	J-92	J-93	402	184.6	2,680.0	1.16	1,696.0	0.73
P-97	J-93	J-94	579	369.4	12,229.0	1.32	7,740.0	0.84
P-98	J-91	J-95	338	67.8	184.0	0.59	117.0	0.37
P-99	J-84	J-96	498	81.4	565.0	1.26	358.0	0.80
P-100	J-96	J-97	548	81.4	272.0	0.60	172.0	0.38
P-102	J-61	J-98	190	57.0	15.0	0.07	10.0	0.04
P-103	J-71	J-99	480	415.6	14,763.0	1.26	9,344.0	0.80
P-104	J-99	J-100	500	415.6	14,477.0	1.24	9,162.0	0.78
P-105	J-100	J-101	500	415.6	14,217.0	1.21	8,998.0	0.77
P-106	J-101	J-102	500	415.6	13,963.0	1.19	8,838.0	0.75
P-107	J-102	J-103	499	415.6	13,596.0	1.16	8,605.0	0.73
P-108	J-103	J-104	503	415.6	13,320.0	1.14	8,431.0	0.72

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-109	J-104	J-105	498	369.4	13,086.0	1.41	8,282.0	0.89
P-110	J-105	J-106	499	369.4	12,806.0	1.38	8,105.0	0.88
P-111	J-106	J-107	500	369.4	12,620.0	1.36	7,987.0	0.86
P-112	J-107	J-108	500	369.4	12,413.0	1.34	7,856.0	0.85
P-113	J-108	J-109	500	369.4	12,107.0	1.31	7,663.0	0.83
P-114	J-109	J-82	8	369.4	(4,419.0)	0.48	(2,797.0)	0.30
P-115	J-109	J-110	5	369.4	16,338.0	1.76	10,340.0	1.12
P-116	J-110	J-111	577	369.4	12,444.0	1.34	7,876.0	0.85
P-118	J-110	J-112	389	184.6	3,043.0	1.32	1,926.0	0.83
P-119	J-112	J-113	316	184.6	1,373.0	0.59	869.0	0.38
P-120	J-54	J-114	8	461.8	14,339.0	0.99	9,075.0	0.63
P-122	J-54	J-115	503	290.8	4,077.0	0.71	2,580.0	0.45
P-123	J-115	J-116	501	290.8	3,898.0	0.68	2,467.0	0.43
P-124	J-116	J-117	500	290.8	3,722.0	0.65	2,356.0	0.41
P-126	J-118	J-119	507	230.8	2,092.0	0.58	1,324.0	0.37
P-127	J-119	J-120	501	230.8	1,964.0	0.54	1,243.0	0.34
P-128	J-120	J-121	497	184.6	1,890.0	0.82	1,196.0	0.52
P-129	J-121	J-122	503	184.6	1,781.0	0.77	1,127.0	0.49
P-130	J-122	J-123	497	184.6	1,697.0	0.73	1,074.0	0.46
P-131	J-123	J-124	499	184.6	1,524.0	0.66	965.0	0.42
P-132	J-124	J-125	495	147.6	1,307.0	0.88	827.0	0.56
P-133	J-125	J-126	504	147.6	1,085.0	0.73	686.0	0.46
P-134	J-126	J-127	498	115.4	915.0	1.01	579.0	0.64
P-135	J-127	J-128	812	115.4	817.0	0.90	517.0	0.57
P-136	J-128	J-129	183	81.4	235.0	0.52	149.0	0.33
P-137	J-118	J-130	511	230.8	1,995.0	0.55	1,263.0	0.35
P-138	J-130	J-131	501	230.8	1,935.0	0.54	1,225.0	0.34
P-139	J-131	J-132	498	184.6	1,875.0	0.81	1,187.0	0.51
P-140	J-132	J-133	501	184.6	1,843.0	0.80	1,167.0	0.50
P-141	J-133	J-134	496	184.6	1,726.0	0.75	1,093.0	0.47
P-142	J-134	J-135	498	184.6	1,535.0	0.66	972.0	0.42
P-143	J-135	J-136	495	147.6	1,274.0	0.86	807.0	0.55
P-144	J-136	J-137	502	147.6	1,096.0	0.74	693.0	0.47
P-145	J-137	J-138	499	115.4	989.0	1.09	626.0	0.69
P-146	J-138	J-139	812	115.4	767.0	0.85	485.0	0.54
P-147	J-139	J-140	183	81.4	229.0	0.51	145.0	0.32
P-148	J-139	J-128	5	115.4	(255.0)	0.28	(161.0)	0.18
P-149	J-128	J-141	230	57.0	14.0	0.06	9.0	0.04
P-150	J-139	J-142	651	81.4	262.0	0.58	166.0	0.37
P-151	J-142	J-143	689	57.0	66.0	0.30	41.0	0.19
P-152	J-114	J-144	494	290.8	4,081.0	0.71	2,583.0	0.45
P-153	J-144	J-145	500	290.8	3,884.0	0.68	2,458.0	0.43
P-154	J-145	J-146	500	290.8	3,699.0	0.64	2,341.0	0.41
P-156	J-147	J-148	496	184.6	1,256.0	0.54	795.0	0.34
P-157	J-148	J-149	442	166.2	1,135.0	0.61	718.0	0.38
P-158	J-149	J-150	581	147.6	1,037.0	0.70	656.0	0.44
P-159	J-150	J-151	490	147.6	1,000.0	0.68	633.0	0.43
P-160	J-151	J-152	481	147.6	912.0	0.62	577.0	0.39
P-161	J-152	J-153	499	147.6	823.0	0.56	521.0	0.35
P-162	J-153	J-154	502	147.6	873.0	0.59	552.0	0.37
P-163	J-154	J-155	447	147.6	784.0	0.53	496.0	0.34
P-164	J-155	J-156	555	115.4	617.0	0.68	390.0	0.43
P-165	J-156	J-157	531	115.4	423.0	0.47	268.0	0.30

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-166	J-157	J-158	470	99.4	336.0	0.50	213.0	0.32
P-167	J-158	J-159	499	99.4	275.0	0.41	174.0	0.26
P-168	J-159	J-160	353	67.8	231.0	0.74	146.0	0.47
P-169	J-147	J-161	498	184.6	1,216.0	0.53	770.0	0.33
P-170	J-161	J-162	447	166.2	1,156.0	0.62	732.0	0.39
P-171	J-162	J-163	581	147.6	1,059.0	0.72	670.0	0.45
P-172	J-163	J-164	490	147.6	971.0	0.66	615.0	0.42
P-173	J-164	J-165	483	147.6	921.0	0.62	583.0	0.39
P-174	J-165	J-166	499	147.6	811.0	0.55	513.0	0.35
P-175	J-166	J-167	499	115.4	492.0	0.54	312.0	0.34
P-176	J-167	J-168	447	115.4	367.0	0.41	232.0	0.26
P-177	J-168	J-169	558	99.4	415.0	0.62	263.0	0.39
P-178	J-169	J-170	526	99.4	442.0	0.66	280.0	0.42
P-179	J-170	J-171	470	99.4	344.0	0.51	218.0	0.32
P-180	J-171	J-172	501	99.4	244.0	0.36	155.0	0.23
P-181	J-172	J-173	351	67.8	179.0	0.57	113.0	0.36
P-182	J-173	J-174	365	57.0	37.0	0.17	23.0	0.11
P-183	J-173	J-160	7	81.4	94.0	0.21	59.0	0.13
P-184	J-160	J-175	493	81.4	233.0	0.52	148.0	0.33
P-185	J-175	J-176	500	57.0	81.0	0.37	51.0	0.23
P-231	J-24	J-20	7	147.6	(908.0)	0.61	(575.0)	0.39
P-232	J-20	J-222	501	81.4	511.0	1.14	323.0	0.72
P-233	J-222	J-223	501	81.4	444.0	0.99	281.0	0.62
P-234	J-223	J-224	500	81.4	401.0	0.89	254.0	0.56
P-235	J-224	J-225	500	81.4	344.0	0.77	218.0	0.48
P-236	J-225	J-226	500	67.8	265.0	0.85	168.0	0.54
P-237	J-226	J-227	500	67.8	154.0	0.49	98.0	0.31
P-238	J-227	J-228	701	57.0	44.0	0.20	28.0	0.13
P-261	J-250	J-251	174	517.2	(21,626.0)	1.19	(13,687.0)	0.75
P-266	J-3	J-256	411	517.2	23,365.0	1.29	14,788.0	0.81
P-267	J-256	J-257	681	517.2	23,316.0	1.28	14,757.0	0.81
P-268	J-11	J-258	713	57.0	8.0	0.04	5.0	0.02
P-269	J-258	J-257	509	57.0	(105.0)	0.48	(66.0)	0.30
P-270	J-257	J-259	539	517.2	23,064.0	1.27	14,597.0	0.80
P-271	J-259	J-260	501	517.2	22,391.0	1.23	14,171.0	0.78
P-272	J-260	J-261	500	517.2	22,283.0	1.23	14,103.0	0.78
P-273	J-261	J-262	501	517.2	22,190.0	1.22	14,044.0	0.77
P-274	J-262	J-263	501	517.2	22,014.0	1.21	13,933.0	0.77
P-275	J-263	J-264	427	517.2	21,753.0	1.20	13,768.0	0.76
P-276	J-264	J-265	505	517.2	21,626.0	1.19	13,687.0	0.75
P-277	J-265	J-266	496	517.2	21,626.0	1.19	13,687.0	0.75
P-279	J-266	J-251	675	517.2	21,626.0	1.19	13,687.0	0.75
P-280	J-250	J-268	504	517.2	21,626.0	1.19	13,687.0	0.75
P-281	J-268	J-269	458	517.2	21,626.0	1.19	13,687.0	0.75
P-282	J-269	J-270	539	517.2	21,626.0	1.19	13,687.0	0.75
P-283	J-270	J-271	505	517.2	21,626.0	1.19	13,687.0	0.75
P-285	J-272	J-273	500	517.2	21,397.0	1.18	13,542.0	0.75
P-286	J-273	J-274	587	517.2	21,064.0	1.16	13,332.0	0.73
P-287	J-274	J-275	558	517.2	20,854.0	1.15	13,198.0	0.73
P-288	J-275	J-276	227	517.2	20,683.0	1.14	13,090.0	0.72
P-289	J-276	J-277	432	517.2	20,470.0	1.13	12,956.0	0.71
P-290	J-277	J-278	358	517.2	20,257.0	1.12	12,821.0	0.71
P-291	J-278	J-279	427	517.2	20,070.0	1.11	12,703.0	0.70

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-292	J-280	J-279	242	57.0	(106.0)	0.48	(67.0)	0.31
P-293	J-279	J-281	500	517.2	19,776.0	1.09	12,517.0	0.69
P-294	J-281	J-282	494	517.2	19,776.0	1.09	12,517.0	0.69
P-295	J-282	J-283	276	517.2	19,776.0	1.09	12,517.0	0.69
P-296	J-283	J-284	498	517.2	19,776.0	1.09	12,517.0	0.69
P-297	J-284	J-285	500	517.2	19,776.0	1.09	12,517.0	0.69
P-298	J-285	J-286	503	517.2	19,776.0	1.09	12,517.0	0.69
P-299	J-286	J-287	378	517.2	19,776.0	1.09	12,517.0	0.69
P-300	J-287	J-288	502	517.2	19,772.0	1.09	12,514.0	0.69
P-301	J-288	J-289	500	461.8	19,768.0	1.37	12,512.0	0.86
P-302	J-289	J-290	822	461.8	19,700.0	1.36	12,468.0	0.86
P-303	J-290	J-291	447	461.8	19,542.0	1.35	12,368.0	0.85
P-304	J-291	J-292	418	461.8	19,286.0	1.33	12,207.0	0.84
P-305	J-292	J-293	338	184.6	2,450.0	1.06	1,551.0	0.67
P-306	J-293	J-294	436	184.6	2,016.0	0.87	1,276.0	0.55
P-307	J-294	J-295	982	184.6	1,585.0	0.69	1,003.0	0.43
P-308	J-295	J-296	483	147.6	1,092.0	0.74	691.0	0.47
P-309	J-296	J-297	499	115.4	532.0	0.59	337.0	0.37
P-310	J-297	J-298	513	81.4	143.0	0.32	91.0	0.20
P-345	J-46	J-62	5	415.6	8.0	0.00	5.0	0.00
P-350	J-85	J-90	5	184.6	(63.0)	0.03	(40.0)	0.02
P-354	J-38	J-39	5	207.8	11,394.0	1.80	7,211.0	1.60
P-356	J-118	J-147	17	290.8	(805.0)	0.14	(509.0)	0.09
P-358	J-122	J-133	5	184.6	(46.0)	0.02	(29.0)	0.01
P-365	J-259	J-331	501	99.4	546.0	0.81	346.0	0.52
P-366	J-331	J-332	501	99.4	546.0	0.81	346.0	0.52
P-367	J-332	J-333	498	81.4	546.0	1.21	346.0	0.77
P-368	J-333	J-334	500	81.4	546.0	1.21	346.0	0.77
P-369	J-334	J-335	501	81.4	506.0	1.13	321.0	0.71
P-370	J-335	J-336	474	81.4	233.0	0.52	148.0	0.33
P-372	J-271	J-337	483	517.2	21,604.0	1.19	13,674.0	0.75
P-373	J-337	J-272	2	517.2	21,604.0	1.19	13,674.0	0.75
P-388	R-1	PMP-7	16	517.2	20,658.0	1.14	32,687.0	1.80
P-389	PMP-7	J-1	27	517.2	20,658.0	1.14	32,687.0	1.80
P-416	J-343	J-147	14	290.8	3,480.0	0.61	2,203.0	0.38
P-418	J-117	J-344	511	290.8	3,498.0	0.61	2,214.0	0.39
P-419	J-344	J-118	5	290.8	3,498.0	0.61	2,214.0	0.39
P-431	J-146	J-345	512	290.8	3,480.0	0.61	2,203.0	0.38
P-434	J-345	J-343	4	290.8	3,480.0	0.61	2,203.0	0.38
P-439	J-25	J-21	5	57.0	(57.0)	0.26	(36.0)	0.16
P-440	J-114	J-71	6	415.6	10,171.0	0.87	6,437.0	0.55
P-441	R-1	PMP-8	16	517.2	20,658.0	1.14	32,687.0	1.80
P-442	PMP-8	J-1	27	517.2	20,658.0	1.14	32,687.0	1.80
P-444	J-1	J-2	7	581.8	29,202.0	1.27	18,482.0	0.80
P-445	J-6	J-30	14	581.8	38,626.0	1.68	24,447.0	1.06
P-450	J-82	J-93	9	369.4	9,958.0	1.08	6,303.0	0.68
P-454	R-1	PMP-9	16	517.2	20,658.0	1.14	-	0.00
P-455	PMP-9	J-1	27	517.2	20,658.0	1.14	-	0.00
P-457	J-168	J-155	5	115.4	(136.0)	0.15	(86.0)	0.10
P-459	R-1	PMP-10	17	517.2	20,658.0	1.14	-	0.00
P-460	PMP-10	J-1	27	517.2	20,658.0	1.14	-	0.00
P-461	R-1	PMP-	16	517.2	20,658.0	1.14	-	0.00

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
		11						
P-462	PMP-11	J-1	28	517.2	20,658.0	1.14	-	0.00

IV.2. Option 2: Use Booster Pump with Water Tower

Figure 9: Network map

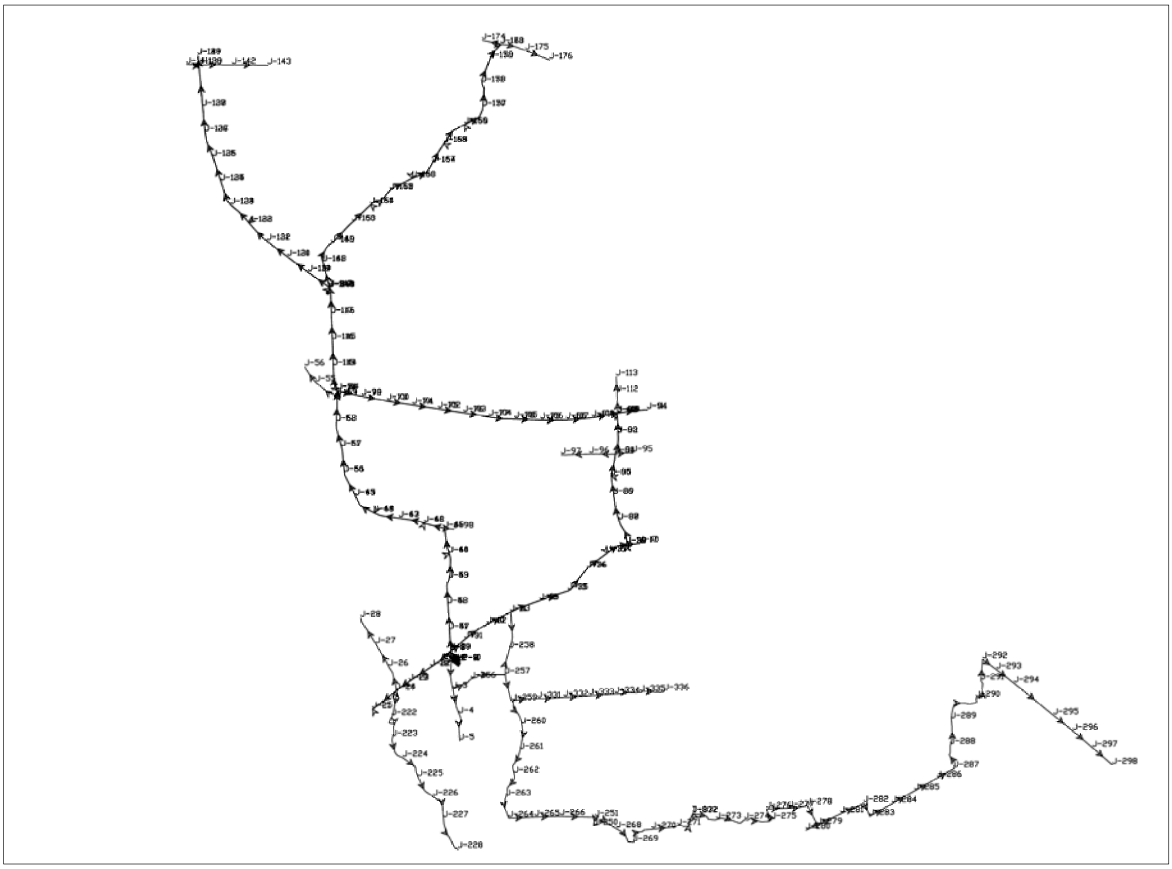


Table 13: Node data in 2025

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-1	13.99	6.81	33.00	33.00
J-2	14.01	20.43	33.00	33.00
J-3	14.84	34.05	32.00	32.00
J-4	14.71	13.62	32.00	32.00
J-5	14.69	6.81	32.00	32.00
J-6	15.01	52.78	32.00	32.00
J-7	15.01	27.24	32.00	32.00
J-8	15.11	87.96	32.00	32.00
J-9	14.70	73.21	31.00	32.00
J-10	14.80	76.28	31.00	31.00
J-11	14.89	84.73	30.00	31.00
J-12	14.65	36.10	29.00	31.00
J-13	15.36	48.14	28.00	30.00
J-14	12.63	71.00	30.00	32.00
J-15	12.45	83.04	29.00	32.00
J-16	13.75	83.04	28.00	31.00
J-17	12.47	4031.81	29.00	32.00
J-18	14.89	27.24	29.00	31.00
J-19	15.29	33.20	27.00	30.00
J-20	15.14	56.84	26.00	29.00
J-21	14.16	10.22	26.00	30.00
J-22	14.89	21.34	30.00	31.00
J-23	15.29	31.70	26.00	29.00
J-24	15.05	167.05	26.00	29.00
J-25	14.16	63.27	26.00	30.00
J-26	13.75	135.31	22.00	28.00
J-27	12.84	96.88	14.00	26.00
J-28	12.82	34.47	13.00	25.00
J-29	15.15	99.49	32.00	32.00
J-30	15.15	28.16	32.00	32.00
J-31	14.70	61.95	31.00	32.00
J-32	14.80	65.47	31.00	31.00
J-33	15.14	44.16	30.00	31.00
J-34	14.65	77.29	29.00	31.00
J-35	15.36	99.04	28.00	30.00
J-36	12.63	158.18	30.00	32.00
J-37	12.45	193.79	29.00	32.00
J-38	12.60	110.74	29.00	32.00
J-39	12.46	66.65	29.00	32.00
J-40	12.47	3990.94	29.00	32.00
J-41	14.39	69.15	32.00	32.00
J-42	14.06	54.32	31.00	32.00
J-43	14.10	44.22	31.00	32.00
J-44	14.01	35.22	30.00	31.00
J-45	13.25	44.77	30.00	32.00
J-46	13.49	56.03	30.00	31.00
J-47	13.26	53.91	29.00	31.00
J-48	13.22	64.05	29.00	31.00
J-49	13.54	65.60	28.00	30.00
J-50	13.58	74.97	27.00	30.00
J-51	13.25	65.21	27.00	30.00
J-52	13.25	118.34	27.00	29.00
J-53	12.88	161.07	26.00	29.00

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-54	12.04	44.95	27.00	30.00
J-55	11.64	104.62	18.00	27.00
J-56	11.30	35.96	18.00	27.00
J-57	14.39	62.81	32.00	32.00
J-58	14.06	53.48	31.00	32.00
J-59	14.10	36.56	31.00	32.00
J-60	14.01	65.06	30.00	31.00
J-61	13.26	82.83	30.00	32.00
J-62	13.49	40.33	30.00	31.00
J-63	13.26	4.49	29.00	31.00
J-64	13.22	1.87	29.00	31.00
J-65	13.54	20.62	28.00	30.00
J-66	13.58	44.98	27.00	30.00
J-67	13.25	56.23	27.00	30.00
J-68	13.25	101.57	27.00	29.00
J-69	12.88	82.48	26.00	29.00
J-70	13.06	76.35	26.00	29.00
J-71	12.85	58.93	26.00	29.00
J-72	12.78	128.16	26.00	29.00
J-73	12.28	89.98	26.00	29.00
J-74	12.46	27.27	25.00	29.00
J-75	12.45	16.36	25.00	29.00
J-76	11.26	38.18	26.00	30.00
J-77	12.20	65.44	24.00	29.00
J-78	11.28	125.84	24.00	29.00
J-79	11.70	209.92	23.00	29.00
J-80	11.24	150.76	23.00	29.00
J-81	11.33	155.80	22.00	29.00
J-82	11.58	169.63	21.00	28.00
J-83	11.19	66.36	23.00	29.00
J-84	11.62	90.08	23.00	29.00
J-85	11.44	95.09	25.00	30.00
J-86	11.61	141.14	27.00	31.00
J-87	11.69	116.69	28.00	32.00
J-88	11.69	206.53	28.00	32.00
J-89	11.61	168.05	26.00	31.00
J-90	11.44	51.97	25.00	30.00
J-91	11.62	131.58	23.00	29.00
J-92	11.19	55.66	23.00	29.00
J-93	11.48	203.91	22.00	28.00
J-94	10.49	4155.24	22.00	29.00
J-95	11.70	79.61	22.00	29.00
J-96	11.00	146.22	18.00	27.00
J-97	11.38	136.49	16.00	26.00
J-98	12.74	6.71	31.00	32.00
J-99	12.78	120.57	26.00	29.00
J-100	12.28	109.05	26.00	29.00
J-101	12.46	106.68	25.00	29.00
J-102	12.45	161.00	25.00	29.00
J-103	11.26	126.48	26.00	30.00
J-104	12.20	118.30	24.00	29.00
J-105	11.28	141.07	24.00	29.00
J-106	11.70	93.27	23.00	29.00
J-107	11.24	104.31	23.00	29.00
J-108	11.33	153.72	22.00	29.00

Node Label	Elevation (m)	Base Demand (m³/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-109	11.52	95.16	21.00	28.00
J-110	11.44	428.12	22.00	28.00
J-111	10.49	4263.88	22.00	29.00
J-112	10.82	721.43	21.00	28.00
J-113	10.87	572.31	20.00	28.00
J-114	12.83	36.73	26.00	29.00
J-115	11.97	75.11	27.00	29.00
J-116	12.91	74.06	26.00	28.00
J-117	13.40	93.81	25.00	27.00
J-118	13.33	90.54	25.00	26.00
J-119	13.07	54.05	25.00	26.00
J-120	12.80	31.02	25.00	27.00
J-121	12.48	46.53	24.00	27.00
J-122	12.44	55.69	24.00	26.00
J-123	12.14	74.26	23.00	26.00
J-124	11.81	92.82	23.00	26.00
J-125	11.77	95.63	22.00	26.00
J-126	11.77	72.52	21.00	26.00
J-127	11.63	42.06	21.00	26.00
J-128	11.40	134.46	18.00	25.00
J-129	11.85	100.82	18.00	24.00
J-130	13.06	25.62	25.00	26.00
J-131	12.80	25.62	25.00	27.00
J-132	12.48	13.64	24.00	27.00
J-133	12.44	30.63	24.00	26.00
J-134	12.14	81.87	23.00	26.00
J-135	11.81	112.02	23.00	26.00
J-136	11.77	76.76	22.00	26.00
J-137	11.77	45.65	21.00	26.00
J-138	11.63	95.33	21.00	25.00
J-139	11.35	226.98	18.00	25.00
J-140	11.85	98.01	18.00	24.00
J-141	10.96	5.61	19.00	25.00
J-142	10.30	84.11	18.00	25.00
J-143	10.23	28.04	17.00	25.00
J-144	12.99	82.66	26.00	28.00
J-145	12.91	78.12	26.00	28.00
J-146	13.18	91.42	25.00	27.00
J-147	13.30	85.96	25.00	26.00
J-148	12.63	51.97	25.00	27.00
J-149	12.44	41.85	25.00	27.00
J-150	11.64	15.92	25.00	27.00
J-151	12.57	24.73	23.00	26.00
J-152	11.73	38.05	23.00	27.00
J-153	11.47	50.81	23.00	27.00
J-154	11.44	37.76	23.00	27.00
J-155	11.43	13.44	22.00	26.00
J-156	11.71	29.87	21.00	26.00
J-157	11.71	37.66	20.00	25.00
J-158	12.03	26.15	19.00	25.00
J-159	11.14	18.68	20.00	26.00
J-160	11.34	38.92	18.00	25.00
J-161	12.64	25.62	25.00	27.00
J-162	12.38	41.53	25.00	27.00
J-163	11.62	37.54	25.00	27.00

Node Label	Elevation (m)	Base Demand (m³/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-164	13.01	34.24	23.00	26.00
J-165	11.75	47.56	23.00	27.00
J-166	11.54	64.54	23.00	27.00
J-167	11.44	53.92	23.00	26.00
J-168	11.40	37.34	22.00	26.00
J-169	11.72	41.82	21.00	26.00
J-170	11.73	42.01	20.00	25.00
J-171	12.05	42.96	19.00	25.00
J-172	11.14	28.03	19.00	25.00
J-173	11.30	20.24	18.00	25.00
J-174	11.07	15.57	18.00	25.00
J-175	11.09	65.39	18.00	25.00
J-176	11.14	34.25	17.00	24.00
J-222	14.11	32.35	22.00	28.00
J-223	14.29	20.43	18.00	27.00
J-224	14.57	27.24	15.00	25.00
J-225	15.25	37.86	12.00	23.00
J-226	15.73	52.69	8.00	22.00
J-227	15.78	53.09	7.00	21.00
J-228	15.78	21.24	6.00	21.00
J-250	15.28	0.00	27.00	27.00
J-251	15.55	0.00	27.00	27.00
J-256	14.08	23.84	32.00	32.00
J-257	14.01	70.41	32.00	32.00
J-258	13.87	54.54	31.00	32.00
J-259	14.59	60.95	31.00	31.00
J-260	15.08	51.80	30.00	30.00
J-261	15.27	44.40	29.00	29.00
J-262	15.21	84.54	29.00	29.00
J-263	15.77	124.76	28.00	28.00
J-264	15.69	61.05	28.00	28.00
J-265	15.51	0.00	28.00	28.00
J-266	15.70	0.00	27.00	27.00
J-268	14.13	0.00	28.00	28.00
J-269	15.24	0.00	27.00	26.00
J-270	15.32	0.00	26.00	26.00
J-271	15.27	9.56	26.00	25.00
J-272	15.54	93.66	25.00	25.00
J-273	15.28	150.05	25.00	25.00
J-274	14.88	94.62	25.00	25.00
J-275	14.29	72.91	25.00	26.00
J-276	14.60	88.48	25.00	25.00
J-277	14.93	88.48	24.00	25.00
J-278	13.56	77.42	25.00	26.00
J-279	11.43	77.42	27.00	28.00
J-280	12.42	44.24	26.00	27.00
J-281	12.42	0.00	26.00	27.00
J-282	11.42	0.00	26.00	28.00
J-283	11.99	0.00	26.00	27.00
J-284	11.38	0.00	26.00	28.00
J-285	11.37	0.00	26.00	27.00
J-286	11.20	0.00	26.00	27.00
J-287	11.15	1.53	25.00	27.00
J-288	12.47	1.53	24.00	26.00
J-289	12.44	30.49	23.00	26.00

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m^3/day)	Peak Demand	Average Demand
J-290	12.11	70.12	23.00	26.00
J-291	12.89	112.80	22.00	25.00
J-292	12.46	6271.29	22.00	25.00
J-293	13.08	207.34	21.00	25.00
J-294	12.63	206.32	21.00	25.00
J-295	12.65	228.44	20.00	24.00
J-296	12.92	252.98	19.00	24.00
J-297	11.29	176.85	20.00	25.00
J-298	11.04	65.36	20.00	25.00
J-331	14.60	0.00	29.00	30.00
J-332	13.75	0.00	28.00	30.00
J-333	13.57	0.00	22.00	28.00
J-334	13.63	18.68	17.00	26.00
J-335	14.14	130.78	12.00	23.00
J-336	8.55	112.10	17.00	28.00
J-337	15.53	0.00	25.00	25.00
J-343	13.30	0.00	25.00	26.00
J-344	13.33	0.00	25.00	26.00
J-345	13.30	0.00	25.00	26.00

Table 14: Pipe data in 2025

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-2	J-1	J-2	5	582	20,311.00	0.88	16,571.00	0.72
P-3	J-2	J-3	544	517	13,418.00	0.74	13,335.00	0.73
P-4	J-3	J-4	500	57	33.00	0.15	21.00	0.09
P-5	J-4	J-5	527	57	11.00	0.05	7.00	0.03
P-6	J-1	J-6	200	582	24,154.00	1.05	17,367.00	0.76
P-7	J-2	J-7	195	582	24,194.00	1.05	17,358.00	0.76
P-8	J-7	J-8	6	582	13,307.00	0.58	10,348.00	0.45
P-9	J-7	J-6	6	582	9,516.00	0.41	6,158.00	0.27
P-10	J-6	J-9	354	416	10,369.00	0.88	6,712.00	0.57
P-11	J-9	J-10	525	416	10,251.00	0.87	6,638.00	0.57
P-12	J-10	J-11	506	416	10,129.00	0.86	6,562.00	0.56
P-13	J-11	J-12	615	416	9,973.00	0.85	6,444.00	0.55
P-14	J-12	J-13	582	416	9,915.00	0.85	6,408.00	0.55
P-15	J-13	J-14	569	416	9,837.00	0.84	6,359.00	0.54
P-16	J-14	J-15	499	416	9,723.00	0.83	6,289.00	0.54
P-17	J-15	J-16	416	416	9,728.00	0.83	6,274.00	0.54
P-18	J-16	J-17	251	369	6,491.00	0.70	4,031.00	0.44
P-19	J-7	J-18	491	148	1,327.00	0.90	824.00	0.56
P-20	J-18	J-19	500	148	1,282.00	0.87	797.00	0.54
P-21	J-19	J-20	320	148	1,230.00	0.83	764.00	0.52
P-23	J-8	J-22	492	68	160.00	0.51	99.00	0.32
P-24	J-22	J-23	500	57	125.00	0.57	78.00	0.35
P-25	J-23	J-24	315	57	74.00	0.34	46.00	0.21
P-26	J-24	J-25	576	57	59.00	0.27	37.00	0.17
P-27	J-21	J-20	572	57	(59.00)	0.27	(37.00)	0.17
P-28	J-24	J-26	494	81	430.00	0.96	267.00	0.59
P-29	J-26	J-27	500	57	211.00	0.96	131.00	0.60
P-30	J-27	J-28	591	57	56.00	0.25	35.00	0.16
P-31	J-8	J-29	9	582	13,005.00	0.57	10,161.00	0.44
P-33	J-30	J-31	342	416	10,358.00	0.88	6,689.00	0.57
P-34	J-31	J-32	527	416	10,258.00	0.88	6,627.00	0.57
P-35	J-32	J-33	506	416	10,152.00	0.87	6,561.00	0.56
P-36	J-33	J-34	616	416	10,082.00	0.86	6,517.00	0.56
P-37	J-34	J-35	582	416	9,958.00	0.85	6,440.00	0.55
P-38	J-35	J-36	567	416	9,798.00	0.84	6,341.00	0.54
P-39	J-36	J-37	500	416	9,543.00	0.81	6,182.00	0.53
P-40	J-37	J-38	413	416	9,093.00	0.78	5,920.00	0.51
P-41	J-16	J-39	5	208	3,104.00	1.06	2,160.00	0.74
P-42	J-39	J-40	251	369	6,425.00	0.69	3,991.00	0.43
P-43	J-29	J-41	398	462	12,845.00	0.89	10,061.00	0.70
P-44	J-41	J-42	500	462	12,733.00	0.88	9,992.00	0.69
P-45	J-42	J-43	502	462	12,646.00	0.87	9,938.00	0.69
P-46	J-43	J-44	497	462	12,575.00	0.87	9,894.00	0.68
P-47	J-44	J-45	491	462	12,488.00	0.86	9,844.00	0.68
P-48	J-45	J-46	395	462	12,416.00	0.86	9,799.00	0.68
P-49	J-46	J-47	499	462	12,323.00	0.85	9,738.00	0.67
P-50	J-47	J-48	502	462	12,236.00	0.85	9,685.00	0.67
P-51	J-48	J-49	504	462	12,244.00	0.85	9,692.00	0.67
P-52	J-49	J-50	501	462	12,138.00	0.84	9,626.00	0.67
P-53	J-50	J-51	500	462	12,017.00	0.83	9,551.00	0.66

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-54	J-51	J-52	502	462	11,913.00	0.82	9,486.00	0.66
P-55	J-52	J-53	502	462	11,722.00	0.81	9,368.00	0.65
P-56	J-53	J-54	103	462	11,237.00	0.78	9,066.00	0.63
P-57	J-53	J-55	495	57	227.00	1.03	141.00	0.64
P-58	J-55	J-56	377	57	58.00	0.26	36.00	0.16
P-59	J-30	J-57	393	462	12,812.00	0.89	10,044.00	0.69
P-60	J-57	J-58	500	462	12,710.00	0.88	9,981.00	0.69
P-61	J-58	J-59	500	462	12,624.00	0.87	9,927.00	0.69
P-62	J-59	J-60	501	462	12,566.00	0.87	9,891.00	0.68
P-63	J-60	J-61	493	462	12,491.00	0.86	9,841.00	0.68
P-64	J-61	J-62	397	462	12,346.00	0.85	9,751.00	0.67
P-65	J-62	J-63	500	462	12,284.00	0.85	9,714.00	0.67
P-66	J-63	J-64	501	462	12,277.00	0.85	9,710.00	0.67
P-67	J-64	J-65	500	462	12,164.00	0.84	9,637.00	0.67
P-68	J-65	J-66	500	462	12,130.00	0.84	9,617.00	0.66
P-69	J-66	J-67	500	462	12,058.00	0.83	9,572.00	0.66
P-70	J-67	J-68	500	462	11,968.00	0.83	9,516.00	0.66
P-71	J-68	J-69	500	462	11,803.00	0.82	9,414.00	0.65
P-72	J-69	J-70	104	462	11,671.00	0.81	9,332.00	0.64
P-73	J-70	J-71	5	462	2,928.00	0.20	4,153.00	0.29
P-74	J-70	J-72	480	416	8,621.00	0.74	5,103.00	0.44
P-75	J-72	J-73	499	416	8,415.00	0.72	4,975.00	0.42
P-76	J-73	J-74	500	416	8,270.00	0.71	4,885.00	0.42
P-77	J-74	J-75	501	416	8,226.00	0.70	4,857.00	0.41
P-78	J-75	J-76	499	416	8,200.00	0.70	4,841.00	0.41
P-79	J-76	J-77	501	416	8,138.00	0.69	4,803.00	0.41
P-80	J-77	J-78	500	369	8,033.00	0.87	4,737.00	0.51
P-81	J-78	J-79	500	369	7,829.00	0.85	4,611.00	0.50
P-82	J-79	J-80	501	369	7,491.00	0.81	4,401.00	0.48
P-83	J-80	J-81	500	369	7,249.00	0.78	4,251.00	0.46
P-84	J-81	J-82	493	369	6,999.00	0.76	4,095.00	0.44
P-85	J-82	J-83	400	185	(1,490.00)	0.64	(1,179.00)	0.51
P-86	J-83	J-84	397	185	(1,598.00)	0.69	(1,246.00)	0.54
P-87	J-84	J-85	429	185	(2,197.00)	0.95	(1,618.00)	0.70
P-88	J-85	J-86	378	185	(2,282.00)	0.99	(1,669.00)	0.72
P-89	J-86	J-87	500	208	(2,509.00)	0.86	(1,810.00)	0.62
P-90	J-87	J-38	503	208	(2,697.00)	0.92	(1,926.00)	0.66
P-91	J-39	J-88	504	208	2,789.00	0.95	1,986.00	0.68
P-92	J-88	J-89	500	208	2,457.00	0.84	1,780.00	0.61
P-93	J-89	J-90	374	185	2,187.00	0.95	1,612.00	0.70
P-94	J-90	J-91	431	185	2,036.00	0.88	1,516.00	0.66
P-95	J-91	J-92	397	185	1,696.00	0.73	1,305.00	0.56
P-96	J-92	J-93	402	185	1,607.00	0.70	1,249.00	0.54
P-97	J-93	J-94	579	369	6,690.00	0.72	4,155.00	0.45
P-98	J-91	J-95	338	68	128.00	0.41	79.00	0.25
P-99	J-84	J-96	498	81	455.00	1.01	283.00	0.63
P-100	J-96	J-97	548	81	220.00	0.49	137.00	0.30
P-102	J-61	J-98	190	57	11.00	0.05	7.00	0.03
P-103	J-71	J-99	480	416	8,970.00	0.77	5,319.00	0.45
P-104	J-99	J-100	500	416	8,775.00	0.75	5,198.00	0.44
P-105	J-100	J-101	500	416	8,600.00	0.73	5,089.00	0.43
P-106	J-101	J-102	500	416	8,429.00	0.72	4,983.00	0.43

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-107	J-102	J-103	499	416	8,170.00	0.70	4,822.00	0.41
P-108	J-103	J-104	503	416	7,967.00	0.68	4,696.00	0.40
P-109	J-104	J-105	498	369	7,777.00	0.84	4,578.00	0.49
P-110	J-105	J-106	499	369	7,550.00	0.82	4,437.00	0.48
P-111	J-106	J-107	500	369	7,400.00	0.80	4,344.00	0.47
P-112	J-107	J-108	500	369	7,231.00	0.78	4,239.00	0.46
P-113	J-108	J-109	500	369	6,984.00	0.75	4,085.00	0.44
P-114	J-109	J-82	8	369	(2,806.00)	0.30	(1,996.00)	0.22
P-115	J-109	J-110	5	369	9,637.00	1.04	5,986.00	0.65
P-116	J-110	J-111	577	369	6,865.00	0.74	4,264.00	0.46
P-118	J-110	J-112	389	185	2,082.00	0.90	1,293.00	0.56
P-119	J-112	J-113	316	185	921.00	0.40	572.00	0.25
P-120	J-54	J-114	8	462	8,687.00	0.60	5,151.00	0.36
P-122	J-54	J-115	503	291	2,477.00	0.43	3,870.00	0.67
P-123	J-115	J-116	501	291	2,356.00	0.41	3,795.00	0.66
P-124	J-116	J-117	500	291	2,236.00	0.39	3,721.00	0.65
P-126	J-118	J-119	507	231	1,442.00	0.40	895.00	0.25
P-127	J-119	J-120	501	231	1,354.00	0.37	841.00	0.23
P-128	J-120	J-121	497	185	1,304.00	0.56	810.00	0.35
P-129	J-121	J-122	503	185	1,229.00	0.53	763.00	0.33
P-130	J-122	J-123	497	185	1,164.00	0.50	723.00	0.31
P-131	J-123	J-124	499	185	1,044.00	0.45	649.00	0.28
P-132	J-124	J-125	495	148	895.00	0.61	556.00	0.38
P-133	J-125	J-126	504	148	741.00	0.50	460.00	0.31
P-134	J-126	J-127	498	148	624.00	0.42	388.00	0.26
P-135	J-127	J-128	812	115	556.00	0.62	345.00	0.38
P-136	J-128	J-129	183	81	163.00	0.36	101.00	0.22
P-137	J-118	J-130	511	231	1,377.00	0.38	855.00	0.24
P-138	J-130	J-131	501	231	1,335.00	0.37	829.00	0.23
P-139	J-131	J-132	498	185	1,293.00	0.56	803.00	0.35
P-140	J-132	J-133	501	185	1,271.00	0.55	789.00	0.34
P-141	J-133	J-134	496	185	1,198.00	0.52	744.00	0.32
P-142	J-134	J-135	498	185	1,066.00	0.46	662.00	0.29
P-143	J-135	J-136	495	148	885.00	0.60	550.00	0.37
P-144	J-136	J-137	502	148	761.00	0.51	473.00	0.32
P-145	J-137	J-138	499	148	688.00	0.47	427.00	0.29
P-146	J-138	J-139	812	115	535.00	0.59	332.00	0.37
P-147	J-139	J-140	183	81	157.00	0.35	98.00	0.22
P-148	J-139	J-128	5	115	(168.00)	0.19	(104.00)	0.12
P-149	J-128	J-141	230	57	8.00	0.04	5.00	0.02
P-150	J-139	J-142	651	81	179.00	0.40	111.00	0.25
P-151	J-142	J-143	689	57	45.00	0.20	28.00	0.13
P-152	J-114	J-144	494	291	2,490.00	0.43	3,889.00	0.68
P-153	J-144	J-145	500	291	2,357.00	0.41	3,806.00	0.66
P-154	J-145	J-146	500	291	2,232.00	0.39	3,728.00	0.65
P-156	J-147	J-148	496	185	866.00	0.37	538.00	0.23
P-157	J-148	J-149	442	166	783.00	0.42	486.00	0.26
P-158	J-149	J-150	581	148	716.00	0.48	445.00	0.30
P-159	J-150	J-151	490	148	691.00	0.47	429.00	0.29
P-160	J-151	J-152	481	148	630.00	0.43	391.00	0.26
P-161	J-152	J-153	499	148	569.00	0.38	353.00	0.24
P-162	J-153	J-154	502	148	602.00	0.41	374.00	0.25

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-163	J-154	J-155	447	148	541.00	0.37	336.00	0.23
P-164	J-155	J-156	555	115	426.00	0.47	264.00	0.29
P-165	J-156	J-157	531	115	292.00	0.32	181.00	0.20
P-166	J-157	J-158	470	99	231.00	0.34	143.00	0.21
P-167	J-158	J-159	499	99	189.00	0.28	117.00	0.18
P-168	J-159	J-160	353	68	159.00	0.51	98.00	0.32
P-169	J-147	J-161	498	185	839.00	0.36	521.00	0.23
P-170	J-161	J-162	447	166	798.00	0.43	495.00	0.26
P-171	J-162	J-163	581	148	731.00	0.49	454.00	0.31
P-172	J-163	J-164	490	148	671.00	0.45	417.00	0.28
P-173	J-164	J-165	483	148	636.00	0.43	395.00	0.27
P-174	J-165	J-166	499	148	560.00	0.38	348.00	0.24
P-175	J-166	J-167	499	115	339.00	0.38	211.00	0.23
P-176	J-167	J-168	447	115	253.00	0.28	157.00	0.17
P-177	J-168	J-169	558	99	287.00	0.43	178.00	0.27
P-178	J-169	J-170	526	99	305.00	0.45	189.00	0.28
P-179	J-170	J-171	470	99	237.00	0.35	147.00	0.22
P-180	J-171	J-172	501	99	167.00	0.25	104.00	0.15
P-181	J-172	J-173	351	68	122.00	0.39	76.00	0.24
P-182	J-173	J-174	365	57	25.00	0.11	16.00	0.07
P-183	J-173	J-160	7	81	65.00	0.15	41.00	0.09
P-184	J-160	J-175	493	81	161.00	0.36	100.00	0.22
P-185	J-175	J-176	500	57	56.00	0.25	35.00	0.16
P-231	J-24	J-20	7	148	(683.00)	0.46	(424.00)	0.29
P-232	J-20	J-222	501	81	395.00	0.88	245.00	0.55
P-233	J-222	J-223	501	81	344.00	0.76	213.00	0.47
P-234	J-223	J-224	500	81	310.00	0.69	193.00	0.43
P-235	J-224	J-225	500	81	266.00	0.59	165.00	0.37
P-236	J-225	J-226	500	68	204.00	0.66	127.00	0.41
P-237	J-226	J-227	500	68	120.00	0.38	74.00	0.24
P-238	J-227	J-228	701	57	35.00	0.16	22.00	0.10
P-261	J-250	J-251	174	517	(12,001.00)	0.66	(12,475.00)	0.69
P-266	J-3	J-256	411	517	13,331.00	0.73	13,280.00	0.73
P-267	J-256	J-257	681	517	13,292.00	0.73	13,256.00	0.73
P-268	J-11	J-258	713	57	20.00	0.09	33.00	0.15
P-269	J-258	J-257	509	57	(68.00)	0.31	(21.00)	0.10
P-270	J-257	J-259	539	517	13,111.00	0.72	13,165.00	0.73
P-271	J-259	J-260	501	517	12,591.00	0.69	12,842.00	0.71
P-272	J-260	J-261	500	517	12,507.00	0.69	12,790.00	0.70
P-273	J-261	J-262	501	517	12,436.00	0.69	12,746.00	0.70
P-274	J-262	J-263	501	517	12,300.00	0.68	12,661.00	0.70
P-275	J-263	J-264	427	517	12,100.00	0.67	12,537.00	0.69
P-276	J-264	J-265	505	517	12,001.00	0.66	12,475.00	0.69
P-277	J-265	J-266	496	517	12,001.00	0.66	12,475.00	0.69
P-279	J-266	J-251	675	517	12,001.00	0.66	12,475.00	0.69
P-280	J-250	J-268	504	517	12,001.00	0.66	12,475.00	0.69
P-281	J-268	J-269	458	517	12,001.00	0.66	12,475.00	0.69
P-282	J-269	J-270	539	517	12,001.00	0.66	12,475.00	0.69
P-283	J-270	J-271	505	517	12,001.00	0.66	12,475.00	0.69
P-285	J-272	J-273	500	517	13,394.00	0.74	8,319.00	0.46
P-286	J-273	J-274	587	517	13,152.00	0.72	8,169.00	0.45

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-287	J-274	J-275	558	517	12,999.00	0.72	8,074.00	0.44
P-288	J-275	J-276	227	517	12,882.00	0.71	8,002.00	0.44
P-289	J-276	J-277	432	517	12,741.00	0.70	7,913.00	0.44
P-290	J-277	J-278	358	517	12,599.00	0.69	7,825.00	0.43
P-291	J-278	J-279	427	517	12,473.00	0.69	7,747.00	0.43
P-292	J-280	J-279	242	57	(71.00)	0.32	(44.00)	0.20
P-293	J-279	J-281	500	517	12,277.00	0.68	7,626.00	0.42
P-294	J-281	J-282	494	517	12,277.00	0.68	7,626.00	0.42
P-295	J-282	J-283	276	517	12,277.00	0.68	7,626.00	0.42
P-296	J-283	J-284	498	517	12,277.00	0.68	7,626.00	0.42
P-297	J-284	J-285	500	517	12,277.00	0.68	7,626.00	0.42
P-298	J-285	J-286	503	517	12,277.00	0.68	7,626.00	0.42
P-299	J-286	J-287	378	517	12,277.00	0.68	7,626.00	0.42
P-300	J-287	J-288	502	517	12,275.00	0.68	7,624.00	0.42
P-301	J-288	J-289	500	517	12,272.00	0.68	7,622.00	0.42
P-302	J-289	J-290	822	517	12,223.00	0.67	7,592.00	0.42
P-303	J-290	J-291	447	517	12,110.00	0.67	7,522.00	0.41
P-304	J-291	J-292	418	517	11,928.00	0.66	7,409.00	0.41
P-305	J-292	J-293	338	231	1,832.00	0.51	1,138.00	0.31
P-306	J-293	J-294	436	231	1,498.00	0.41	931.00	0.26
P-307	J-294	J-295	982	185	1,166.00	0.50	724.00	0.31
P-308	J-295	J-296	483	148	798.00	0.54	496.00	0.34
P-309	J-296	J-297	499	115	391.00	0.43	243.00	0.27
P-310	J-297	J-298	513	81	106.00	0.24	66.00	0.15
P-345	J-46	J-62	5	416	3.00	0.00	4.00	0.00
P-350	J-85	J-90	5	185	(68.00)	0.03	(45.00)	0.02
P-354	J-38	J-39	5	208	6,218.00	2.12	3,884.00	1.33
P-356	J-118	J-147	17	291	(2,964.00)	0.52	(1,841.00)	0.32
P-358	J-122	J-133	5	185	(24.00)	0.01	(15.00)	0.01
P-365	J-259	J-331	501	99	421.00	0.63	262.00	0.39
P-366	J-331	J-332	501	99	421.00	0.63	262.00	0.39
P-367	J-332	J-333	498	81	421.00	0.94	262.00	0.58
P-368	J-333	J-334	500	81	421.00	0.94	262.00	0.58
P-369	J-334	J-335	501	81	391.00	0.87	243.00	0.54
P-370	J-335	J-336	474	81	181.00	0.40	112.00	0.25
P-372	J-271	J-337	483	517	11,986.00	0.66	12,466.00	0.69
P-388	R-1	PMP-7	16	517	12,362.00	0.68	12,022.00	0.66
P-389	PMP-7	J-1	27	517	12,362.00	0.68	12,022.00	0.66
P-418	J-117	J-344	511	291	2,085.00	0.36	3,627.00	0.63
P-431	J-146	J-345	512	291	2,084.00	0.36	3,637.00	0.63
P-434	J-345	J-343	4	291	2,084.00	0.36	3,637.00	0.63
P-439	J-25	J-21	5	57	(43.00)	0.19	(26.00)	0.12
P-440	J-114	J-71	6	416	6,137.00	0.52	1,225.00	0.10
P-441	R-1	PMP-8	16	517	12,362.00	0.68	12,022.00	0.66
P-442	PMP-8	J-1	27	517	12,362.00	0.68	12,022.00	0.66
P-444	J-1	J-2	7	582	17,335.00	0.75	14,143.00	0.62
P-445	J-6	J-30	14	582	23,216.00	1.01	16,761.00	0.73
P-450	J-82	J-93	9	369	5,410.00	0.58	3,109.00	0.34
P-454	R-1	PMP-9	16	517	12,362.00	0.68	12,022.00	0.66
P-455	PMP-9	J-1	27	517	12,362.00	0.68	12,022.00	0.66
P-457	J-168	J-155	5	115.4	-93	0.1	-58	0.06
P-459	R-1	PMP-10	17	517.2	12362	0.68	12022	0.66

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-460	PMP-10	J-1	27	517.2	12362	0.68	12022	0.66
P-461	R-1	PMP-11	16	517.2	12362	0.68	0	0
P-462	PMP-11	J-1	28	517.2	12362	0.68	0	0
P-463	J-37	J-15	5	327.8	139	0.02	69	0.01
P-465	J-44	J-60	5	415.6	30	0	15	0
P-474	J-343	T-6	11	290.8	4169	0.73	7264	1.27
P-475	T-6	J-147	13	290.8	4807	0.84	2986	0.52
P-476	J-337	T-8	8	517.2	11986	0.66	12466	0.69
P-477	T-8	J-272	6	517.2	13545	0.75	8413	0.46
P-478	J-344	J-343	8	290.8	2085	0.36	3627	0.63

Table 15: Node data in 2040

Node Label	Elevation (m)	Base Demand (m³/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-1	13.99	8.99	50.0	50.0
J-2	14.01	26.97	50.0	50.0
J-3	14.84	44.96	48.0	48.0
J-4	14.71	17.98	47.0	48.0
J-5	14.69	8.99	47.0	48.0
J-6	15.01	69.68	48.0	49.0
J-7	15.01	35.96	48.0	49.0
J-8	15.11	118.90	48.0	48.0
J-9	14.70	96.65	47.0	48.0
J-10	14.80	100.70	45.0	47.0
J-11	14.89	111.86	43.0	46.0
J-12	14.65	47.66	41.0	45.0
J-13	15.36	63.55	38.0	44.0
J-14	12.63	93.74	39.0	45.0
J-15	12.45	109.62	37.0	45.0
J-16	13.75	109.62	34.0	43.0
J-17	12.47	7664.51	35.0	44.0
J-18	14.89	35.96	44.0	47.0
J-19	15.29	43.83	40.0	45.0
J-20	15.14	75.09	38.0	44.0
J-21	14.16	13.49	38.0	44.0
J-22	14.89	28.85	45.0	47.0
J-23	15.29	42.86	39.0	45.0
J-24	15.05	225.82	38.0	44.0
J-25	14.16	85.52	38.0	44.0
J-26	13.75	182.92	30.0	42.0
J-27	12.84	130.96	17.0	37.0
J-28	12.82	46.60	16.0	36.0
J-29	15.15	134.49	48.0	48.0
J-30	15.15	38.07	48.0	48.0
J-31	14.70	83.74	47.0	48.0
J-32	14.80	88.50	45.0	47.0
J-33	15.14	59.70	43.0	46.0
J-34	14.65	104.48	41.0	45.0
J-35	15.36	136.46	38.0	44.0
J-36	12.63	228.61	39.0	45.0
J-37	12.45	284.96	37.0	45.0
J-38	12.60	162.83	36.0	44.0
J-39	12.46	98.01	36.0	44.0
J-40	12.47	7611.45	35.0	44.0
J-41	14.39	93.48	48.0	48.0
J-42	14.06	73.42	46.0	47.0
J-43	14.10	59.77	44.0	46.0
J-44	14.01	47.61	43.0	45.0
J-45	13.25	60.52	42.0	45.0
J-46	13.49	75.74	41.0	43.0
J-47	13.26	72.87	39.0	42.0
J-48	13.22	90.97	38.0	41.0
J-49	13.54	98.90	36.0	40.0
J-50	13.58	113.03	34.0	39.0
J-51	13.25	96.92	33.0	38.0
J-52	13.25	177.02	31.0	37.0
J-53	12.88	242.84	30.0	36.0
J-54	12.04	67.77	31.0	37.0

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m^3/day)	Peak Demand	Average Demand
J-55	11.64	157.73	12.0	29.0
J-56	11.30	54.22	11.0	29.0
J-57	14.39	84.91	48.0	48.0
J-58	14.06	72.29	46.0	47.0
J-59	14.10	49.42	44.0	46.0
J-60	14.01	87.94	43.0	45.0
J-61	13.26	111.97	42.0	45.0
J-62	13.49	54.52	41.0	43.0
J-63	13.26	6.07	39.0	42.0
J-64	13.22	2.83	38.0	41.0
J-65	13.54	31.08	36.0	40.0
J-66	13.58	67.82	34.0	39.0
J-67	13.25	84.78	33.0	38.0
J-68	13.25	153.14	31.0	37.0
J-69	12.88	124.36	30.0	36.0
J-70	13.06	115.11	30.0	36.0
J-71	12.85	88.86	30.0	36.0
J-72	12.78	193.22	29.0	35.0
J-73	12.28	135.66	28.0	35.0
J-74	12.46	41.11	27.0	35.0
J-75	12.45	20.57	25.0	34.0
J-76	11.26	47.99	26.0	35.0
J-77	12.20	82.27	23.0	34.0
J-78	11.28	158.18	22.0	34.0
J-79	11.70	263.87	20.0	33.0
J-80	11.24	189.50	19.0	32.0
J-81	11.33	195.84	17.0	32.0
J-82	11.58	213.22	15.0	31.0
J-83	11.19	88.14	18.0	33.0
J-84	11.62	130.89	20.0	34.0
J-85	11.44	139.82	25.0	38.0
J-86	11.61	207.55	29.0	40.0
J-87	11.69	171.59	32.0	42.0
J-88	11.69	303.69	32.0	42.0
J-89	11.61	247.11	28.0	40.0
J-90	11.44	76.42	25.0	38.0
J-91	11.62	193.48	20.0	35.0
J-92	11.19	77.52	18.0	33.0
J-93	11.48	259.15	15.0	31.0
J-94	10.49	7816.69	14.0	31.0
J-95	11.70	117.06	18.0	34.0
J-96	11.00	185.88	12.0	31.0
J-97	11.38	171.57	9.0	30.0
J-98	12.74	9.07	43.0	45.0
J-99	12.78	181.79	29.0	35.0
J-100	12.28	164.42	28.0	35.0
J-101	12.46	160.84	26.0	35.0
J-102	12.45	232.69	25.0	34.0
J-103	11.26	174.44	25.0	35.0
J-104	12.20	148.71	23.0	34.0
J-105	11.28	177.33	22.0	34.0
J-106	11.70	117.24	20.0	33.0
J-107	11.24	131.12	19.0	32.0
J-108	11.33	193.22	17.0	32.0
J-109	11.52	119.62	15.0	31.0

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-110	11.44	538.15	15.0	31.0
J-111	10.49	7953.25	14.0	31.0
J-112	10.82	1056.85	13.0	30.0
J-113	10.87	869.41	12.0	30.0
J-114	12.83	55.37	30.0	36.0
J-115	11.97	113.24	30.0	34.0
J-116	12.91	111.66	28.0	30.0
J-117	13.40	141.43	26.0	27.0
J-118	13.33	136.31	25.0	24.0
J-119	13.07	81.50	25.0	24.0
J-120	12.80	46.77	24.0	24.0
J-121	12.48	69.01	23.0	24.0
J-122	12.44	82.26	22.0	24.0
J-123	12.14	109.68	21.0	23.0
J-124	11.81	137.10	20.0	23.0
J-125	11.77	141.24	18.0	22.0
J-126	11.77	107.10	16.0	22.0
J-127	11.63	62.11	15.0	21.0
J-128	11.40	198.60	10.0	19.0
J-129	11.85	148.90	9.0	18.0
J-130	13.06	37.84	25.0	24.0
J-131	12.80	37.84	24.0	25.0
J-132	12.48	20.14	23.0	24.0
J-133	12.44	45.23	22.0	24.0
J-134	12.14	120.92	21.0	23.0
J-135	11.81	165.45	20.0	23.0
J-136	11.77	113.38	18.0	22.0
J-137	11.77	67.43	16.0	21.0
J-138	11.63	140.79	15.0	21.0
J-139	11.35	335.25	10.0	19.0
J-140	11.85	144.76	9.0	18.0
J-141	10.96	8.28	10.0	20.0
J-142	10.30	124.23	8.0	19.0
J-143	10.23	41.41	7.0	19.0
J-144	12.99	124.62	29.0	33.0
J-145	12.91	117.79	28.0	30.0
J-146	13.18	137.83	26.0	27.0
J-147	13.30	129.01	25.0	25.0
J-148	12.63	76.98	25.0	25.0
J-149	12.44	61.81	24.0	25.0
J-150	11.64	23.51	23.0	25.0
J-151	12.57	36.53	21.0	23.0
J-152	11.73	56.19	21.0	24.0
J-153	11.47	75.05	20.0	23.0
J-154	11.44	55.77	19.0	23.0
J-155	11.43	19.85	18.0	23.0
J-156	11.71	44.12	15.0	21.0
J-157	11.71	55.62	14.0	21.0
J-158	12.03	38.62	13.0	20.0
J-159	11.14	27.59	13.0	21.0
J-160	11.34	57.49	10.0	19.0
J-161	12.64	37.84	25.0	25.0
J-162	12.38	61.34	24.0	25.0
J-163	11.62	55.44	23.0	25.0
J-164	13.01	50.57	20.0	23.0

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m³/day)	Peak Demand	Average Demand
J-165	11.75	70.24	21.0	23.0
J-166	11.54	95.32	20.0	23.0
J-167	11.44	79.65	19.0	23.0
J-168	11.40	55.15	18.0	23.0
J-169	11.72	61.77	15.0	21.0
J-170	11.73	62.05	13.0	20.0
J-171	12.05	63.45	12.0	19.0
J-172	11.14	41.39	12.0	20.0
J-173	11.30	29.89	10.0	19.0
J-174	11.07	23.00	10.0	19.0
J-175	11.09	96.58	8.0	19.0
J-176	11.14	50.59	7.0	18.0
J-222	14.11	42.71	32.0	42.0
J-223	14.29	26.97	26.0	39.0
J-224	14.57	35.96	21.0	37.0
J-225	15.25	49.98	16.0	35.0
J-226	15.73	69.56	10.0	32.0
J-227	15.78	70.09	8.0	31.0
J-228	15.78	28.03	8.0	31.0
J-250	15.28	0.00	32.0	32.0
J-251	15.55	0.00	32.0	32.0
J-256	14.08	31.47	47.0	47.0
J-257	14.01	92.95	45.0	46.0
J-258	13.87	72.00	44.0	46.0
J-259	14.59	80.47	43.0	44.0
J-260	15.08	68.39	41.0	42.0
J-261	15.27	58.61	40.0	40.0
J-262	15.21	111.61	39.0	39.0
J-263	15.77	164.70	37.0	37.0
J-264	15.69	80.60	36.0	36.0
J-265	15.51	0.00	35.0	35.0
J-266	15.70	0.00	33.0	34.0
J-268	14.13	0.00	31.0	32.0
J-269	15.24	0.00	29.0	29.0
J-270	15.32	0.00	28.0	28.0
J-271	15.27	13.43	27.0	27.0
J-272	15.54	131.58	25.0	25.0
J-273	15.28	210.79	24.0	25.0
J-274	14.88	132.92	24.0	25.0
J-275	14.29	107.76	23.0	25.0
J-276	14.60	134.97	23.0	25.0
J-277	14.93	134.97	21.0	24.0
J-278	13.56	118.10	22.0	25.0
J-279	11.43	118.10	24.0	27.0
J-280	12.42	67.49	22.0	26.0
J-281	12.42	0.00	22.0	26.0
J-282	11.42	0.00	22.0	26.0
J-283	11.99	0.00	21.0	26.0
J-284	11.38	0.00	21.0	26.0
J-285	11.37	0.00	20.0	26.0
J-286	11.20	0.00	19.0	25.0
J-287	11.15	2.31	19.0	25.0
J-288	12.47	2.31	17.0	23.0
J-289	12.44	43.59	16.0	23.0
J-290	12.11	100.27	15.0	23.0

Node Label	Elevation (m)	Base Demand (m³/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-291	12.89	161.30	13.0	22.0
J-292	12.46	10761.81	13.0	22.0
J-293	13.08	274.59	12.0	21.0
J-294	12.63	273.13	12.0	21.0
J-295	12.65	311.92	10.0	20.0
J-296	12.92	354.11	8.0	19.0
J-297	11.29	246.37	8.0	20.0
J-298	11.04	90.65	7.0	20.0
J-331	14.60	0.00	40.0	42.0
J-332	13.75	0.00	38.0	42.0
J-333	13.57	0.00	29.0	38.0
J-334	13.63	24.66	21.0	34.0
J-335	14.14	172.65	13.0	31.0
J-336	8.55	147.99	17.0	36.0
J-337	15.53	0.00	25.0	25.0
J-343	13.30	0.00	25.0	25.0
J-344	13.33	0.00	25.0	25.0
J-345	13.30	0.00	25.0	25.0

Table 16: Pipe data in 2040

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-2	J-1	J-2	5	581.8	32,192.0	1.40	30,973.0	1.35
P-3	J-2	J-3	544	517.2	27,957.0	1.54	26,286.0	1.45
P-4	J-3	J-4	500	57.0	27.0	0.12	27.0	0.12
P-5	J-4	J-5	527	57.0	9.0	0.04	9.0	0.04
P-6	J-1	J-6	200	581.8	31,750.0	1.38	31,145.0	1.36
P-7	J-2	J-7	195	581.8	31,682.0	1.38	31,093.0	1.35
P-8	J-7	J-8	6	581.8	19,215.0	0.84	18,692.0	0.81
P-9	J-7	J-6	6	581.8	11,328.0	0.49	11,261.0	0.49
P-10	J-6	J-9	354	415.6	12,107.0	1.03	12,030.0	1.03
P-11	J-9	J-10	525	415.6	12,011.0	1.02	11,933.0	1.02
P-12	J-10	J-11	506	415.6	11,909.0	1.02	11,832.0	1.01
P-13	J-11	J-12	615	415.6	11,718.0	1.00	11,652.0	0.99
P-14	J-12	J-13	582	415.6	11,670.0	1.00	11,604.0	0.99
P-15	J-13	J-14	569	415.6	11,606.0	0.99	11,540.0	0.98
P-16	J-14	J-15	499	415.6	11,513.0	0.98	11,447.0	0.98
P-17	J-15	J-16	416	415.6	11,599.0	0.99	11,537.0	0.98
P-18	J-16	J-17	251	369.4	7,665.0	0.83	7,665.0	0.83
P-19	J-7	J-18	491	147.6	1,103.0	0.75	1,103.0	0.75
P-20	J-18	J-19	500	147.6	1,067.0	0.72	1,067.0	0.72
P-21	J-19	J-20	320	147.6	1,023.0	0.69	1,023.0	0.69
P-23	J-8	J-22	492	67.8	133.0	0.43	133.0	0.43
P-24	J-22	J-23	500	57.0	104.0	0.47	104.0	0.47
P-25	J-23	J-24	315	57.0	61.0	0.28	61.0	0.28
P-26	J-24	J-25	576	57.0	50.0	0.22	50.0	0.22
P-27	J-21	J-20	572	57.0	(50.0)	0.23	(50.0)	0.23
P-28	J-24	J-26	494	81.4	361.0	0.80	361.0	0.80
P-29	J-26	J-27	500	57.0	178.0	0.81	178.0	0.81
P-30	J-27	J-28	591	57.0	47.0	0.21	47.0	0.21
P-31	J-8	J-29	9	581.8	18,963.0	0.83	18,440.0	0.80
P-33	J-30	J-31	342	415.6	12,061.0	1.03	11,992.0	1.02
P-34	J-31	J-32	527	415.6	11,978.0	1.02	11,908.0	1.02
P-35	J-32	J-33	506	415.6	11,889.0	1.01	11,820.0	1.01
P-36	J-33	J-34	616	415.6	11,830.0	1.01	11,760.0	1.00
P-37	J-34	J-35	582	415.6	11,725.0	1.00	11,655.0	0.99
P-38	J-35	J-36	567	415.6	11,589.0	0.99	11,519.0	0.98
P-39	J-36	J-37	500	415.6	11,360.0	0.97	11,290.0	0.96
P-40	J-37	J-38	413	415.6	10,879.0	0.93	10,805.0	0.92
P-41	J-16	J-39	5	207.8	3,825.0	1.31	3,762.0	1.28
P-42	J-39	J-40	251	369.4	7,612.0	0.82	7,612.0	0.82
P-43	J-29	J-41	398	461.8	18,828.0	1.30	18,305.0	1.26
P-44	J-41	J-42	500	461.8	18,735.0	1.29	18,212.0	1.26
P-45	J-42	J-43	502	461.8	18,661.0	1.29	18,139.0	1.25
P-46	J-43	J-44	497	461.8	18,602.0	1.29	18,079.0	1.25
P-47	J-44	J-45	491	461.8	18,538.0	1.28	18,014.0	1.24
P-48	J-45	J-46	395	461.8	18,477.0	1.28	17,954.0	1.24
P-49	J-46	J-47	499	461.8	18,390.0	1.27	17,867.0	1.23
P-50	J-47	J-48	502	461.8	18,317.0	1.27	17,794.0	1.23
P-51	J-48	J-49	504	461.8	18,327.0	1.27	17,804.0	1.23
P-52	J-49	J-50	501	461.8	18,229.0	1.26	17,706.0	1.22
P-53	J-50	J-51	500	461.8	18,116.0	1.25	17,593.0	1.22
P-54	J-51	J-52	502	461.8	18,019.0	1.25	17,496.0	1.21

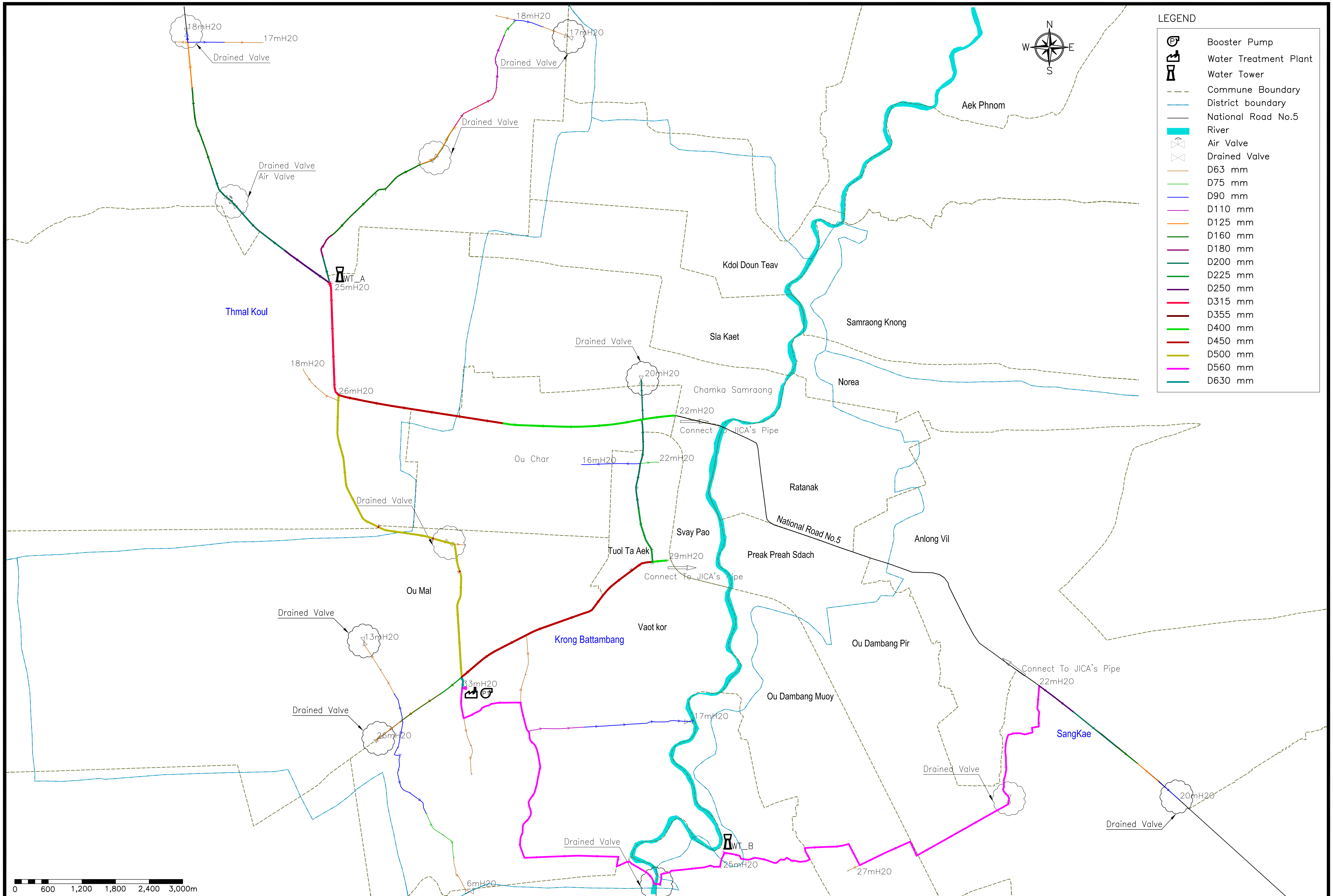
Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-55	J-52	J-53	502	461.8	17,842.0	1.23	17,319.0	1.20
P-56	J-53	J-54	103	461.8	17,386.0	1.20	16,863.0	1.17
P-57	J-53	J-55	495	57.0	213.0	0.96	213.0	0.96
P-58	J-55	J-56	377	57.0	54.0	0.25	54.0	0.25
P-59	J-30	J-57	393	461.8	18,800.0	1.30	18,277.0	1.26
P-60	J-57	J-58	500	461.8	18,716.0	1.29	18,193.0	1.26
P-61	J-58	J-59	500	461.8	18,643.0	1.29	18,120.0	1.25
P-62	J-59	J-60	501	461.8	18,594.0	1.28	18,071.0	1.25
P-63	J-60	J-61	493	461.8	18,522.0	1.28	18,000.0	1.24
P-64	J-61	J-62	397	461.8	18,400.0	1.27	17,878.0	1.24
P-65	J-62	J-63	500	461.8	18,358.0	1.27	17,835.0	1.23
P-66	J-63	J-64	501	461.8	18,352.0	1.27	17,829.0	1.23
P-67	J-64	J-65	500	461.8	18,248.0	1.26	17,725.0	1.22
P-68	J-65	J-66	500	461.8	18,217.0	1.26	17,694.0	1.22
P-69	J-66	J-67	500	461.8	18,149.0	1.25	17,627.0	1.22
P-70	J-67	J-68	500	461.8	18,065.0	1.25	17,542.0	1.21
P-71	J-68	J-69	500	461.8	17,912.0	1.24	17,389.0	1.20
P-72	J-69	J-70	104	461.8	17,787.0	1.23	17,265.0	1.19
P-73	J-70	J-71	5	461.8	9,276.0	0.64	8,685.0	0.60
P-74	J-70	J-72	480	415.6	8,397.0	0.72	8,465.0	0.72
P-75	J-72	J-73	499	415.6	8,203.0	0.70	8,271.0	0.71
P-76	J-73	J-74	500	415.6	8,068.0	0.69	8,136.0	0.69
P-77	J-74	J-75	501	415.6	8,026.0	0.68	8,094.0	0.69
P-78	J-75	J-76	499	415.6	8,005.0	0.68	8,073.0	0.69
P-79	J-76	J-77	501	415.6	7,957.0	0.68	8,025.0	0.68
P-80	J-77	J-78	500	369.4	7,875.0	0.85	7,943.0	0.86
P-81	J-78	J-79	500	369.4	7,717.0	0.83	7,785.0	0.84
P-82	J-79	J-80	501	369.4	7,453.0	0.80	7,521.0	0.81
P-83	J-80	J-81	500	369.4	7,264.0	0.78	7,332.0	0.79
P-84	J-81	J-82	493	369.4	7,068.0	0.76	7,136.0	0.77
P-85	J-82	J-83	400	184.6	(2,325.0)	1.01	(2,257.0)	0.98
P-86	J-83	J-84	397	184.6	(2,413.0)	1.04	(2,345.0)	1.01
P-87	J-84	J-85	429	184.6	(2,901.0)	1.25	(2,833.0)	1.23
P-88	J-85	J-86	378	184.6	(2,995.0)	1.30	(2,928.0)	1.27
P-89	J-86	J-87	500	207.8	(3,203.0)	1.09	(3,135.0)	1.07
P-90	J-87	J-38	503	207.8	(3,375.0)	1.15	(3,307.0)	1.13
P-91	J-39	J-88	504	207.8	3,457.0	1.18	3,389.0	1.16
P-92	J-88	J-89	500	207.8	3,154.0	1.08	3,085.0	1.05
P-93	J-89	J-90	374	184.6	2,907.0	1.26	2,838.0	1.23
P-94	J-90	J-91	431	184.6	2,785.0	1.20	2,717.0	1.17
P-95	J-91	J-92	397	184.6	2,475.0	1.07	2,407.0	1.04
P-96	J-92	J-93	402	184.6	2,397.0	1.04	2,329.0	1.01
P-97	J-93	J-94	579	369.4	7,817.0	0.84	7,817.0	0.84
P-98	J-91	J-95	338	67.8	117.0	0.37	117.0	0.37
P-99	J-84	J-96	498	81.4	358.0	0.80	358.0	0.80
P-100	J-96	J-97	548	81.4	172.0	0.38	172.0	0.38
P-102	J-61	J-98	190	57.0	10.0	0.04	10.0	0.04
P-103	J-71	J-99	480	415.6	8,717.0	0.74	8,785.0	0.75
P-104	J-99	J-100	500	415.6	8,536.0	0.73	8,604.0	0.73
P-105	J-100	J-101	500	415.6	8,371.0	0.71	8,440.0	0.72
P-106	J-101	J-102	500	415.6	8,211.0	0.70	8,279.0	0.71
P-107	J-102	J-103	499	415.6	7,978.0	0.68	8,047.0	0.69
P-108	J-103	J-104	503	415.6	7,804.0	0.67	7,872.0	0.67

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-109	J-104	J-105	498	369.4	7,655.0	0.83	7,723.0	0.83
P-110	J-105	J-106	499	369.4	7,478.0	0.81	7,546.0	0.81
P-111	J-106	J-107	500	369.4	7,361.0	0.79	7,429.0	0.80
P-112	J-107	J-108	500	369.4	7,229.0	0.78	7,298.0	0.79
P-113	J-108	J-109	500	369.4	7,036.0	0.76	7,104.0	0.77
P-114	J-109	J-82	8	369.4	(3,501.0)	0.38	(3,433.0)	0.37
P-115	J-109	J-110	5	369.4	10,417.0	1.13	10,417.0	1.13
P-116	J-110	J-111	577	369.4	7,953.0	0.86	7,953.0	0.86
P-118	J-110	J-112	389	184.6	1,926.0	0.83	1,926.0	0.83
P-119	J-112	J-113	316	184.6	869.0	0.38	869.0	0.38
P-120	J-54	J-114	8	461.8	8,472.0	0.59	8,539.0	0.59
P-122	J-54	J-115	503	290.8	8,847.0	1.54	8,257.0	1.44
P-123	J-115	J-116	501	290.8	8,734.0	1.52	8,144.0	1.42
P-124	J-116	J-117	500	290.8	8,623.0	1.50	8,033.0	1.40
P-126	J-118	J-119	507	230.8	1,324.0	0.37	1,324.0	0.37
P-127	J-119	J-120	501	230.8	1,243.0	0.34	1,243.0	0.34
P-128	J-120	J-121	497	184.6	1,196.0	0.52	1,196.0	0.52
P-129	J-121	J-122	503	184.6	1,127.0	0.49	1,127.0	0.49
P-130	J-122	J-123	497	184.6	1,068.0	0.46	1,068.0	0.46
P-131	J-123	J-124	499	184.6	959.0	0.41	959.0	0.41
P-132	J-124	J-125	495	147.6	821.0	0.56	821.0	0.56
P-133	J-125	J-126	504	147.6	680.0	0.46	680.0	0.46
P-134	J-126	J-127	498	147.6	573.0	0.39	573.0	0.39
P-135	J-127	J-128	812	115.4	511.0	0.57	511.0	0.57
P-136	J-128	J-129	183	81.4	149.0	0.33	149.0	0.33
P-137	J-118	J-130	511	230.8	1,263.0	0.35	1,263.0	0.35
P-138	J-130	J-131	501	230.8	1,225.0	0.34	1,225.0	0.34
P-139	J-131	J-132	498	184.6	1,187.0	0.51	1,187.0	0.51
P-140	J-132	J-133	501	184.6	1,167.0	0.50	1,167.0	0.50
P-141	J-133	J-134	496	184.6	1,099.0	0.48	1,099.0	0.48
P-142	J-134	J-135	498	184.6	978.0	0.42	978.0	0.42
P-143	J-135	J-136	495	147.6	813.0	0.55	813.0	0.55
P-144	J-136	J-137	502	147.6	699.0	0.47	699.0	0.47
P-145	J-137	J-138	499	147.6	632.0	0.43	632.0	0.43
P-146	J-138	J-139	812	115.4	491.0	0.54	491.0	0.54
P-147	J-139	J-140	183	81.4	145.0	0.32	145.0	0.32
P-148	J-139	J-128	5	115.4	(155.0)	0.17	(155.0)	0.17
P-149	J-128	J-141	230	57.0	9.0	0.04	9.0	0.04
P-150	J-139	J-142	651	81.4	166.0	0.37	166.0	0.37
P-151	J-142	J-143	689	57.0	41.0	0.19	41.0	0.19
P-152	J-114	J-144	494	290.8	8,886.0	1.55	8,294.0	1.45
P-153	J-144	J-145	500	290.8	8,762.0	1.53	8,170.0	1.42
P-154	J-145	J-146	500	290.8	8,644.0	1.51	8,052.0	1.40
P-156	J-147	J-148	496	184.6	795.0	0.34	795.0	0.34
P-157	J-148	J-149	442	166.2	718.0	0.38	718.0	0.38
P-158	J-149	J-150	581	147.6	656.0	0.44	656.0	0.44
P-159	J-150	J-151	490	147.6	633.0	0.43	633.0	0.43
P-160	J-151	J-152	481	147.6	577.0	0.39	577.0	0.39
P-161	J-152	J-153	499	147.6	521.0	0.35	521.0	0.35
P-162	J-153	J-154	502	147.6	552.0	0.37	552.0	0.37
P-163	J-154	J-155	447	147.6	496.0	0.34	496.0	0.34
P-164	J-155	J-156	555	115.4	390.0	0.43	390.0	0.43
P-165	J-156	J-157	531	115.4	268.0	0.30	268.0	0.30

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-166	J-157	J-158	470	99.4	213.0	0.32	213.0	0.32
P-167	J-158	J-159	499	99.4	174.0	0.26	174.0	0.26
P-168	J-159	J-160	353	67.8	146.0	0.47	146.0	0.47
P-169	J-147	J-161	498	184.6	770.0	0.33	770.0	0.33
P-170	J-161	J-162	447	166.2	732.0	0.39	732.0	0.39
P-171	J-162	J-163	581	147.6	670.0	0.45	670.0	0.45
P-172	J-163	J-164	490	147.6	615.0	0.42	615.0	0.42
P-173	J-164	J-165	483	147.6	583.0	0.39	583.0	0.39
P-174	J-165	J-166	499	147.6	513.0	0.35	513.0	0.35
P-175	J-166	J-167	499	115.4	312.0	0.34	312.0	0.34
P-176	J-167	J-168	447	115.4	232.0	0.26	232.0	0.26
P-177	J-168	J-169	558	99.4	263.0	0.39	263.0	0.39
P-178	J-169	J-170	526	99.4	280.0	0.42	280.0	0.42
P-179	J-170	J-171	470	99.4	218.0	0.32	218.0	0.32
P-180	J-171	J-172	501	99.4	155.0	0.23	155.0	0.23
P-181	J-172	J-173	351	67.8	113.0	0.36	113.0	0.36
P-182	J-173	J-174	365	57.0	23.0	0.11	23.0	0.11
P-183	J-173	J-160	7	81.4	59.0	0.13	59.0	0.13
P-184	J-160	J-175	493	81.4	148.0	0.33	148.0	0.33
P-185	J-175	J-176	500	57.0	51.0	0.23	51.0	0.23
P-231	J-24	J-20	7	147.6	(575.0)	0.39	(575.0)	0.39
P-232	J-20	J-222	501	81.4	323.0	0.72	323.0	0.72
P-233	J-222	J-223	501	81.4	281.0	0.62	281.0	0.62
P-234	J-223	J-224	500	81.4	254.0	0.56	254.0	0.56
P-235	J-224	J-225	500	81.4	218.0	0.48	218.0	0.48
P-236	J-225	J-226	500	67.8	168.0	0.54	168.0	0.54
P-237	J-226	J-227	500	67.8	98.0	0.31	98.0	0.31
P-238	J-227	J-228	701	57.0	28.0	0.13	28.0	0.13
P-261	J-250	J-251	174	517.2	(26,859.0)	1.48	(25,178.0)	1.39
P-266	J-3	J-256	411	517.2	27,885.0	1.54	26,215.0	1.44
P-267	J-256	J-257	681	517.2	27,854.0	1.53	26,184.0	1.44
P-268	J-11	J-258	713	57.0	80.0	0.36	69.0	0.31
P-269	J-258	J-257	509	57.0	8.0	0.04	(3.0)	0.01
P-270	J-257	J-259	539	517.2	27,769.0	1.53	26,088.0	1.44
P-271	J-259	J-260	501	517.2	27,343.0	1.51	25,662.0	1.41
P-272	J-260	J-261	500	517.2	27,275.0	1.50	25,593.0	1.41
P-273	J-261	J-262	501	517.2	27,216.0	1.50	25,535.0	1.41
P-274	J-262	J-263	501	517.2	27,105.0	1.49	25,423.0	1.40
P-275	J-263	J-264	427	517.2	26,940.0	1.48	25,258.0	1.39
P-276	J-264	J-265	505	517.2	26,859.0	1.48	25,178.0	1.39
P-277	J-265	J-266	496	517.2	26,859.0	1.48	25,178.0	1.39
P-279	J-266	J-251	675	517.2	26,859.0	1.48	25,178.0	1.39
P-280	J-250	J-268	504	517.2	26,859.0	1.48	25,178.0	1.39
P-281	J-268	J-269	458	517.2	26,859.0	1.48	25,178.0	1.39
P-282	J-269	J-270	539	517.2	26,859.0	1.48	25,178.0	1.39
P-283	J-270	J-271	505	517.2	26,859.0	1.48	25,178.0	1.39
P-285	J-272	J-273	500	517.2	13,649.0	0.75	13,649.0	0.75
P-286	J-273	J-274	587	517.2	13,438.0	0.74	13,438.0	0.74
P-287	J-274	J-275	558	517.2	13,305.0	0.73	13,305.0	0.73
P-288	J-275	J-276	227	517.2	13,197.0	0.73	13,197.0	0.73
P-289	J-276	J-277	432	517.2	13,062.0	0.72	13,062.0	0.72
P-290	J-277	J-278	358	517.2	12,927.0	0.71	12,927.0	0.71
P-291	J-278	J-279	427	517.2	12,809.0	0.71	12,809.0	0.71

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-292	J-280	J-279	242	57.0	(67.0)	0.31	(67.0)	0.31
P-293	J-279	J-281	500	517.2	12,623.0	0.70	12,623.0	0.70
P-294	J-281	J-282	494	517.2	12,623.0	0.70	12,623.0	0.70
P-295	J-282	J-283	276	517.2	12,623.0	0.70	12,623.0	0.70
P-296	J-283	J-284	498	517.2	12,623.0	0.70	12,623.0	0.70
P-297	J-284	J-285	500	517.2	12,623.0	0.70	12,623.0	0.70
P-298	J-285	J-286	503	517.2	12,623.0	0.70	12,623.0	0.70
P-299	J-286	J-287	378	517.2	12,623.0	0.70	12,623.0	0.70
P-300	J-287	J-288	502	517.2	12,620.0	0.70	12,620.0	0.70
P-301	J-288	J-289	500	517.2	12,618.0	0.70	12,618.0	0.70
P-302	J-289	J-290	822	517.2	12,575.0	0.69	12,575.0	0.69
P-303	J-290	J-291	447	517.2	12,474.0	0.69	12,474.0	0.69
P-304	J-291	J-292	418	517.2	12,313.0	0.68	12,313.0	0.68
P-305	J-292	J-293	338	230.8	1,551.0	0.43	1,551.0	0.43
P-306	J-293	J-294	436	230.8	1,276.0	0.35	1,276.0	0.35
P-307	J-294	J-295	982	184.6	1,003.0	0.43	1,003.0	0.43
P-308	J-295	J-296	483	147.6	691.0	0.47	691.0	0.47
P-309	J-296	J-297	499	115.4	337.0	0.37	337.0	0.37
P-310	J-297	J-298	513	81.4	91.0	0.20	91.0	0.20
P-345	J-46	J-62	5	415.6	12.0	0.00	11.0	0.00
P-350	J-85	J-90	5	184.6	(46.0)	0.02	(45.0)	0.02
P-354	J-38	J-39	5	207.8	7,342.0	2.51	7,336.0	2.50
P-356	J-118	J-147	17	290.8	(2,723.0)	0.47	(2,723.0)	0.47
P-358	J-122	J-133	5	184.6	(23.0)	0.01	(23.0)	0.01
P-365	J-259	J-331	501	99.4	346.0	0.52	346.0	0.52
P-366	J-331	J-332	501	99.4	346.0	0.52	346.0	0.52
P-367	J-332	J-333	498	81.4	346.0	0.77	346.0	0.77
P-368	J-333	J-334	500	81.4	346.0	0.77	346.0	0.77
P-369	J-334	J-335	501	81.4	321.0	0.71	321.0	0.71
P-370	J-335	J-336	474	81.4	148.0	0.33	148.0	0.33
P-372	J-271	J-337	483	517.2	26,845.0	1.48	25,164.0	1.39
P-388	R-1	PMP-7	16	517.2	22,856.0	1.26	22,140.0	1.22
P-389	PMP-7	J-1	27	517.2	22,856.0	1.26	22,140.0	1.22
P-418	J-117	J-344	511	290.8	8,481.0	1.48	7,891.0	1.38
P-431	J-146	J-345	512	290.8	8,506.0	1.48	7,914.0	1.38
P-434	J-345	J-343	4	290.8	8,506.0	1.48	7,914.0	1.38
P-439	J-25	J-21	5	57.0	(36.0)	0.16	(36.0)	0.16
P-440	J-114	J-71	6	415.6	(470.0)	0.04	189.0	0.02
P-441	R-1	PMP-8	16	517.2	22,856.0	1.26	22,140.0	1.22
P-442	PMP-8	J-1	27	517.2	22,856.0	1.26	22,140.0	1.22
P-444	J-1	J-2	7	581.8	27,474.0	1.20	26,434.0	1.15
P-445	J-6	J-30	14	581.8	30,900.0	1.35	30,307.0	1.32
P-450	J-82	J-93	9	369.4	5,679.0	0.61	5,747.0	0.62
P-454	R-1	PMP-9	16	517.2	22,856.0	1.26	22,140.0	1.22
P-455	PMP-9	J-1	27	517.2	22,856.0	1.26	22,140.0	1.22
P-457	J-168	J-155	5	115.4	(86.0)	0.10	(86.0)	0.10
P-459	R-1	PMP-10	17	517.2	22,856.0	1.26	22,140.0	1.22
P-460	PMP-10	J-1	27	517.2	22,856.0	1.26	22,140.0	1.22
P-463	J-37	J-15	5	327.8	196.0	0.03	200.0	0.03
P-474	J-343	T-6	11	290.8	16,987.0	2.96	15,805.0	2.75
P-475	T-6	J-147	13	290.8	-	0.00	4,417.0	0.77
P-476	J-337	T-8	8	517.2	26,845.0	1.48	25,164.0	1.39

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-477	T-8	J-272	6	517.2	-	0.00	13,780.0	0.76
P-478	J-344	J-343	8	290.8	8,481.0	1.48	7,891.0	1.38



KINGDOM OF CAMBODIA
Ministry of Industry and Handicraft

Supporting the Cities Development Initiative for Asia- Pre-Feasibility Studies and Preliminary Engineering-PWSSP (47285-001)

Consultants:

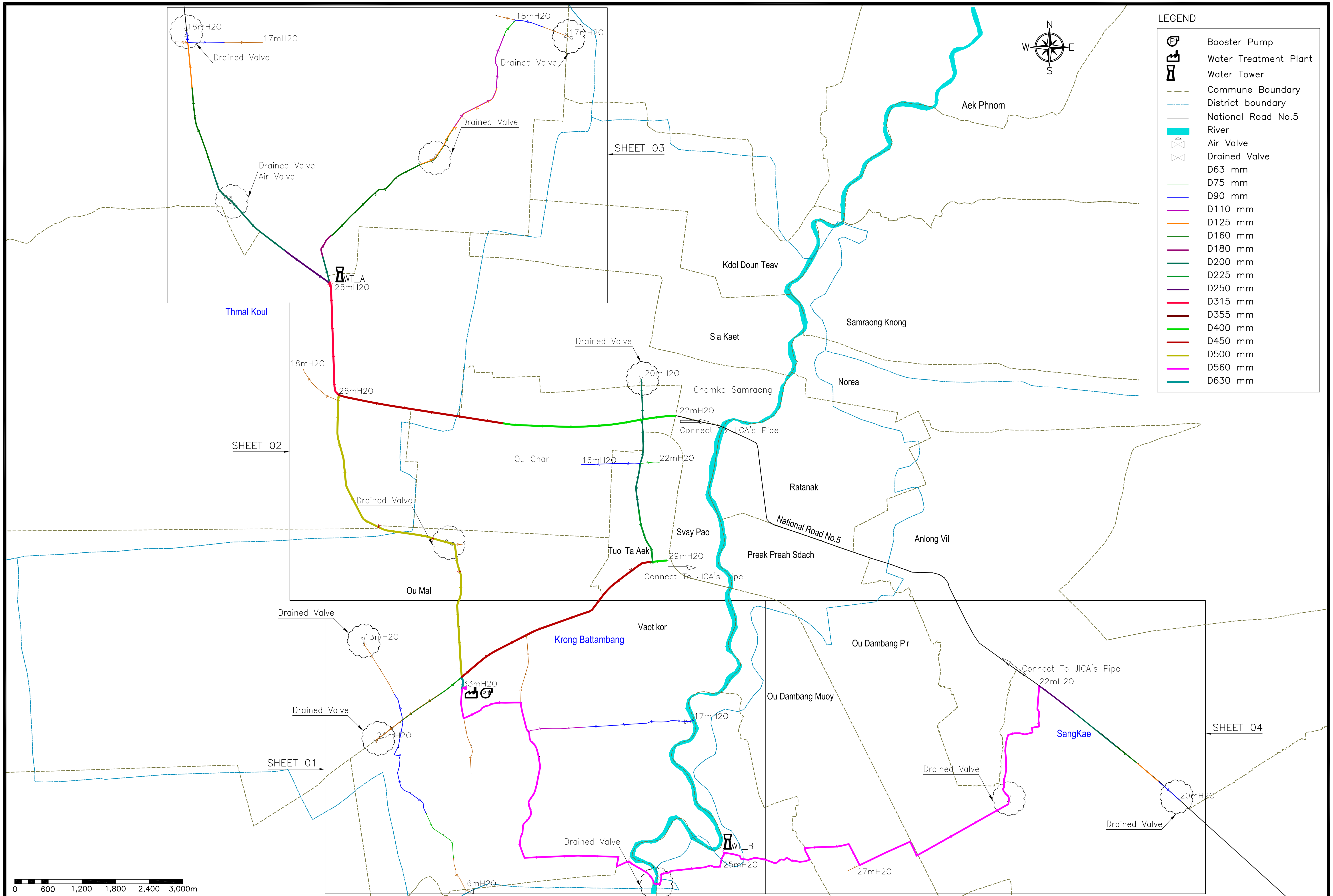
egis in association with

Drawing Title:
Battambang Water Supply Sub-project
Plan View of Pipe Network In Extension Area
In 2025

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale : 1/600

Contract No
Drawing No BTB-WSP-G
Rev. ....



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Consultants: egis in association with

Drawing Title: Battambang Water Supply Sub-project Plan View of Pipe Network In Extension Area In 2025

Consultants	Name	Date
Drawn by :	Chhay Theara	
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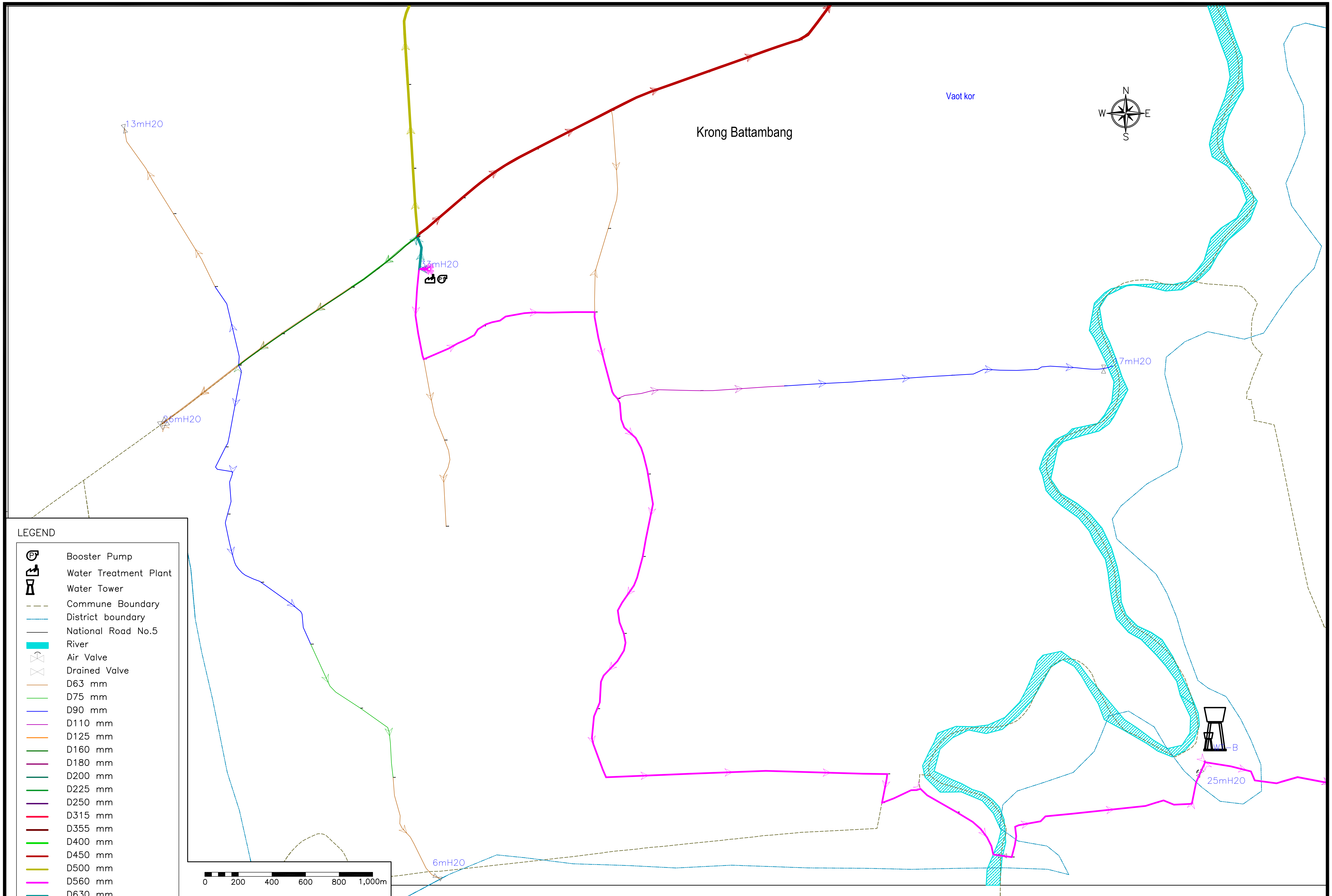
Scale : 1/600

Contract No	
Drawing No	BTB-WSP-G

Rev. ....



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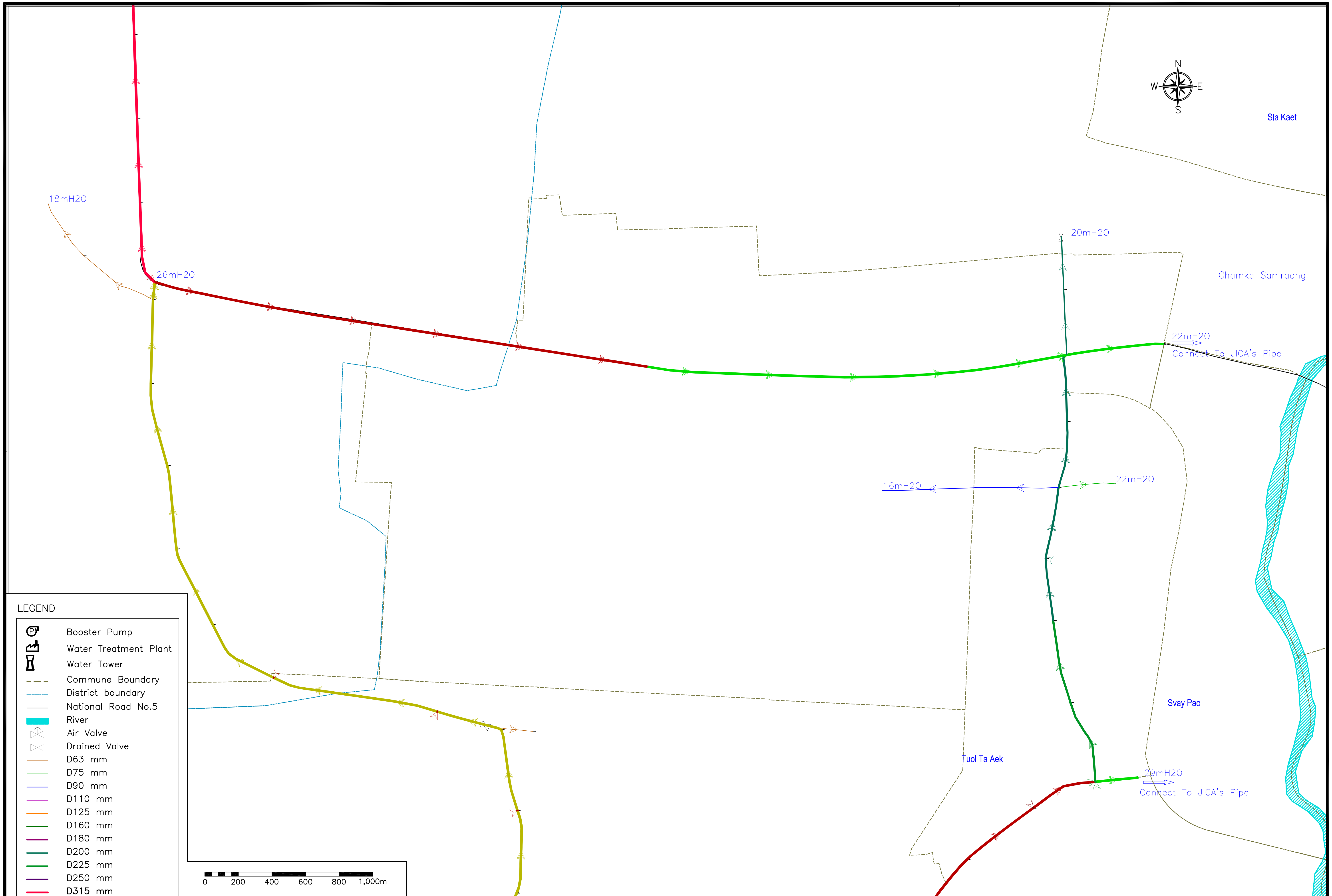
Consultants: egis in association with

Drawing Title: Battambang Water Supply Sub-project
Plan View of Pipe Network In Extension Area
In 2025 (Sheet 01)

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale : 1/200

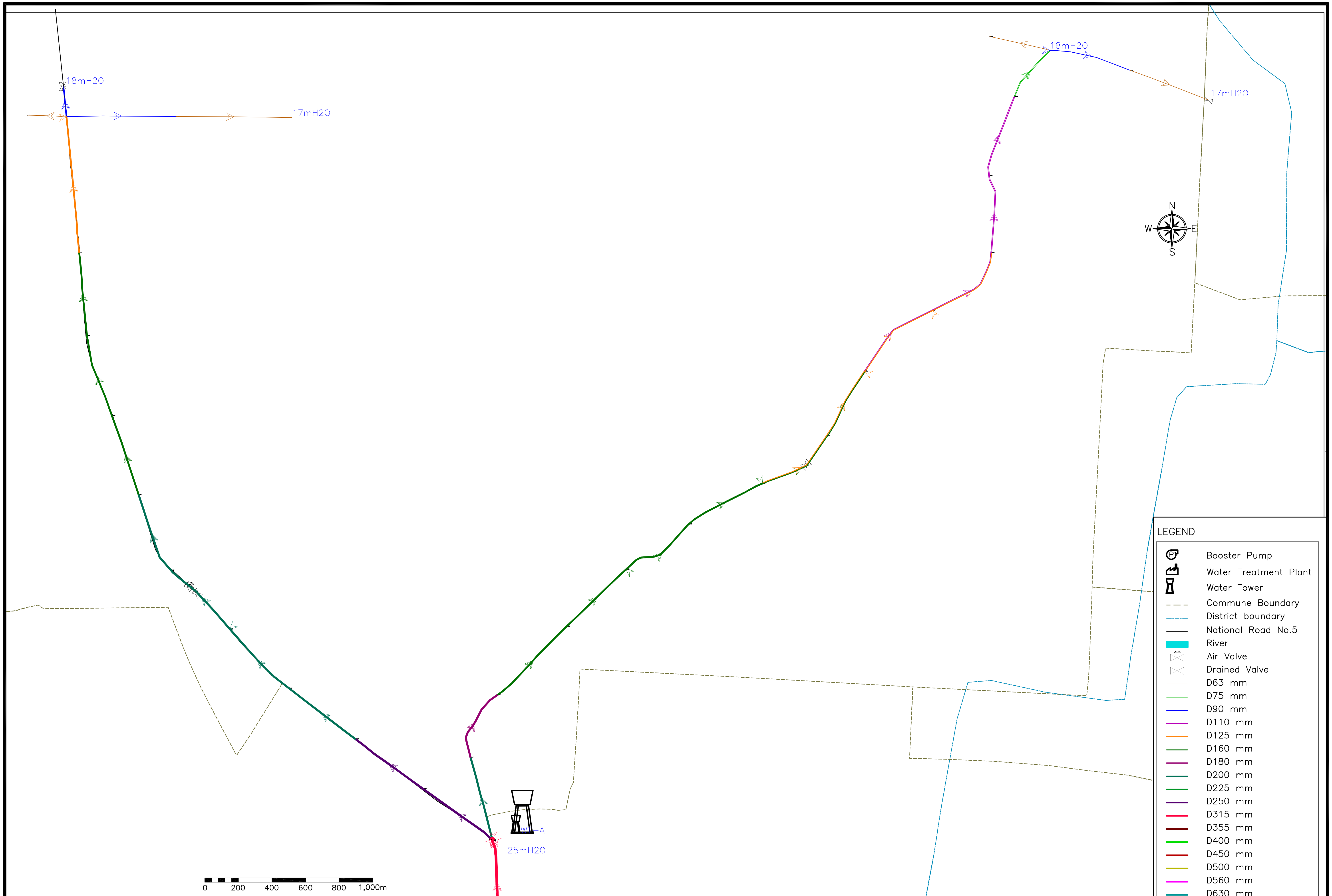
Contract No		
Drawing No	BTB-WSP-S-01	Rev. ....



KINGDOM OF CAMBODIA Ministry of Industry and Handicraft Supporting the Cities Development Initiative for Asia- Pre-Feasibility Studies and Preliminary Engineering-PWSSP (47285-001)	Consultants: egis in association with	Drawing Title: Battambang Water Supply Sub-project Plan View of Pipe Network In Extension Area In 2025 (Sheet 02)

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale :	1/200	
Contract No		Rev. ....
Drawing No	BTB-WSP-S-02	



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Consultants: egis in association with

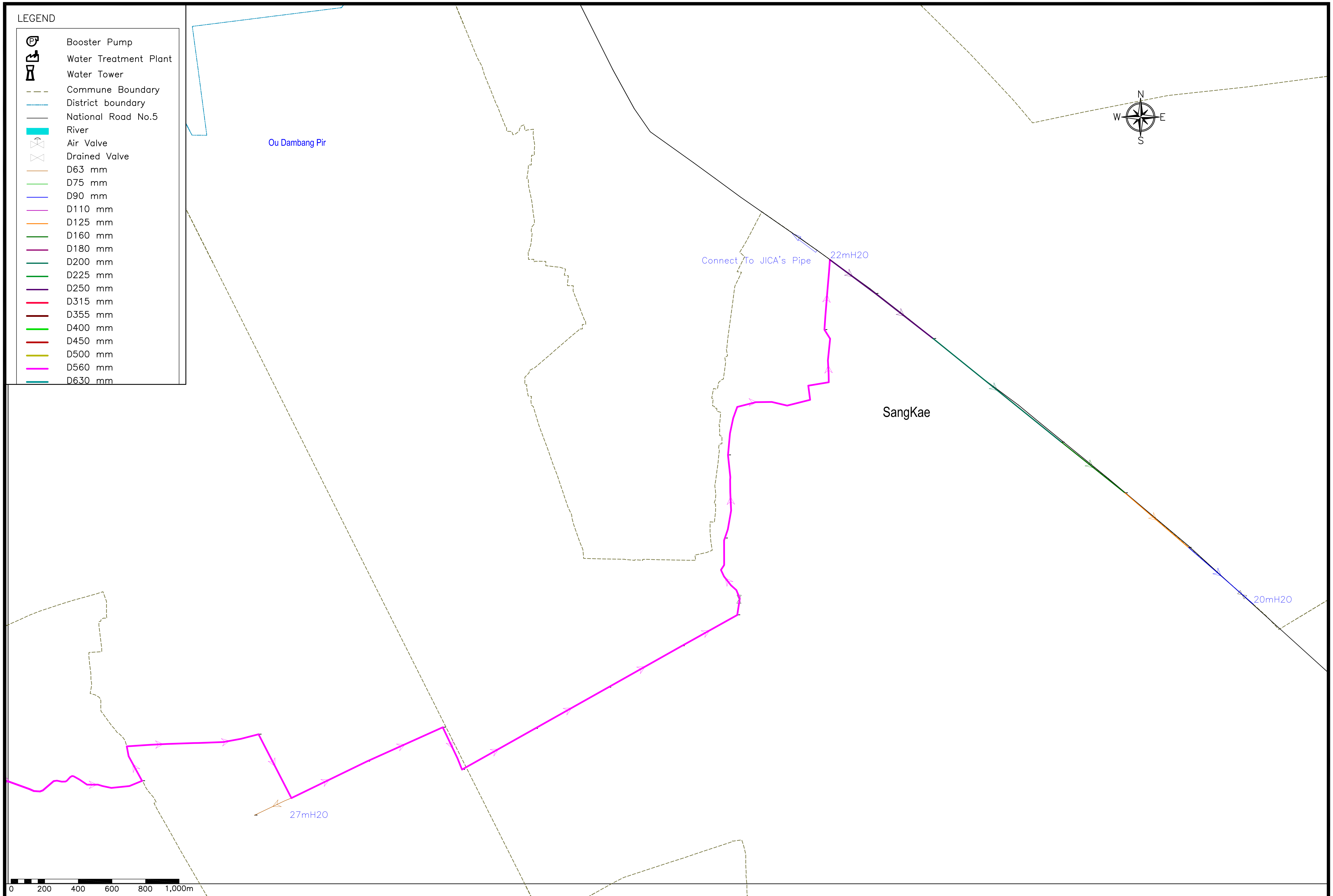
Drawing Title: Battambang Water Supply Sub-project Plan View of Pipe Network In Extension Area In 2025 (Sheet 03)

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale : 1/250

Contract No	
Drawing No	BTB-WSP-S-03

Rev. ....



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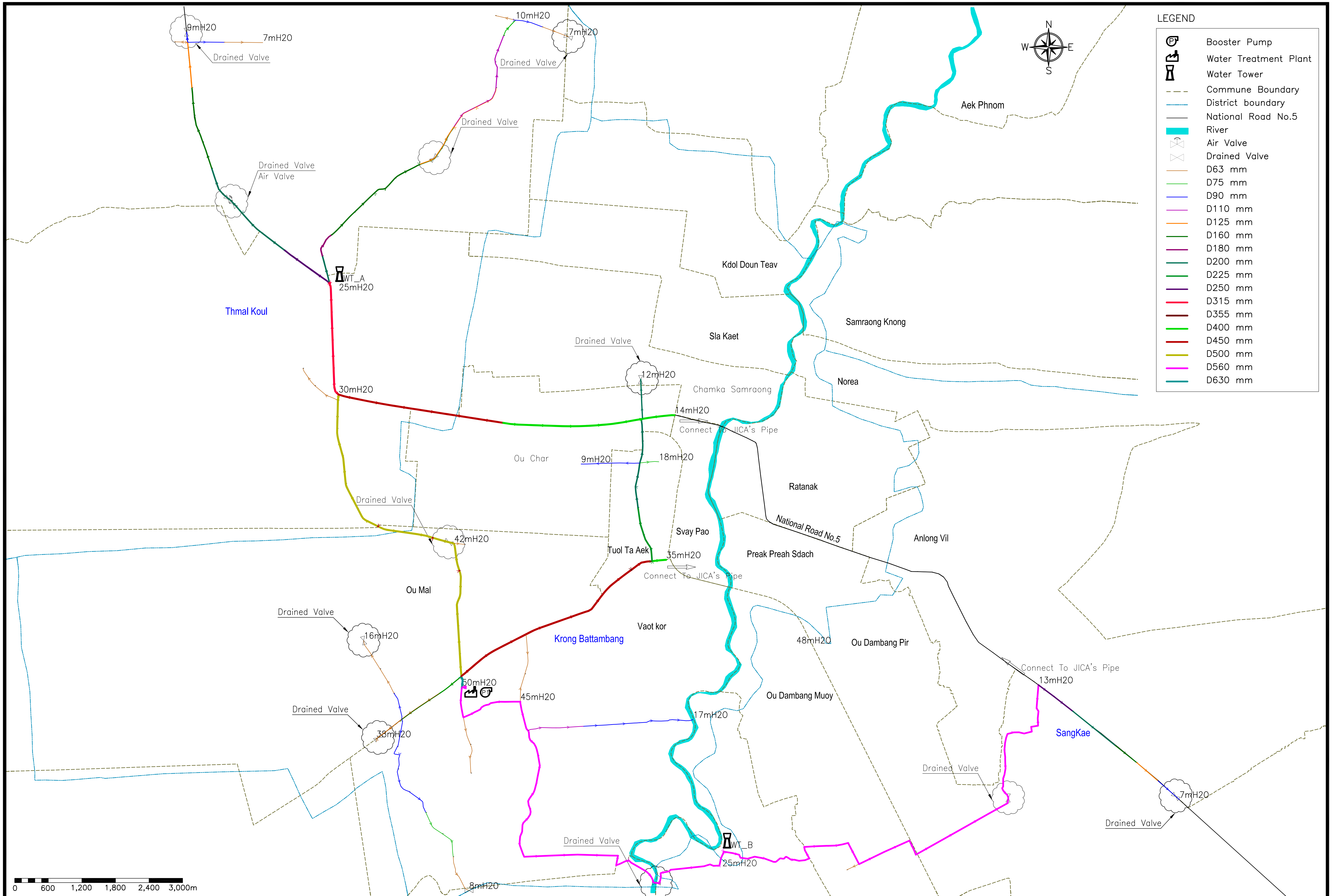
Consultants: egis in association with

Drawing Title: Battambang Water Supply Sub-project
Plan View of Pipe Network In Extension Area
In 2025 (Sheet 04)

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale : 1/250

Contract No		Rev. ....
Drawing No	BTB-WSP-S-04	



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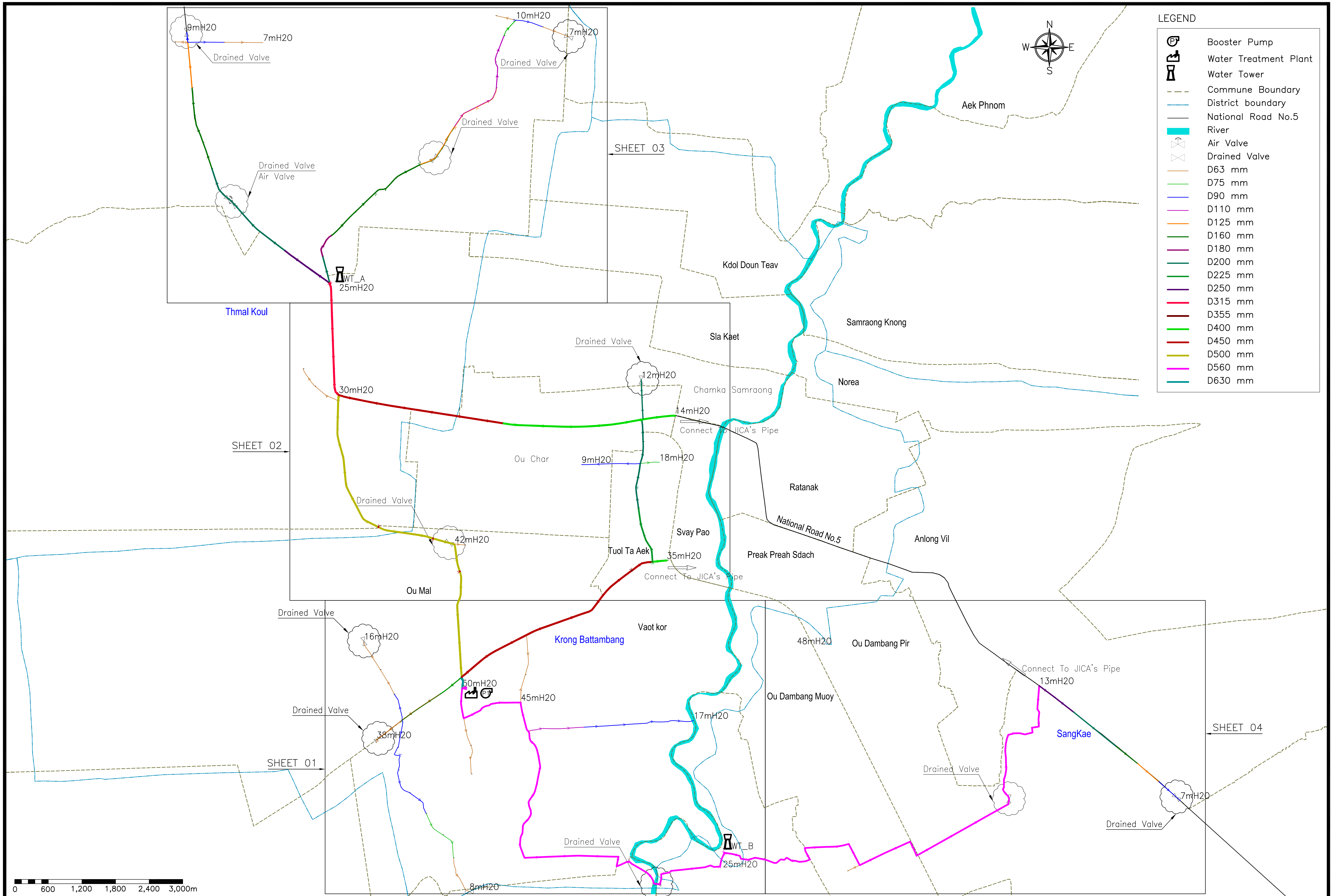
Consultants: egis in association with

Drawing Title: Battambang Water Supply Sub-project
Plan View of Pipe Network In Extension Area
In 2040

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale : 1/600

Contract No
Drawing No BTB-WSP-G
Rev. ....



KINGDOM OF CAMBODIA
Ministry of Industry and Handicraft

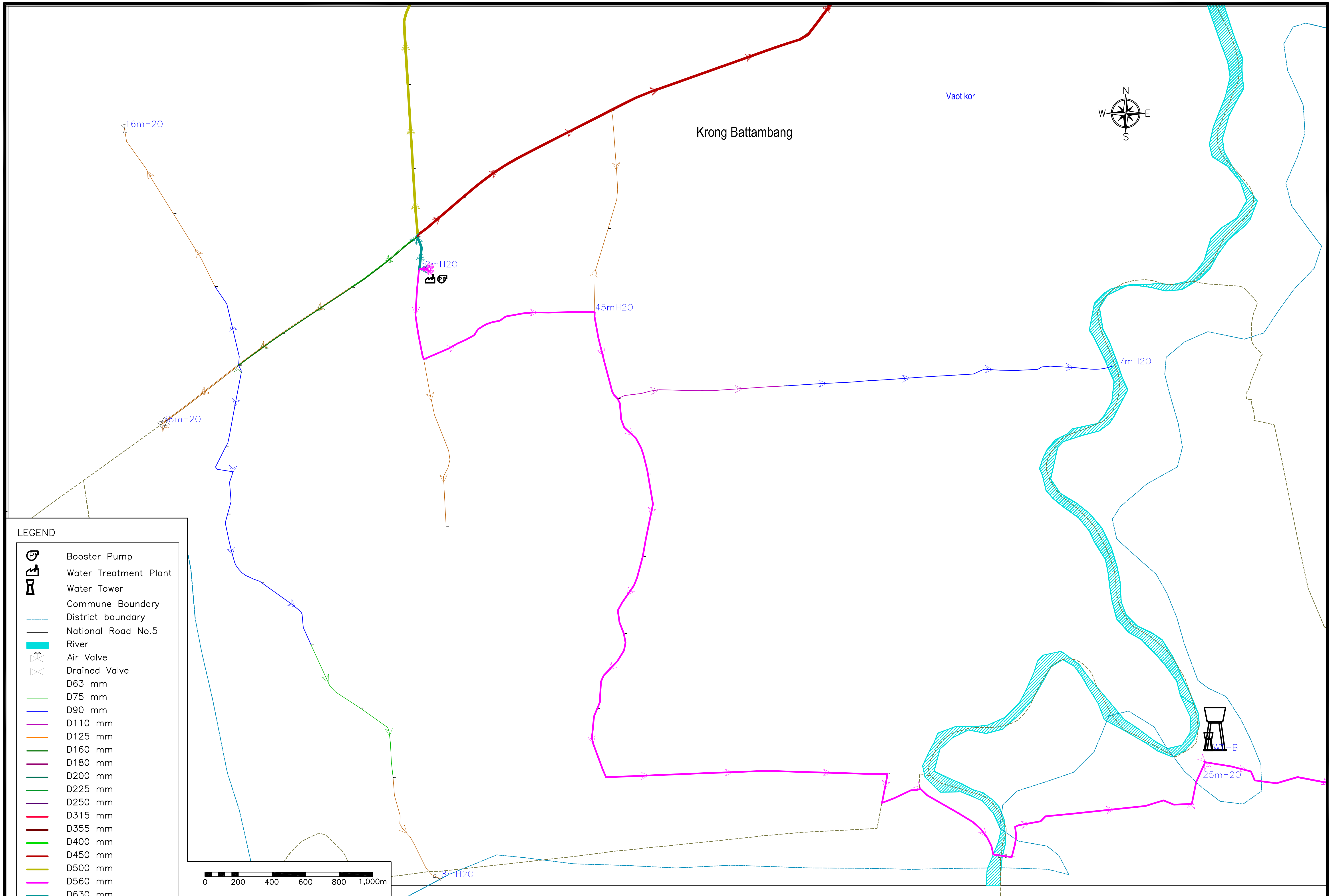
Supporting the Cities Development Initiative for Asia- Pre-Feasibility Studies and Preliminary Engineering-PWSSP (47285-001)

Consultants: egis in association with

Drawing Title: Battambang Water Supply Sub-project Plan View of Pipe Network In Extension Area In 2040

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale :	1/600
Contract No	
Drawing No	BTB-WSP-G
Rev.	....



KINGDOM OF CAMBODIA
Ministry of Industry and Handicraft

Supporting the Cities Development Initiative for Asia- Pre-Feasibility Studies and Preliminary Engineering-PWSSP (47285-001)

Consultants: egis in association with

Drawing Title: Battambang Water Supply Sub-project
Plan View of Pipe Network In Extension Area
In 2040 (Sheet 01)

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale :	1/200	
Contract No		Rev. ....
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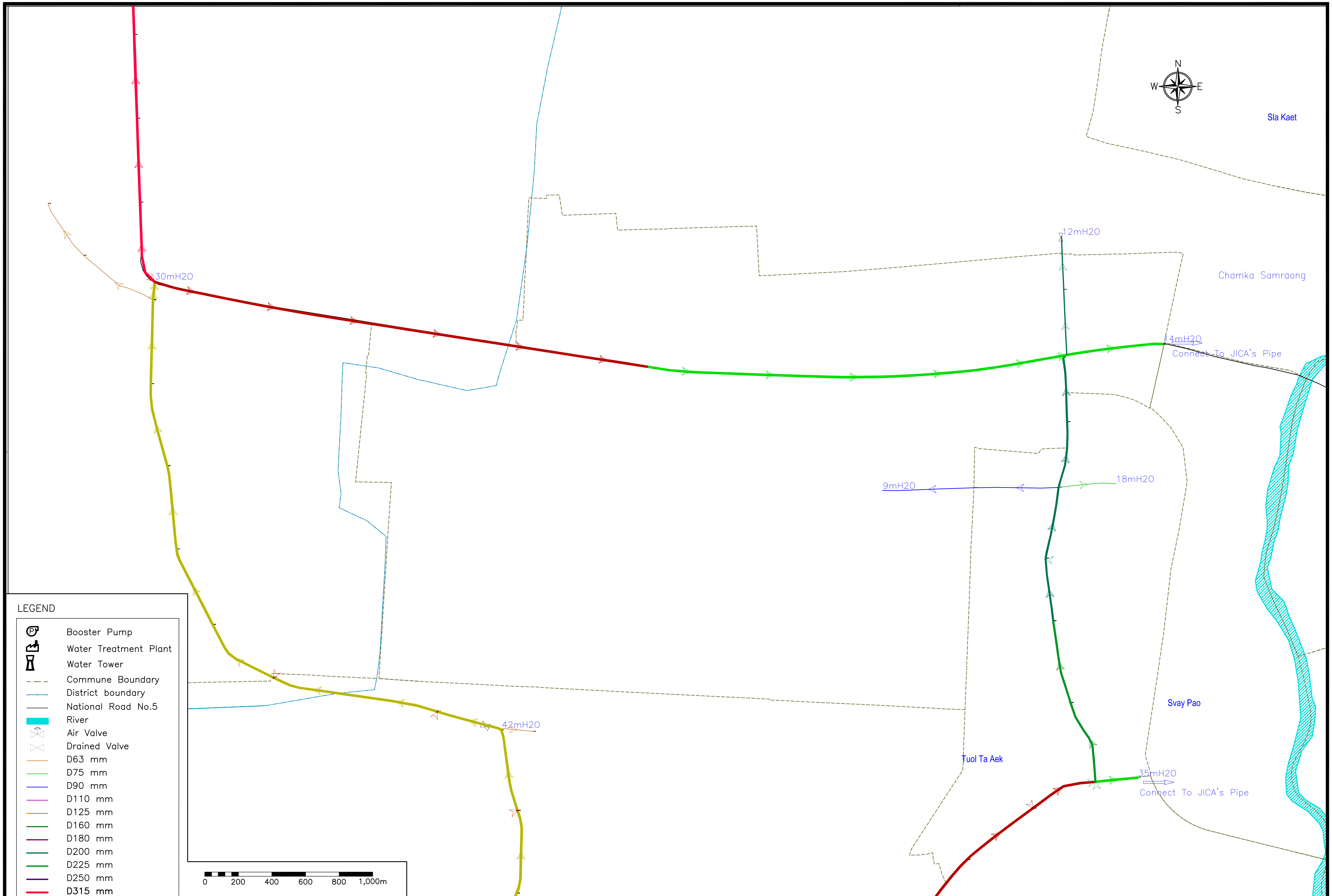

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Consultants: egis in association with

Drawing Title: Battambang Water Supply Sub-project
Plan View of Pipe Network In Extension Area
In 2040 (Sheet 02)

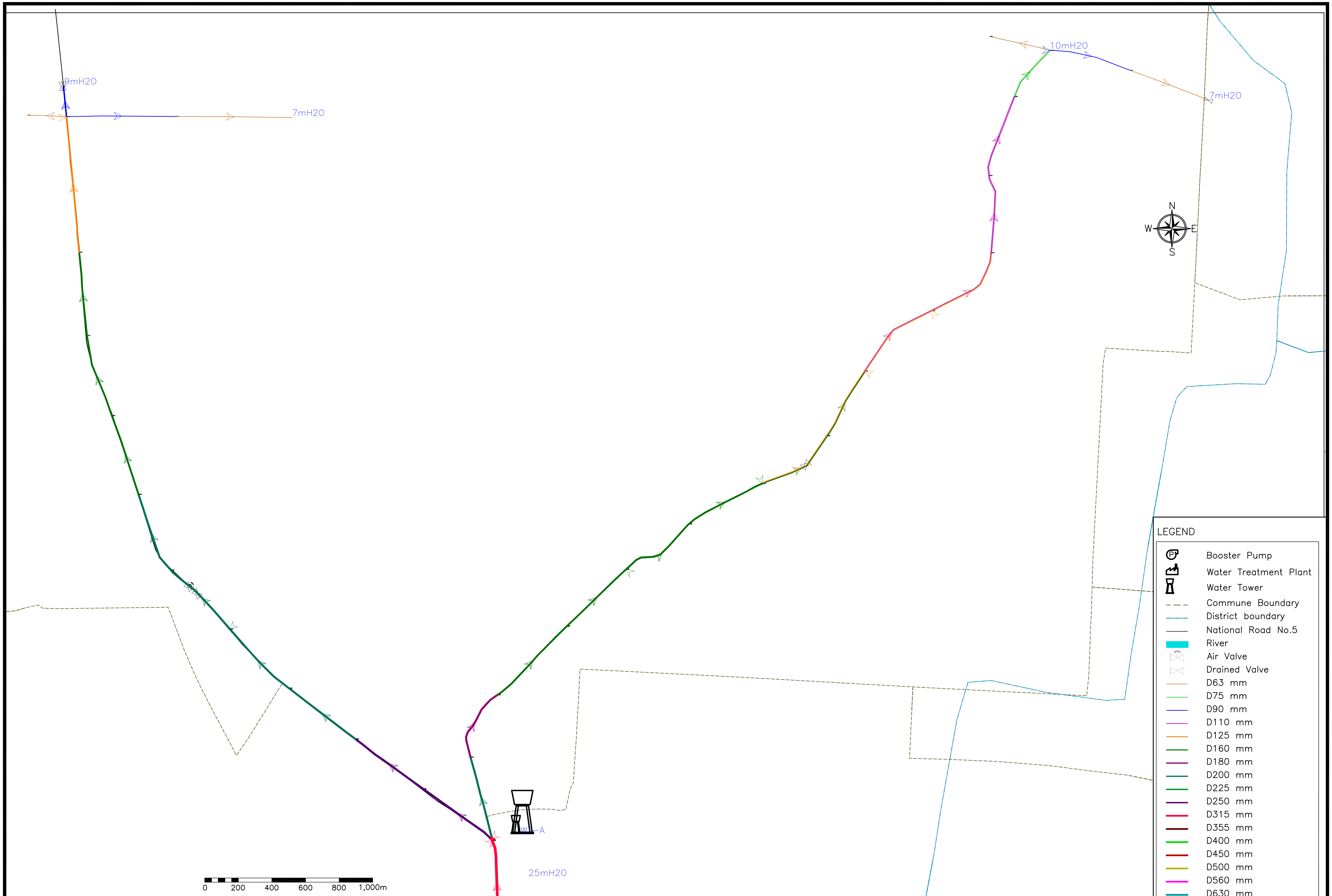
Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale : 1/200

Contract No

Drawing No BTB-WSP-S-02

Rev. ....



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Supporting the Cities Development Initiative for Asia- Pre-Feasibility Studies and Preliminary Engineering-PWSSP (47285-001)

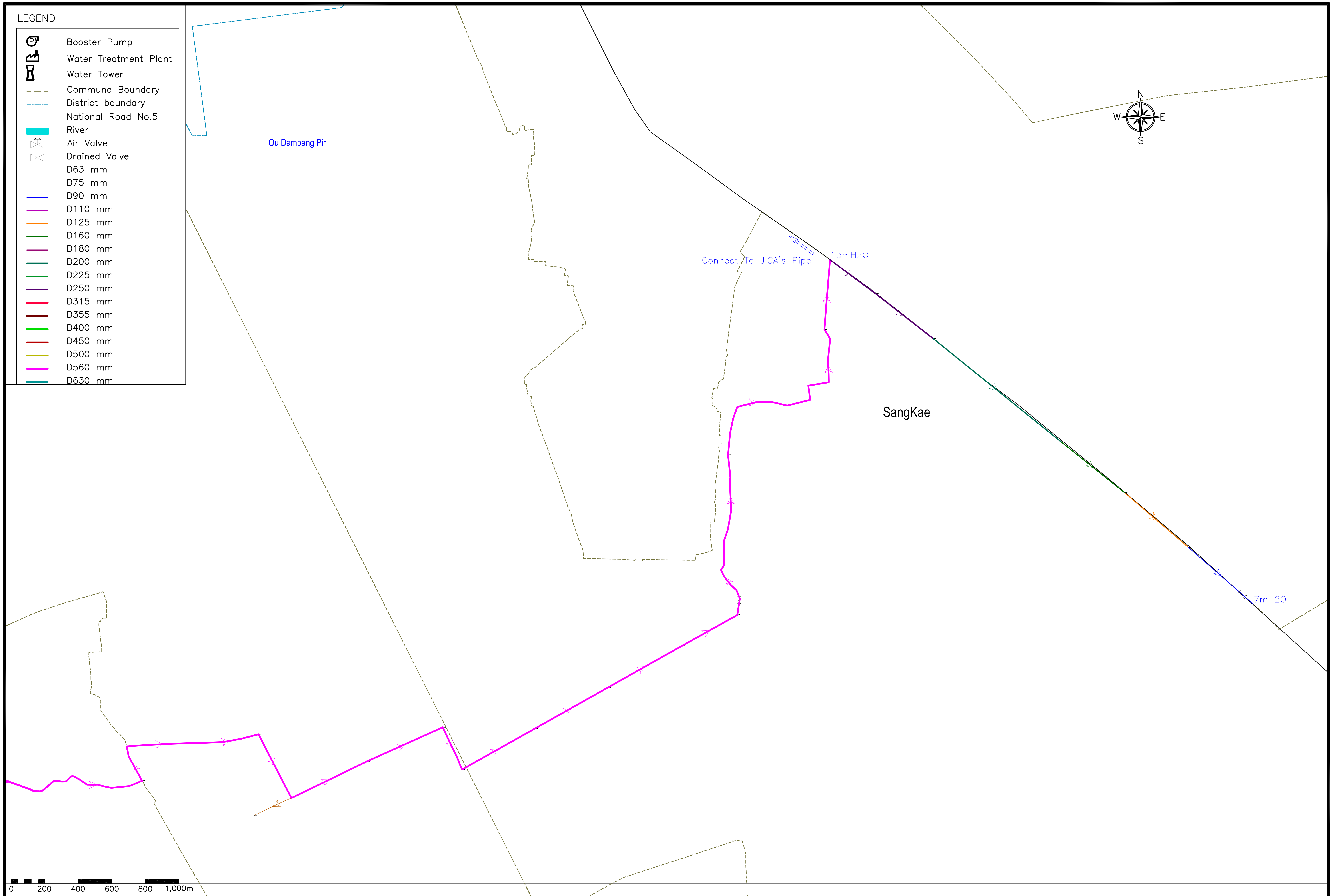
Consultants: egis in association with

Drawing Title: Battambang Water Supply Sub-project
Plan View of Pipe Network In Extension Area
In 2040 (Sheet 03)

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale : 1/250

Contract No		Rev. ....
Drawing No	BTB-WSP-S-03	



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Supporting the Cities Development Initiative for Asia- Pre-Feasibility Studies and Preliminary Engineering-PWSSP (47285-001)

Consultants: egis in association with

Drawing Title: Battambang Water Supply Sub-project
Plan View of Pipe Network In Extension Area
In 2040 (Sheet 04)

Consultants	Name	Date
Drawn by :	Chhay Theara	
Design by :	Sok Bunheng	
Checked by :	Taing Sophannara	
Approved by :	Michael Lee	

Scale : 1/250

Contract No		Rev. ....
Drawing No	BTB-WSP-S-04	

Appendix 2
Preliminary Engineering Design Report – Kampong Cham Water Supply Subproject

Appendix H2 Preliminary Engineering Design Report – Kampong Cham Water Supply Subproject

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1 Introduction

1. Kampong Cham is the provincial town of Kampong Cham province, located in the center of the country. The town has a total area of 22.15 Km^2 and is divided into 4 Sangkats with 32 villages, as shown in the figure below. Many economic development activities have increased in recent years, including tourism. However, the urban infrastructure especially water supply is still limited. In 2015 only 41% of the population of the original service area was connected to the water supply system. With the ADB funded WTP of 8,500m3/d and the 11,500 m3/d WTP recently constructed under a JICA Project (Phase II) in operation since July 2016, the total WTP capacity of Kampong Cham waterworks is 19,500 m3/d.

2. From the rapid assessment using selected criteria and a weighting system approved with Ministry of Industry and Handicraft (MIH), site investigations and discussion with Kampong Cham's Waterworks in April 2016, Kampong Cham was selected for inclusion in the CDIA project for a water supply sub-project. This was confirmed by all stakeholders during the Inception Workshop dated 28 April 2016.

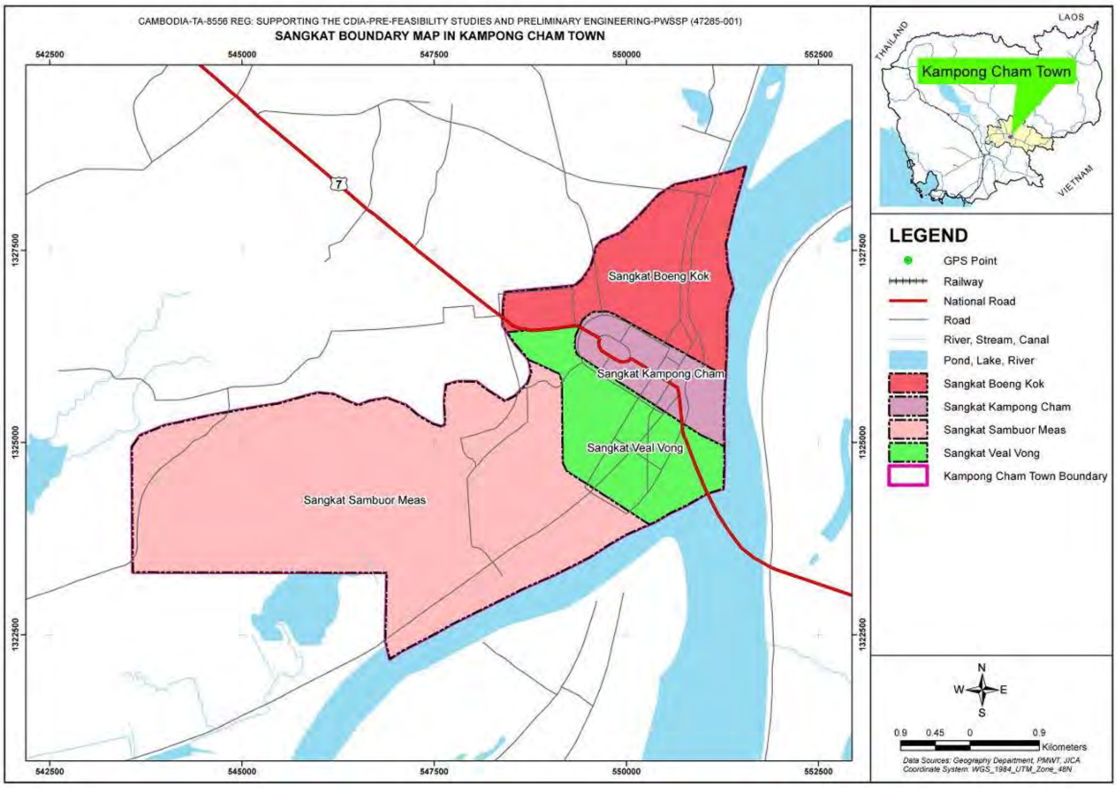
3. This subproject aims at fulfilling the gap in between the existing capacity including JICA Phase II project for the original service area and future demand within the extended service areas.

4. The proposed subproject components will supplement the existing system and support RGC's policy on water supply in providing safe water to all people by 2025. To cover the water demand of next 20 years (2040), and following the proposed served population coverage of 90-100% in Kampong Cham and 85% of the three communities nearby the town, the main components of the sub-project are:

   - A New WTP of at least 26,100 m3/d to provide sufficient capacity until 2040;
   - 120km of network (coverage area of 3,489 ha);
   - Served population (to 2040): approximately of 31,182 families with total 140,318 persons, out of which 71,562 are women (50.9%).

5. The construction of the WTP and networks would be phased to meet the demand of target Year 2025 initially with a design capacity of 11,600m3/d and is proposed to be under a Design and Build (DB) contract (Yellow Fidic), and tendered on a performance basis with a 1 year contract period. The network is to be implemented under a Build contract (Red Fidic) following detailed design to be performed by PIAC/DPIH, with a 2.5 year contract period from 2018 to 2020.

Figure 1. Kampong Cham provincial town showing sangkhat boundaries



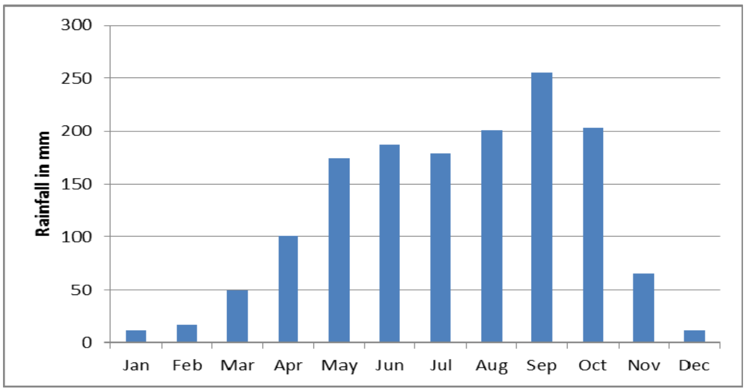
2 Present Condition

2.1 Climate

Rainfall

6. Cambodia's climate is tropical monsoonal divided into two seasons, rainy and dry. As shown in Figure 2. , the rainy season regularly starts in late May and finishes in late October, and the dry season covers November to April/May. Kampong Cham is situated on the west side of the Mekong and lies on the central flood plain of the country, with low precipitation. A total average annual rainfall of 1,456mm (1981-2011) was found from the data of the Department of Meteorology in Phnom Penh and the Office of Meteorology in Kampong Cham.

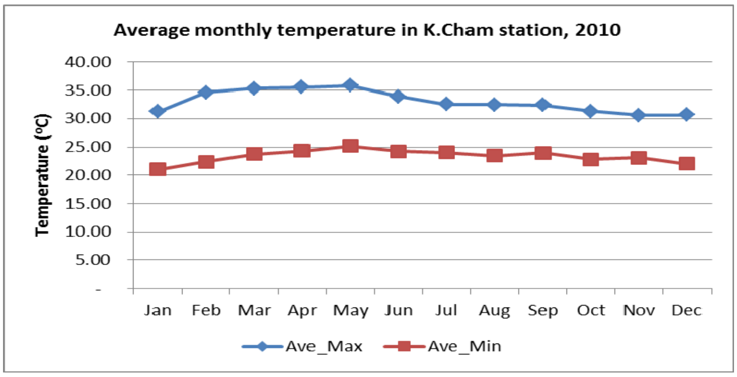
Figure 2. Monthly rainfall in Kampong Cham station



Temperature

7. Based on data recorded at the Kampong Cham Meteorology Station from 2008 to 2010, the lowest monthly mean temperature is 21.3°C, the average annual minimum temperature is 24.2°C, and the average annual maximum temperature is 33°C. The hottest monthly temperature is 39.9°C in April, and the coolest 16.5°C in January. Figure 3. shows average monthly temperatures in Kampong Cham in 2010.

Figure 3. Average monthly minimum and maximum temperature in Kampong Cham in 2010

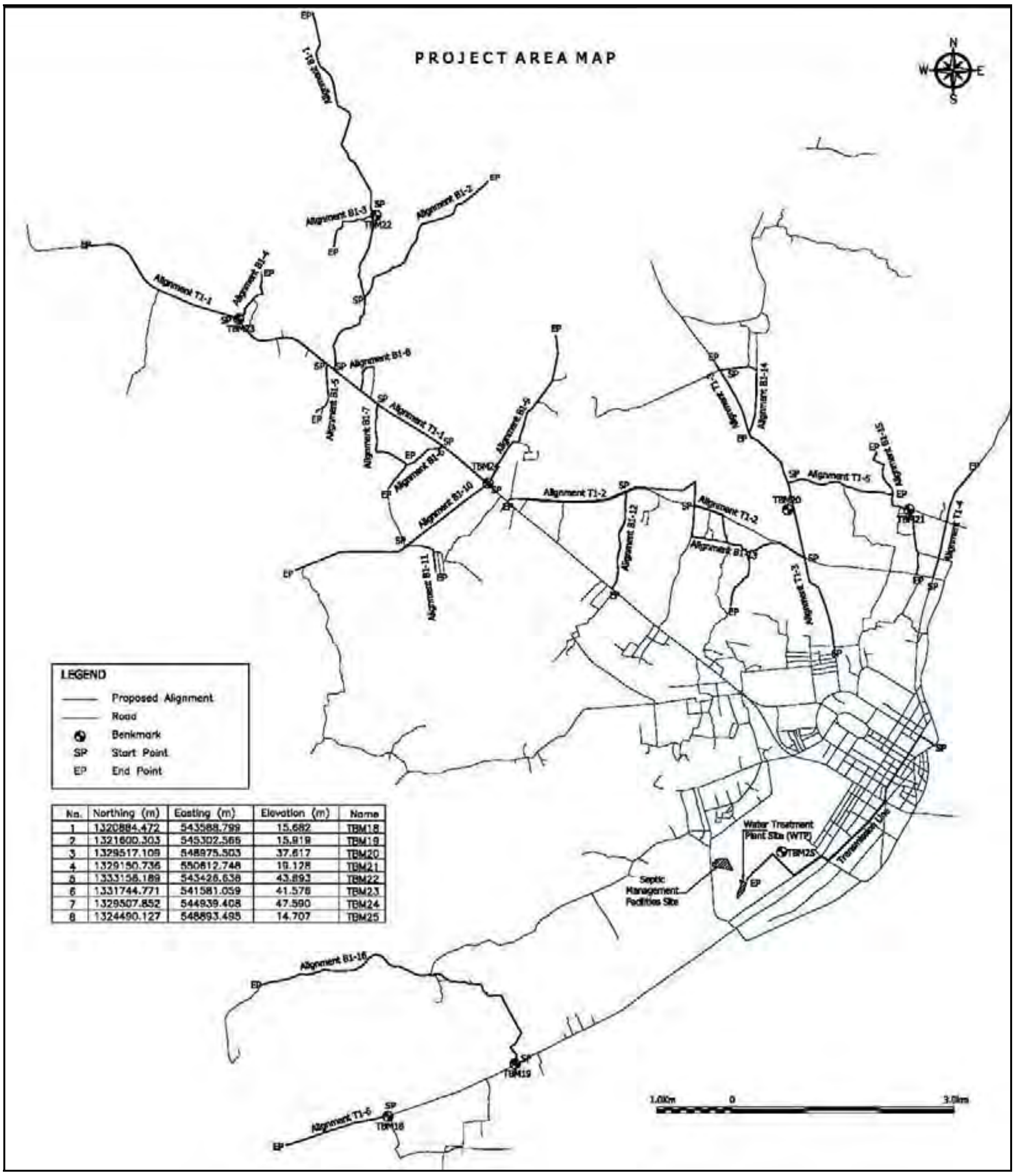


2.2 Geography and Topography

8. Kampong Cham is the provincial town of Kampong Cham province, in the central part of Cambodia. The town is situated on the bank of the Mekong and extends 0.5-1.5km to the west. Topo survey carried out under the CDIA Project showed the new WTP site to be relatively flat with elevation between 12.10 -12.80 amsl, while the ground level along the main road toward Phnom Penh, to the west of the town gradually rises to 47 amsl at the Phnom Pros area.

9. Kampong Cham's primary geological feature consists of three basalt outcrops that form hills. These are surrounded to the southeast by old alluvium, whilst all other areas are young alluvium. The higher parts of the hills are basalt, surrounded by good quality soil formed from deposition around them. The soil in and surrounding the project area is of various types such as great lake, alluvial lithosols, alumisols, and cultural hydromophic. Figure 4. presents the roads included under the topographic surveys of the proposed coverage expansion areas carried out under this subproject.

Figure 4. Topographical survey study, August 2016



No.	Northing (m)	Easting (m)	Elevation (m)	Name
1	1320884.472	543588.799	15.682	TBM18
2	1321600.303	545302.565	15.919	TBM19
3	1329517.109	548975.503	37.617	TBM20
4	1329150.736	550812.748	19.128	TBM21
5	1333158.189	543428.638	43.893	TBM22
6	1331744.771	541581.059	41.578	TBM23
7	1329507.852	544939.408	47.590	TBM24
8	1324490.127	548893.495	14.707	TBM25

2.3 Existing Water Supply System

10. In 2015, Kampong Cham water supply covered about 79% of the total families (8,823) with 6,931 connections and 36,041 served people in the current service area. The existing capacity of the WTP is sufficient to produce 8,000m3/d and the new JICA WTP capacity is an additional 11,500 m3/d, commissioned in July 2016. The water source of Kampong Cham Waterworks is from groundwater for the existing supply network, but from Mekong river for the new JICA project. Based on billing records for the period 2012 to 2015, the average daily water consumption is 130-140 liters/capita/day (rainy and dry season). The system is under the management of the Department of Industry and Handicraft in Kampong Cham Province.

11. The water supply system is 24 hours/day by Kampong Cham Waterworks which is under the management of the Department of Industry and Handicraft (DIH) of Kampong Cham Province.

12. The cost of water production per cubic meter in August and September 2016 is detailed in Table 1. The tariff set by MIH is categorized by amount of water consumed, wth a higher tariff for higher consumption as shown in Table 2.

Table 1. Breakdowns of production costs in 2013 and 2014

Cost, Riel/m^3	Aug.2016	Sept.2016
Salary	162.69	16.30
Electricity	264.95	281.29
Diesel Oil	19.60	25.92
Chemical	77.88	79.70
O&M	245.30	152.52
Amortization	178.54	167.01
Administration	27.59	28.82
Others (including Tax)	9.77	7.52
Total	**986.31**	**759.07**

Table 2. MIH water tariff

Quantity, m^3	Riel/m^3	US$/m^3
0-6	900	0.23
7-15	1,250	0.31
>15	1,600	0.40

Note: Exchange rate 4000R = 1US$

13. There are four recent or ongoing water sector projects: ADB TPI, MEKWATSAN/UNHABITAT, ADB-UWSP and , JICA Phase II. The projects facilities are described below.

Table 3. Existing ADB/TPI, MEKWATSAN, ADB-3232, JICA Phase II facilities

Facilities	Capacity	Remarks/ Funding
WTP	- 8,500m3/d - 11,500m3/d	- ADB TPI on groundwater - JICA Phase II from surface water
Well	3 wells (1 new and 2 old wells)	ADB-UWSP: ongoing
Pipe network	- 94 Km - 14.5 Km - 14 Km - 57.8 Km	- ADB TPI - MEKWATSAN/UNHABITAT - ADB-UWSP - JICA Phase II

Source: Data obtained from KC waterworks, May 2016

14. In July 2016, a new water treatment plant (JICA phase II) was commissioned with a capacity of 11,500m3/d. The JICA II project also consists of a new intake facility with pumping capacity of 12,650m3/d, raw water transmission pipeline of 0.9Km, and distribution pipeline of 57.8Km. The coverage area of the JICA phase II project is 2.297ha as shown in Figure 5. , and the detailed components and specifications are described in Table 4. and Table 5. based on the design study report.[1]

Table 4. Intake facilities of JICA Phase II

Facility			Component
Intake	Intake Shaft	Main body	Reinforced Concrete Structure Rectangular Shape: W7.65 m x L10.70m (inner dimension) Depth 18.45 m (Depth at HWL 16.75 m)
		Operation Room of Intake Pump	Reinforced Concrete Structure Rectangular Shape: W5.50 m x L11.80 m x H5.10 m (under the beam) (inner dimension) Equipment: Power Receiving Panel, Operating Panel, Switchboard, Secondary Equipment Panel, Circumference Plumbing of Pump, Overhead Crane (3t)
	Intake Pump Facility	Intake Pump	Vertical Shaft Type Mixed Flow Pump 4 sets (Ordinary Use 3 sets, Spare 1 set) Q=2.93 m^3/min h=25.8 m P=22 KW 3ΦP380V 50Hz
		Generator	Reinforced Concrete Structure Rectangular Shape: W5.50 m x L4.90 m x H4.30 m (under the beam) (inner dimension) Generator: 60 KVA (Soundproofing Type)
Raw Water Transmission	Transmission Main		DIPΦ400, L = 920 m

Source: "Preparatory Survey on the Project on Additional New Water Treatment Plants for Kampong Cham and Battambang Waterworks in the Kingdom of Cambodia, March 2013, JICA"

Table 5. Water Treatment Plant facilities of JICA Phase II

Facility/Equipment		Component
Water Treatment Plant	Receiving Well	Reinforced Concrete Structure Internal Dimension: 1.60 m width x 4.10 m length x 4.50 m depth Volume and Detention Time: V=29.5 m3, T=3.4 min in dry season (T≥1.5 min)
	Rapid Mixing Tank	Reinforced Concrete Structure Gravitational force mixing using a weir Internal Dimension: 1.60 m width X 1.50 m length X 3.88 m depth Volume and Detention Time: V=9.3 m3, T=11 min (1<T< 5 min)
	Flocculation Basin	Reinforced Concrete Structure Slow Mixing Method: Vertical channel bands flocculator Internal Dimension: 6.90 m width x 3.25 m length x 4.50 m height + 3.46 m average effective depth Quantity : 3
	Sedimentation Basin	Reinforced Concrete Structure Horizontal-Flow Sedimentation Basin Supernatant Collecting System: Collecting Trough + Submerged Orifice Internal Dimension: 6.90 m width x 21.50 m length x 3.99 m average depth Quantity: 3 Surface Loading: Q/A=20.0 mm/min (15-30 mm/min) Mean Velocity: V=0.11 m/min (below 0.40 m/min)

[1] The project completion report is not yet available.,

Facility/Equipment		Component
	Rapid Sand Filter	Reinforced Concrete Structure Internal Dimension: 2.50 m width x 7.00 m length Quantity: 6 Filter Sand Thickness: 100 cm Underdrain System: Porous Filter Bed Method Filtration Rate: V=120.5 m/day (120-150 m/day) Flow Control: Lower Part Control Method Backwash Method: Simultaneous Backwash Method by Air and Water
	Treated Water Reservoir	Reinforced Concrete Structure using Flat Slab Structure Quantity: 2 Effective Volume: V=2,500 m3 (1,250 m3 x 2) Effective Water Depth: H=3.80 m (3-6 m) Detention Time: T=5.2 hours Internal Dimension: 10.40 m width x 32.00 m length x 4.50m height
	Wastewater Basin	Reinforced Concrete Structure Quantity: 2 Effective Volume: V=211 m3 (105.5 m3 X 2) Internal Dimension: 4.00 m width x 11.00 m length x 5.60m height + 2.40m depth
	Sludge Drying Bed (Lagoon)	Reinforced Concrete Structure Quantity: 4 Effective Area: A=790 m^2
	Chemical Feeding Facility	Alum, Lime: at Administration Building Chlorine: Chlorination House (Floor Area 61.3m^2)
	Emergency Generator	450 KVA (Soundproof type, Equipped with Internal Water Tank) At Administration Building
	Administration Building	Reinforced Concrete Structure, Three Storey Building, Total Floor Area: 588 m2 Usage: 1st Floor: Staff Room, Workshop, Storage, Emergency Generator Room, Toilet, Chemical Carry-in Room (1-3 Fl. Open Ceiling) 2nd Floor: Manager Room , Meeting Room, Monitoring Room, Toilet, Chemical Dissolving Tank Room (2-3 Fl. Open Ceiling), Chemical Feeding Pump Room 3rd Floor: Chemical Dissolving Tank Room (2-3 Fl. Open Ceiling) Common: Staircase

Source: "Preparatory Survey on the Project on Additional New Water Treatment Plants for Kampong Cham and Battambang Waterworks in the Kingdom of Cambodia, March 2013, JICA"

15. After JICA Phase II commissioning, the total capacity of water treatment plants in Kampong Cham is 19,500m3/d with a total network of 166.5km and overall coverage area of 4,311ha (43km2) including Ro-Ang and Ampil communes in the Kampong Siem District. The existing pipe network is shown in Figure 6. The Kampong Cham waterworks plans to connect 1,400 households per year in the future, but the current water supply production capacity can only provide sufficient water for the current service area up to 2019[2].

[2] "Preparatory Survey on the Project on Additional New Water Treatment Plants for Kampong Cham and Battambang Waterworks in the Kingdom of Cambodia, March 2013, JICA".

Figure 5. Coverage area of the existing and JICA Phase II project

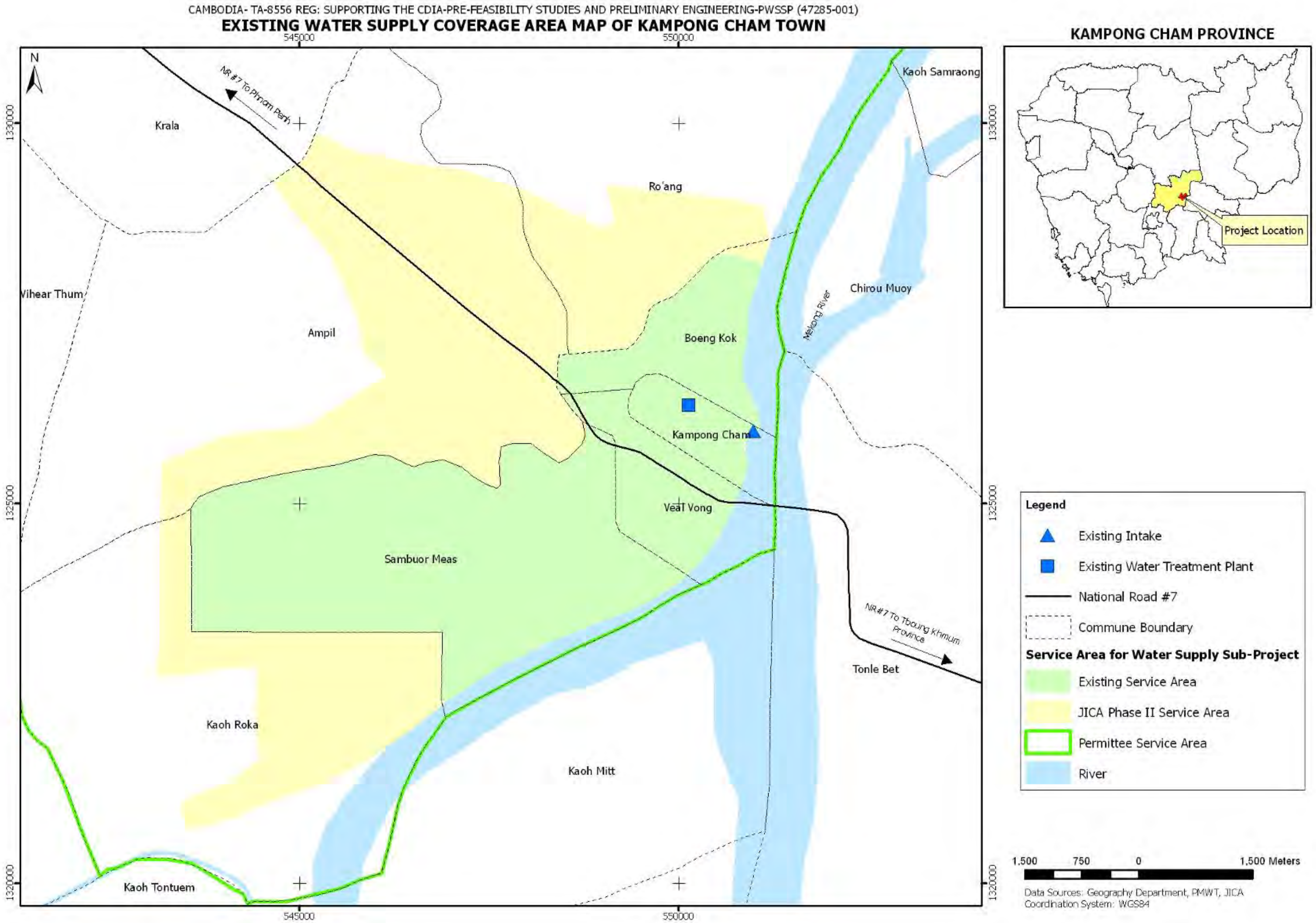
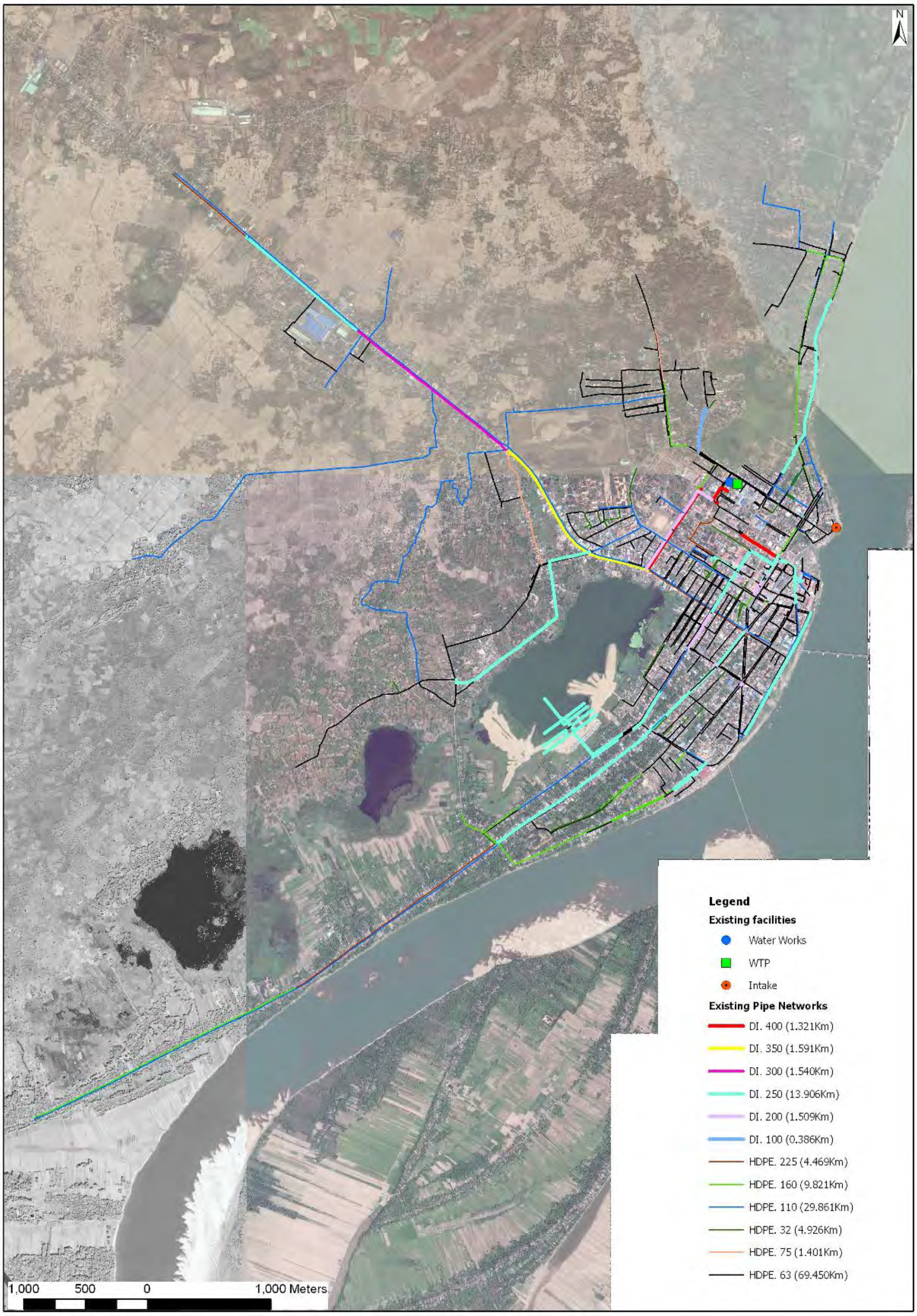


Figure 6. Kampong Cham Existing WTP and Network location map



Source: Kampong Cham waterworks, August 2016

2.4 Commercial/tourism facilities

16. Based on data obtained from Kampong Cham Waterworks, the number of commercial facilities and their monthly water consumption is summarized in the following table.

Table 6. Water consumption of commercial/tourism facilities in Kampong Cham

Facility	Unit	2012	2013	2014	2015
Number of Commercial/tourism facilities	Base	502	746	954	2,357
Average daily water consumption	m3/d	1,093	1,638	2,265	2,729

Source: Kampong Cham Waterworks, June 2016

2.5 Public buildings

17. There are many public facilities/buildings including school, hospital, pagoda/church, and governmental offices present in Kampong Cham. The public buildings served by the town water supply are summarized in the next table.

Table 7. Water consumption of public buildings in Kampong Cham

Building	Unit	2012	2013	2014	2015
Number of public buildings	Base	64	66	79	124
Average daily water consumption	m3/d	584	683	817	792

Source: Kampong Cham Waterworks, June 2016

2.6 Industries and Handicraft

18. Based on the data collection from the Department of Industrial and Handicraft (DIH) of Kampong Cham province, there are a few small industries and handicrafts[3] settled in Kampong Cham (K. Cham) and Kampong Siem (K. Siem) Districts which consume water for their production, as summarized in Table 8. However, at present none of them are using town water supply, according to the Kampong Cham Waterworks, but are using their own water supply facilities.

Table 8. Industries and handicraft in K.Cham and K.Siem District

No	Factory/ handicraft	Location			Production per Year		Employee	
		Village	Commune	District	Unit	Amount	Female	Male
1	Rice Mill	Sra Lau	Ampil	K. Siem	Ton	700	1	5
2	Rice Mill	Andongchros	Ampil	K. Siem	Ton	500	1	8
3	Rice Mill	Trapaing Chrey	Kralar	K. Siem	Ton	300	1	3
4	Rice Mill	Cheung Kork	Kralar	K. Siem	Ton	150	1	2
5	Rice Mill	Tuol Lorng	Kralar	K. Siem	Ton	1,200	1	5
6	Rice Mill	Andongchros	Ampil	K. Siem	Ton	350	1	2
7	Rice Mill	Khelchey	Ousvay	K. Siem	Ton	1,500	2	4
8	Brick kiln	Tamang	Kohrokar	K. Siem	Piece	700,000	2	3
9	Brick kiln	Thmor Koul	Ro Ang	K. Siem	Piece	1,000,000	2	3
10	Pure Water	Vealsbov	Ousvay	K. Siem	Dozen	20,000	3	5
11	Pure Water	Ampiller	Ampil	K. Siem	Dozen	15,000	3	4

[3] The term “handicrafts” as used by the GoC does not relate to tourist souveneirs etc. It refers to smaller family type businesses – ice making, fish sauce manufacturing, meat processing etc.

No	Factory/ handicraft	Location			Production per Year		Employee	
		Village	Commune	District	Unit	Amount	Female	Male
12	Pure Water	Andongchros	Ampil	K. Siem	Dozen	20,000	2	5
13	Pure Water	Ampiller	Ampil	K. Siem	Dozen	10,000	1	3
14	Pure Water	Ampiller	Ampil	K. Siem	Dozen	10,000	1	3
15	Pure Water	CheungKork	Ampil	K. Siem	Dozen	20,000	1	3
16	Brick kiln	Neang Konghing	Sambur Meas	K. Cham	Piece	12,000,000	5	10
17	Brick kiln	Lor Et	Boeung Kok	K. Cham	Piece	1,000,000	3	8
18	Brick kiln	Boeung Kok2	Boeung Kok	K. Cham	Piece	7,000,000	5	8
19	Brick kiln	Boeung Kok2	Boeung Kok	K. Cham	Piece	7,000,000	3	6
20	Brick kiln	Lor Et	Boeung Kok	K. Cham	Piece	7,000,000	3	5
21	Pure Water	Boeung Kok2	Boeung Kok	K. Cham	Dozen	20,000	5	9
22	Pure Water	Boeung Snay	Vealvong	K. Cham	Dozen	20,000	3	6
23	Pure Water	Boeung Snay	Vealvong	K. Cham	Dozen	10,000	3	5
24	Pure Water	Village 6	Vealvong	K. Cham	Dozen	20,000	3	5
25	Pure Water	Boeung Kok2	Boeung Kok	K. Cham	Dozen	15,000	4	7
26	Pure Water	Village 14	K. Cham	K. Cham	Dozen	27,000	3	5
27	Chili Sauce	Village 6	Vealvong	K. Cham	Dozen	500	1	2
28	Chili Sauce	Village 1	Vealvong	K. Cham	Dozen	500	1	2
29	Ceramic Tiles	Boeung Snay	Sambur Meas	K. Cham	Piece	1,800	1	2
30	Ceramic Tiles	Village 5	Vealvong	K. Cham	Piece	2,500	1	2
31	Ceramic Tiles	Prek Chan	Sambur Meas	K. Cham	Piece	2,000	1	2

Source: Data obtained from DIH of Kampong Cham, August 2016

2.7 Water Source

19. The main water source of Kampong Cham waterworks is Mekong River. In the past, groundwater was also used.

20. The Mekong River flows through Kampong Cham and continues to the south. The catchment area of the Mekong River in Cambodia is 154,895km2, including the main stream basin of 61,337km2 and the Tonle Sap River basin of 79,310km2. The maximum and minimum monthly average flow is 34,400m3/s and 2,200m3/s respectively, recorded from 1990 to 2011, and is adequate to supply raw water to the existing and the proposed WTP in an amount of 0.36m3/s and 0.53m3/s in 2025 and 2040, respectively.

3 Development Plan

3.1 Urban Plan & Land Use

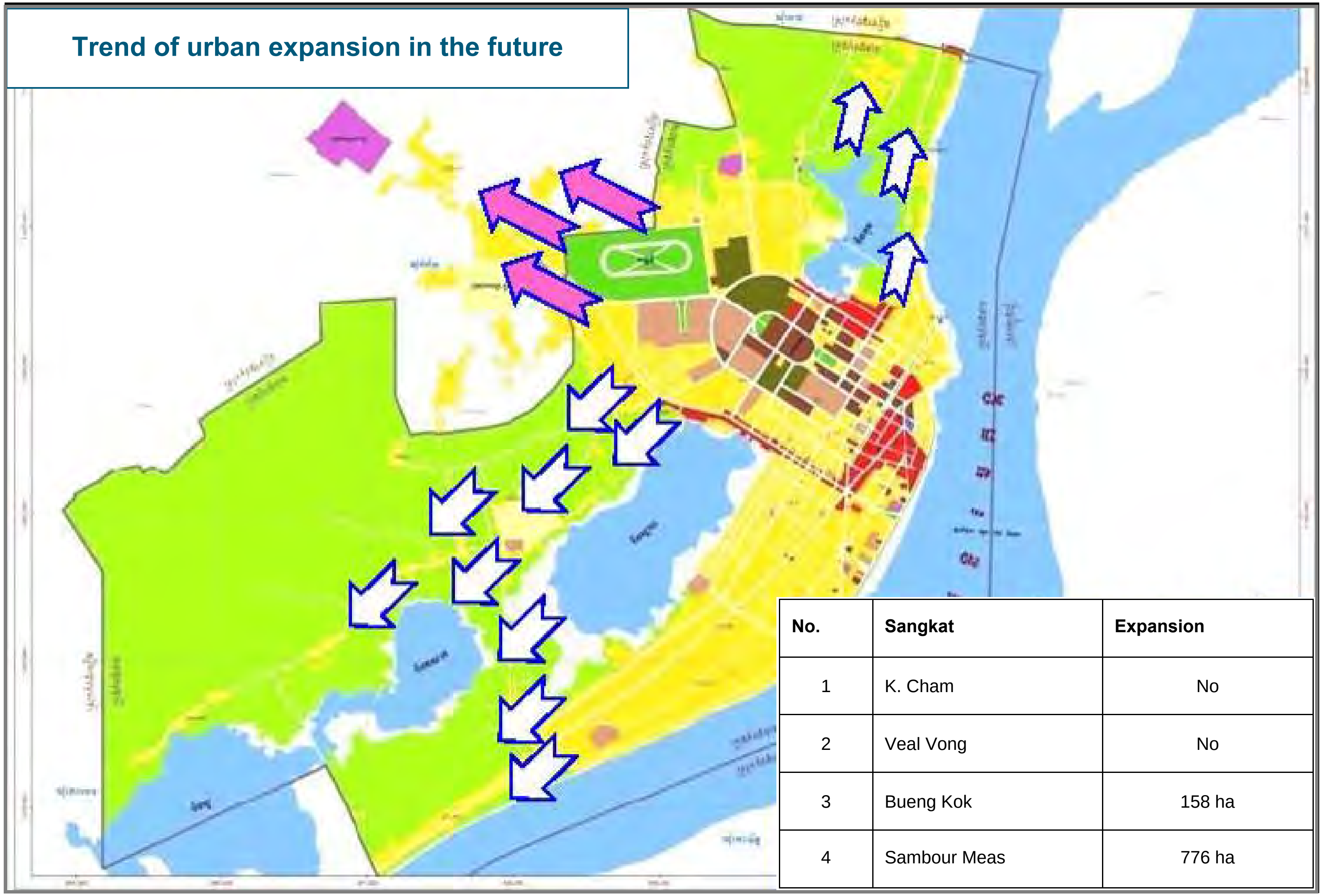
21. Kampong Cham consists of four (4) Sangkats: Kampong Cham, Veal Vong, Sambour Meas and Beung Kok as shown in Figure 1.

22. In the Development Vision 2020 prepared by the Master Planning Team of Kampong Cham Municipality, March 2009, the objective for good governance, green, and healthy development is clearly indicated. The specific goals and objectives which were also supported by CDIA include:

- Strengthening the capacity of public servants and administrative services;
- Setting up socially acceptable solutions for informal settlements;
- Providing sufficient technical infrastructure for tourists (water, waste management);
- Improve living conditions of local people and public sanitation;
- Setting up and maintain appropriate supply and service systems (drinking water, wastewater treatment, drainage, solid waste management);
- Reconstructing the existing water treatment plant to its full capacity;
- Reserving a suitable place for a new water treatment plant on the east bank of the town;
- Setting up a joint network for better coordination of the infrastructure systems;
- Develop Kampong Cham Municipality as a Green and Healthy Town and promote good environmental conditions in towns and villages, and
- Establishing suitable locations for temporary waste disposal sites (storage) for all markets; and Keeping and improving the existing waste disposal site and establish a waste utilization plant.

23. According to the Master Plan on Land Use of Kampong Cham prepared by the National Land Management Urban Planning, March 2009, the planned development of Kampong Cham is expansion of the urban areas in the southwest, the northeast, and the northwest directions, as shown in Figure 6. The land use planning map for in 2030 is shown in Figure 8. Following the Master Plan, this proposed water supply service area expansion under this subproject follows the direction of the future development plan.

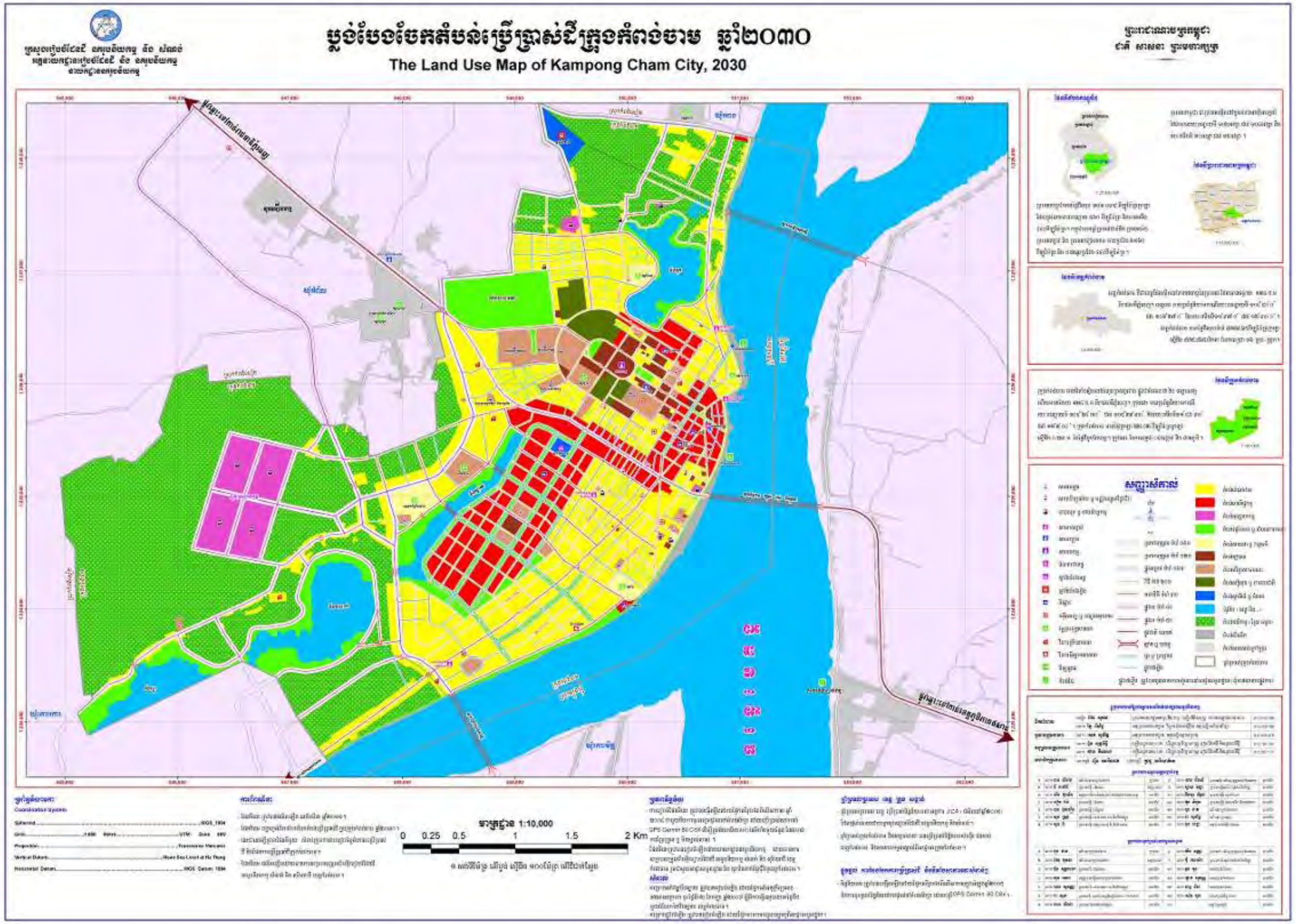
Figure 7. Expansion Plan of Kampong Cham



No.	Sangkat	Expansion
1	K. Cham	No
2	Veal Vong	No
3	Bueng Kok	158 ha
4	Sambour Meas	776 ha

Source : Final Report : Urban Environmental Infrastructure Improvement Project- Kampong Cham, Cambodia, CDIA, 2010

Figure 8. Urban Land Use in Kampong Cham, 2030



Source: Royal Government of Cambodia, Municipality. Technical Report on the Land Use Plan

3.2 Coverage Area

24. Based on the results of field investigation and consultations with Kampong Waterworks, and in accordance with the future development plan, the coverage area can be divided into three target periods: year 2015 (Existing system), year 2019 (JICA Phase II system), and year 2040 (this CDIA/ADB subproject with 20-year design service period), as shown in Figure 9. The blue area (2,014 ha) shows the present coverage area served by the existing system, the red area (2,297 ha) is the coverage area served by JICA Phase II, and the yellow area is the planned expanded coverage area (3,489 ha) under this subproject. The coverage area by each Sangkat/commune is shown in the following table.

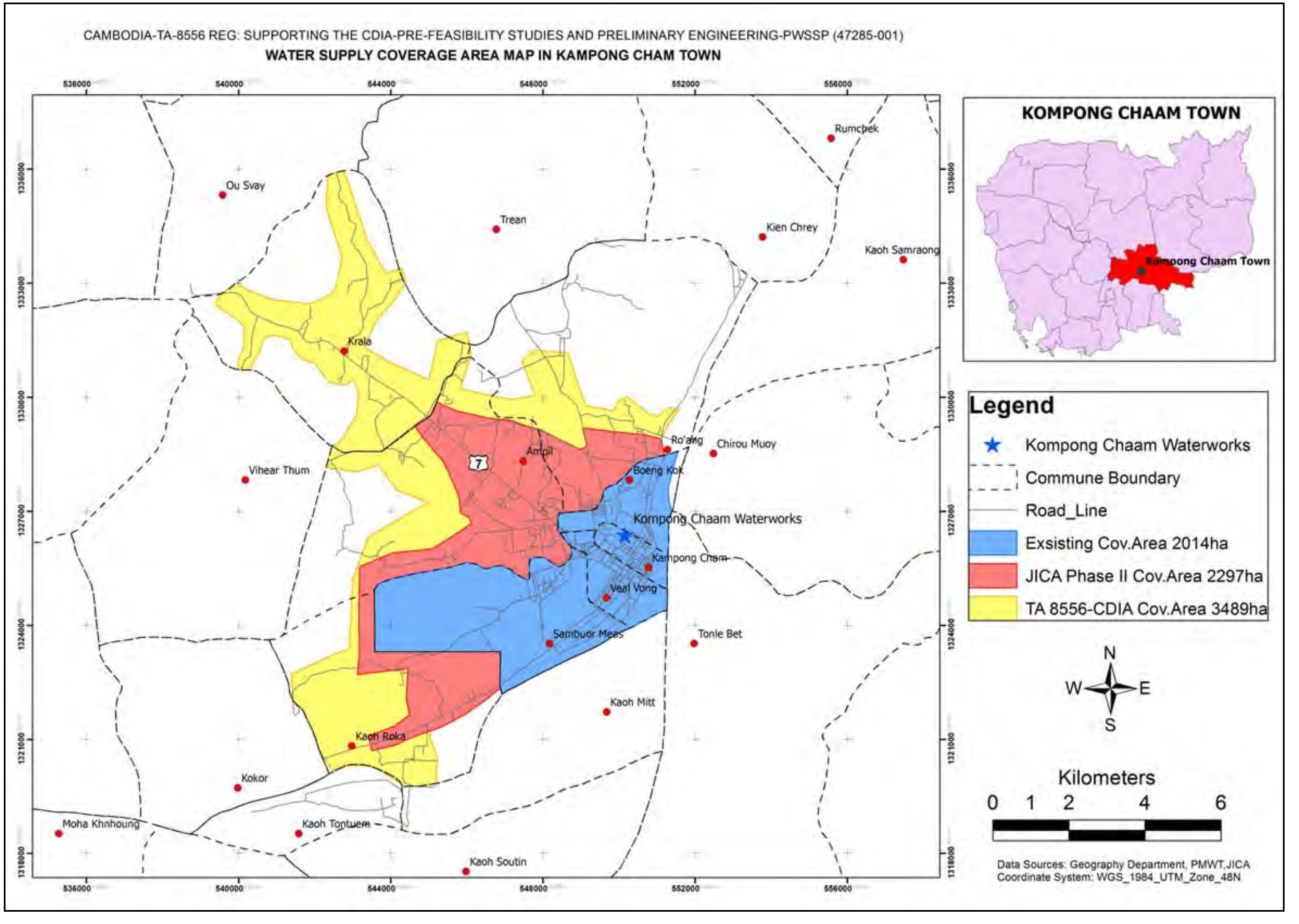
Table 9. Coverage area by Sanglat/commune

No.	Commune Name	Total Commune Area (ha)	Service Area							
			TA 8556-CDIA		JICA Phase II		Existing		Total	
			Coverage area		Coverage area		Coverage area		Coverage area	
			(ha)	(%)	(ha)	(%)	(ha)	(%)	(ha)	(%)
Kg. Cham	Boeng Kok	385	-	-	12	3	277	72	289	75
	Kampong Cham	162	-	-	-	-	128	79	128	79
	Sambuor Meas	1,431	-	-	1	0	1,359	95	1,361	95
	Veal Vong	314	-	-	0	0	247	78	247	79
	Sub-Total:	***2,293***	**-**	**-**	***13***	***3***	***2,012***	***324***	***2,025***	***328***
Kg. Siem	Koh Roka	1,863	819	44	604	32	0	0	1,424	76
	Krala	2,890	1,454	50	1	0	-	-	1,455	50
	Ro Ang	3,300	518	16	404	12	0	0	922	28
	Ampil	2,746	697	25	1,277	46	1	0	1,974	72
	Sub-Total:	***10,798***	***3,489***	***135***	***2,286***	***91***	***1***	***0***	***5,776***	***227***
	Total	**13,091**	**3,489**	**135**	**2,299**	**94**	**2,012**	**325**	**7,800**	**554**

The proposed service expansion area is within the approved service boundary signed by the DIH director[4].

[4] as agreed by the Kampong Cham Provincial Governor.

Figure 9. Coverage area of Kampong Cham Water Supply System



3.3 Population

25. Recently, Kampong Cham has seen many development activities such as urban/residential development, tourism and commercial development, and other development activities. The population has increased, with individual sangkhat or commune growth rates 1-3% per annum as summarized in Table 10.

26. For this project the average growth rate over 5 years, shown in the last column of Table 10. has been used to calculate the future population projection for water demand purposes.

27. This subproject intends to serve the population in the proposed extended coverage area up to 85-100% by 2025 and 95-100% by 2040, with further communities nearby the town boundary to be included for the Year 2025. The present population in each Sangkat and commune of Kampong Cham town and Kampong Siem district is described in Table 11. The population projection is calculated with the formula below, and is commonly used for population projections in SEA and elsewhere, and the results of the projection up to 2040 are provided in Table 11.

Formula: $P_t = P_0 * (1 + e)^t$

Where P_t - *Future population at projection time*

P_o - *Present population*

e - Population grow rate (%)

t - The period of projection

Table 10. Population growth rate

Town/ District	Sangkat/ Commune	Population by year						Average Grow rate	Adopted Grow Rate
		2009	2010	2011	2012	2013	2014	(%)	(%)
Kg. Cham	Boeng Kok	7,981	7,794	7,946	8,438	8,962	8,720	3.00	2.5
	Kampong Cham	6,481	6,160	6,495	6,153	6,703	5,367	1.04	1.0
	Sambuor Meas	13,326	14,006	14,327	14,323	14,671	14,290	2.45	2.0
	Veal Vong	13,612	13,508	13,757	13,991	14,192	11,656	1.05	1.0
	Sub-Total:	***41,400***	***41,468***	***42,525***	***42,905***	***44,528***	***40,033***		
Kg. Siem	Koh Roka	7,360	7,535	8,402	9,784	8,713	8,933	4.85	3.0
	Krala	9,801	10,177	10,565	10,998	11,045	10,763	3.04	2.5
	Ro Ang	11,538	10,948	11,115	12,694	12,411	11,981	2.10	2.0
	Ampil	15,090	15,562	15,989	17,253	17,447	17,520	3.10	2.0
	Sub-Total:	***28,699***	***28,660***	***30,082***	***33,476***	***32,169***	***31,677***		
	Total	**70,099**	**70,128**	**72,607**	**76,381**	**76,697**	**71,710**		

Source: Commune Database (CDB), Ministry of Planning

Table 11. Population Projection by Sangkat and Commune in Kampong Cham and Kampong Siem

Town/ District	Sangkat/Commune	Baseline 2014	Growth rate (%)[i]	Population Projection (person)				
				2020	2025	2030	2035	2040
Kg. Cham	Boeng Kok	8,720	2.50	10,113	11,441	12,945	14,646	16,571
	Kampong Cham	5,367	1.00	5,697	5,988	6,293	6,614	6,952
	Sambuor Meas	14,290	2.00	16,093	17,768	19,617	21,659	23,913
	Veal Vong	11,656	1.00	12,373	13,004	13,668	14,365	15,098
	Sub-Total:	***40,033***		***44,276***	***48,201***	***52,523***	***57,284***	***62,533***
Kg. Siem	Koh Roka	8,933	3.00	10,666	12,365	14,335	16,618	19,265
	Krala	10,763	2.50	12,482	14,122	15,978	18,077	20,453
	Ro Ang	11,981	2.00	13,493	14,897	16,447	18,159	20,049
	Ampil	17,520	2.00	19,730	21,784	24,051	26,554	29,318
	Sub-Total:	***49,197***		***56,371***	***63,168***	***70,811***	***79,409***	***89,085***
	Total	**89,230**		**100,647**	**111,369**	**123,334**	**136,693**	**151,618**

Note: [i] Adopted growth rate for future projection

4 Served Population and Other Facilities

4.1 Served Population

28. Based on the proposed service area of Kampong Cham Waterworks, the served population for different target years is described in Table 12. The table indicates that presently the utility covers 4 Sangkats with 79% of the population served, and 2% of the population of two communes (Ampil and Ro-Ang) of Kampong Siem District are also covered.

29. According to the national policy on urban water supply, the RGC will provide clean and safe water to all Cambodian people. Due to the geographical spread of the community and the settlement pattern, it is assumed that 85-100% of people will be served by 2025 and 90-100% of people will be served by 2040 as shown.

30. The remaining people are proposed to be served by other source (rural water supply system such as groundwater or rainwater collection).

Table 12. Served population in 2025 and 2040

City/	Sangkat/	Population 2015		Served Population, 2015				Total Population 2025	Served Population in 2025		Total Population 2040	Served Population in 2040	
District	Commune	Family	Person	Family	Family size	Person	%	Persons	%	Persons	Persons	%	Persons
Kg. Cham	Kampong Cham	1,235	6,223	1,206	5.04	6,077	97.65	11,441	100	11,441	16,571	100	16,571
	Veal Vong	2,878	10,735	2,463	3.73	9,187	85.58	5,988	100	5,988	6,952	100	6,952
	Sambour Meas	2,831	13,809	1,605	4.88	7,829	56.69	17,768	90	15,991	23,913	90	21,522
	Beung Kok	1,879	8,538	1,440	4.54	6,543	76.63	13,004	100	13,004	15,098	100	15,098
	Sub-total:	***8,823***	***39,305***	***6,714***		***29,636***	***79.14***	***48,201***		***46,424***	***62,533***		***60,142***
Kg. Siem	Ampil	4,007	17,392	8	4.34	35	0.2	12,365	85	10,511	19,265	90	17,338
	Kosh Rokar	2,430	8,765	-	3.61	-	0	14,122	85	12,004	20,453	90	18,408
	Krola	2,421	10,718	-	4.43	-	0	14,897	85	12,662	20,049	90	18,044
	Ro Ang	2,529	9,918	47	3.92	184	1.85	21,784	85	18,516	29,318	90	26,386
	Sub-total:	***11,387***	***46,793***	***55***		***219***		***63,168***		***53,693***	***89,085***		***80,177***
	Total:	**20,210**	**86,098**	**6,769**		**29,855**		**159,571**		**100,117**	**151,618**		**140,318**

4.2 Served Commercial/ Tourism Facilities

31. Kampong Cham has a high potential for further development in tourism and other sectors. The town still has open land for the future development of tourism and other sector service facilities.

32. Based on the water bills of the Kampong Cham waterworks in 2015, the commercial facilities are 2,357 in total. Assuming that the tourism facilities are to increase by 1.5% per year, the projection number of the commercial in future is described in the following table.

Table 13. Served commercial facilities in the future

Facility	2015(i)	2020	2025	2030	2035	2040
Hotel and Guesthouse Restaurant Supermarket/store	2,357	2,539	2,735	2,947	3,175	3,420
Total :	**2,357**	**2,539**	**2,735**	**2,947**	**3,175**	**3,420**

Note: (i) *Data obtained from Kampong Cham Waterworks, April 2016*

4.3 Served Public Buildings

33. As the provincial town of Battambang province, there are many public buildings including government offices, hospitals, schools, university, and religious buildings. The average population growth rate is 2% for the period 2015-2020 and 2.5% for 2026-2040, so the number of schools will increase in proportion with this population growth rate. Public buildings are expected to increase by approximately 1% per year, as shown in Table 14.

Table 14. Estimation of served public buildings in the future

Building	2015(i)	2020	2025	2030	2035	2040
School/Pagoda/church/ government building	124	134	144	155	167	180
Total :	**124**	**134**	**144**	**155**	**167**	**180**

Note: (i) *Data obtained from Kampong Cham Waterworks, April 2016*

5 Water Demand

5.1 Domestic Water Demand

34. According to available consumption data[5] shown in Table 15. , the average daily water consumption per capita decreased from 142 lpcd to 130 lpcd due to network expansion toward the outskirts where less water was consumed. The average daily water consumption per capita is 139 liters, and as economic growth continues it is projected to continue to increase to 145 and 160 liters for Year 2025 and 2040 respectively. Domestic water demand by the target Year 2025 and 2040 is described below.

Table 15. Daily water consumption

Description	2012	2013	2014	2015	Average
Domestic water consumption per year (m^3)	1,334,276	1,445,973	1,574,605	1,480,579	
Total served population (person)	25,713	28,395	30,132	31,190	
Water consumption lpcd	142	140	143	130	139

Note: lpcd – litres per capita per day

Source: Kampong Cham Waterworks, June 2016

Table 16. Domestic water demand in 2025 and 2040

City/ District	Sangkat/ commune	2025			2040		
		Served Population in 2025	Daily water consumption lpcd	Water Demand m3/d	Served Population in 2040	Daily water consumption lpcd	Water Demand m3/d
Kg. Cham	Boeng Kok	11,441	145	1,659	16,571	160	2,651
	Kampong Cham	5,988	145	868	6,952	160	1,112
	Sambuor Meas	15,991	145	2,319	21,522	160	3,443
	Veal Vong	13,004	145	1,886	15,098	160	2,416
	Sub-total :	***46,424***		***6,732***	***60,142***		***9,623***
Kg. Siem	Koh Roka	10,511	145	1,524	17,338	160	2,774
	Krala	12,004	145	1,741	18,408	160	2,945
	Ro Ang	12,662	145	1,836	18,044	160	2,887
	Ampil	18,516	145	2,685	26,386	160	4,222
	Sub-total :	***53,693***		***7,785***	***80,177***		***12,828***
	Total	***100,117***		***14,517***	***140,318***		***22,451***

Note: lpcpd – litres per capita per day

5.2 Commercial, Public Building, and Industrial Water Demand

35. Water consumption data from Kampong Cham waterworks from 2012 to 2015 indicate that the annual average water demand of commercial properties is 41.25% of annual domestic water demand, and the annual average water demand of public buildings is 17.39%. The water demand of industry is minimal as shown in Table 17. However, as shown in Table 8. there are

[5] Data from Battambang Waterworks 2012-2015

currently 31 small businesses classified as industrial or handicraft settled within the proposed service area, and as expected by MIH/DIH most of these will use the town water supply once they can be connected. The industrial water demand is suggested by MIH/DIH to be up to 15% of the total domestic water demand[6].

36. Following the trend of urban development and as suggested by MIH, the adopted commercial water demand is 40% of domestic demand, public building water demand is 17% of domestic water demand, and industrial water demand is 15% of domestic water demand for year 2025. For the year 2040, we assume the water demand of commercial, public buildings, and industries to be 35%, 15%, and 15% of the domestic water demand. The water demand by customer category for 2025 and 2040 are shown in the two tables below.

Table 17. Commercial and public building water demand by year

No.	Category	Water consumption, m^3				Comparison with domestic
		2012	2013	2014	2015	Average in %
1	Domestic	1,334,276	1,445,973	1,574,605	1,480,579	
2	Commercial	399,010	597,759	826,708	996,045	41.25
3	Public building	213,275	249,316	298,161	289,049	17.39
4	Industries	-	-	-		0
	Total:	1,946,561	2,293,048	2,699,474	2,765,673	

Source: Kampong Cham Waterworks, June 2016

Table 18. Water demand by customers for 2025 and2040

No.	Category	2025			2040	
		Adopted criteria (1)		Water Demand (m3/d)	Adopted criteria (2)	Water Demand (m3/d)
1	Domestic	145 lpcpd	daily consumption	14,517	160 lpcpd	22,451
2	Commercial	40%	of domestic	5,807	35%	7,858
3	Public buildings	17%	of domestic	2,468	15%	3,368
4	Industries	15%	of domestic	2,178	15%	3,368

Note: (1) From billing records of Kampong Cham waterworks 2012-2015

(2) It was assumed that in 2040 the commercial and public building consumption will be lower in percentage of the domestic daily consumption than in 2025, as the expansion is toward the suburban/rural areas with less commercial activities and public services building.

Table 19. Water demand by customers for 2040

No.	Category	Adopted criteria (2)		Water Demand (m3/d)
1	Domestic	160 lpcpd	daily consumption	22,451
2	Commercial	35%	of domestic	7,858
3	Public buildings	15%	of domestic	3,368
4	Industries	15%	of domestic	3,368

5.3 Non-Revenue Water/Water Loss

[6] Consultant advised to use this percentage by Secretary of State, MIH at mid-term workshop

37. Based on four years recorded data the trend of water loss, a decrease from 20.47% in 2012 to 10.96% in 2015 can be observed (see Table 20.). A larger amount of water was lost in 2012 and 2013 due to older leaking pipeline replaced by JICA Phase II project (2014-2016), reducing the overall NRW figure. For design purposes a NRW figure of 15% is adopted.

Table 20. Water loss in the existing system

No.	Category	2012	2013	2014	2015	Average
1	Total production per year, m^3	2,205,586	2,567,406	2,955,318	3,001,096	
2	Total served population	25,713	28,395	30,132	31,190	
3	Served water/year (total collection from water bills), m^3	1,946,561	2,293,048	2,699,474	2,765,673	
4	Water lost (%)	11.74	10.69	8.66	7.84	9.73

Source: Data obtained from Kampong Cham Waterworks, June 2016

5.4 Seasonal Peak Factor

38. According to the data recorded by Kampong Cham Waterworks from 2009-2011, the actual daily (or seasonal) peak factor from flow records (ratio of daily maximum to daily average) was 1.18. This data represents the seasonal variations between the dry (February-May) and wet season (June-August) period. Therefore 1.2 is adopted as the daily peak factor used in calculating the required WTP capacity.

Table 21. Daily peak factor in Kampong Cham

			2009	2010	2011	2012	2013	2014	2015	Average
Daily average		m^3/day	4,148	4,863	5,131	6,042	7,033	8,096	8,202	
Daily Maximum	amount	m^3/day	4,608	6,960	5,649	7,428	7,896	8,212	10,129	
	day		7-Aug	16-May	15-May	26-Jun	14-May	9-May	16-Feb	
Peak Factor			1.11	1.43	1.10	1.23	1.12	1.01	1.23	1.18

Source: Obtained from Kampong Cham Waterworks, September 2016

5.5 Summary of Water Demand

39. The total water demand by target year 2025 and 2040 is summarized in Table 22. The total water demand including daily peak factor has been used in determining the WTP capacity and pump capacity at each station.

Table 22. Summary of total water demand and water to be produced in 2025 and2040

No.	Description	Unit	2025	2040
1	Total Population in coverage area	person	159,571	151,618
2	Served population in coverage area	person	100,117	140,318
3	Domestic	m3/d	14,517	22,451
4	Commercial	m3/d	5,807	7,858
5	Public building	m3/d	2,468	3,368
6	Industrial	m3/d	2,178	3,368
7	Total water demand	**m3/d**	24,969	**37,044**
8	NRW (15%)	m3/d	3,745	**5,557**
9	***Total water to be distributed***	**m3/d**	***28,715***	**42,601**
10	Existing WTP	m3/d	8,000	8,000
11	Existing WTP (JICA phase II, 2016)	m3/d	11,500	11,500
12	**Total water to be produced**	**m3/d**	**9,215**	***23,101***
13	Daily peak		1.2	1.2
14	**Total water to be produced with peak factor**	**m3/d**	**11,058**	**27,721**

6 Design Criteria

6.1 Phasing of the Facilities

40. The water supply subproject is designed to provide urban water supply for a 20 year period, i.e. designed for capacity to serve 2040 demand. However, to ensure an economical aspect to the project, the implementation of physical construction is proposed to be phased, initially to meet the target year 2025 as per the government's development goal on urban water supply and economic vision, and later to meet 2040 demands. The proposed two phases are shown in Table 23.

Table 23. Components for proposed sub-project - Phases 1 and 2

	Target year	Component	Civil Works to be implemented	Equipment to be installed
Phase I	2025	Intake	For 2040 target year	For 2025: 1 duty + 1 standby pumps
		Raw water transmission main	4 km	
		WTP Phase I	11,600 m3/d	2 duty + 1 standby booster pumps
		Distribution network	Primary mains: 52 km And28 km to connect 2nd and 3rd row houses Service areas: > 3,489 ha	
		Pump along network		2 duty + 1 standby booster pumps
Phase II	2040	Intake	Already provided in 2025	For 2040: 1 additional pumps
		Raw water transmission main	4 Km in parallel of the existing one	
		WTP Phase II	Extension 17,000 m3/d	Additional 2 duty pumps
		Distribution network	40 km of primary and secondary mains	
		Pump along network		Additional 1 pump

41. The purpose of the PWSSP is to assist the Cambodian Government in meeting national targets on improvement and expansion of urban water supply coverage (for 100% of population) and align with proposed government targets for urban sanitation. The project will aim at filling the gap between the existing capacity, including that from the recent JICA phase II project, to serve the current and proposed extended service areas to 2040.

6.2 WTP & Intake Capacity

42. The table below summarizes the calculation for the proposed WTP & Intake capacity. The existing WTP (Existing and JICA Phase II) capacity is 19,500m3/d. The proposal is to construct the required WTP for 2040 in two stages to avoid having a larger WTP operating well under capacity for the initial decade[7]. The JICA phase II project reported that from 2019 a new WTP would be needed, due to the water demand from the original service area surpassing the current 19,500m3/d capacity.

Table 24. WTP and Intake capacity in 2025 and 2040

No.	Description	Unit	2025	2040
1	Total Population in coverage area	person	159,571	151,618
2	Served population in coverage area	person	100,117	140,318
3	Domestic	m3/d	14,517	22,451
4	Commercial	m3/d	5,807	7,858
5	Public building	m3/d	2,468	3,368
6	Industrial	m3/d	2,178	3,368
7	**Total water demand**	**m3/d**	**24,969**	**37,044**
8	NRW (15%)	m3/d	3,745	5,557
9	Total water to be distributed	m3/d	28,715	42,601
10	Existing WTP	m3/d	8,000	8,000
11	Existing WTP (JICA phase II, 2016)	m3/d	11,500	11,500
12	**Base water demand for CDIA project**	**m3/d**	**9,215**	**23,101**
13	Daily peak		1.20	1.20
14	CDIA Ultimate WTP capacity (2025+2040)	m3/d	**11,058**	27,721
15	CDIA WTP capacity for 2025	m3/d		11,600
16	Future water required for 2040 WTP	m3/d		**16,121**
17	WTP process losses (5%)	m3/d	553	806
18	Total WTP production requirement/intake extraction	m3/d	11,610	16,927
19	**Design capacity of new WTP**	**m3/d**	**11,600**	**17,000**

6.3 Intake location & design criteria

6.3.1 Intake location

43. There were four possible intake sites suggested by the Kampong Cham Waterworks: two locations upstream of the JICA II intake, one location downstream of the JICA II intake, and one close to the proposed new WTP site, as shown in Figure 9. Advantages and disadvantages of each intake site option are presented in Table 25.

44. Potential flooding erosion may occur near the two upstream locations. Contamination from the sewerage discharge along with construction impacts of the transmission line near the riverfront cause difficulties for the option near the proposed new WTP. Following site

[7] As requested by HE Ek Son Chan.

investigation and engineering assessment, a new Mekong intake approximately 50m downstream of the existing JICA II intake (see Figure 11.) is recommended.

Table 25. Advantage and disadvantage of the intake site

	Advantage	Disadvantage
Site 1	• Less community residential settlements • Free from pollution source (both liquid and solid waste)	• 5 km upstream from new WTP • Unstable riverbank • No available land • More study on power source connection required • Higher operational cost for the pump
Site 2	• Close to the town center • Less cost for power connection	• 4.5 Km upstream of new WTP • Unstable riverbank, high potential of land slide • No available land
Site 3	• Closer to new WTP than other sites • Less cost for power connection	• Nearby the outlet of the town sewer • Easily polluted by waste discharged from the town • Unstable riverbank • No available land
Site 4	• Close to the town center and close to JICA intake • Easy connection to the power source • Easy maintenance and security	• 4 km upstream of new WTP • Stable bank compared to other sites • Public land with supported document from Governor

Figure 10. Proposed Intake locations

Figure 11. **Selected Intake location**

6.3.2 Type of Intake

45. Different types of intakes have been considered, including an intake tower and riverbank side intake. As the river on which the intake will be constructed is relatively narrow and will be used for boat traffic, the intake tower cannot be considered and the side intake type is recommended.

46. Key design considerations are the stability of the river bank, and positioning the intake pipe below the dry season water level to avoid floating debris.

47. This intake facility will deliver raw water from the Mekong about 4km along the existing road to the proposed new WTP as shown in 54

6.3.3 Intake Design criteria

48. Based on records[8] the high water level (HWL) is 15.18m and the low water level (LWL) is 0.63m above mean sea level.

49. The raw water pumping mains for 2025 and for 2040 are proposed as two separate parallel pipes to meet demands at those design years.

6.4 Water Treatment Plant (WTP)

6.4.1 WTP site location

50. The originally proposed site for the new WTP was an existing compound (28m wide and 39m long) where an elevated water storage built in 1960-1970 is currently present. It is located in Village 6, Sangkat Veal Vong, Krong Kampong Cham, as shown in Figure 12.

Figure 12. Photos of originally proposed site for new WTP

51. Based on site visits and engineering assessment the previous site was found to be too small, so a second location was proposed. From available public land, a site with total area of 1.3 ha at Ta-Neng village, Sangkat Sambour Meas, Kampong Cham was selected as the proposed

[8] “Preparatory Survey on the Project on Additional New Water Treatment Plants for Kampong Cham and Battambang Waterworks in the Kingdom of Cambodia, March 2013, JICA”

WTP site. It is located in a new development area, 3km from the Waterworks office and 1.5Km from Neang Kong Hing roundabout as shown in 54

52. The conventional treatment process (coagulation – flocculation - sedimentation - rapid sand filtration – chlorination – clear water storage), the same as the process used at the vast majority of plants in Cambodia, is deemed most appropriate and this was agreed by the MIH/DIH. The conventional treatment process is also widely utilized in Laos and Vietnam and is relatively simple to operate and maintain.

53. The processes adopted are listed below:

- Mixing basin
- Flocculation basin
- Sedimentation basin
- Rapid sand filter
- Chlorination unit
- Treated water storage/distribution basin
- Distribution pump
- Electrical facilities
- Sludge drying beds

54. The treatment process was adopted to suit the Mekong water and its quality. The conventional treatment process (coagulation, sedimentation and filtration) is appropriate in addressing high turbidity raw water during the wet season, and commonly adopted in Cambodia. Following economic considerations, the WTP will be installed with the initial capacity to serve year 2025 water demand only, and space will be reserved for additional units to meet capacity of year 2040 water demand. The figure bellow shows the proposed process.

Figure 13. Process diagram

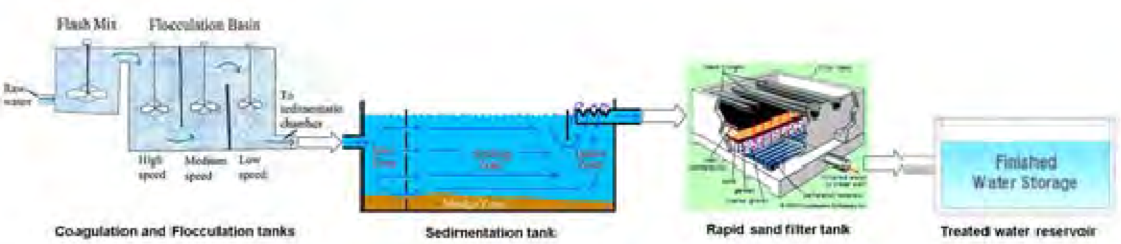
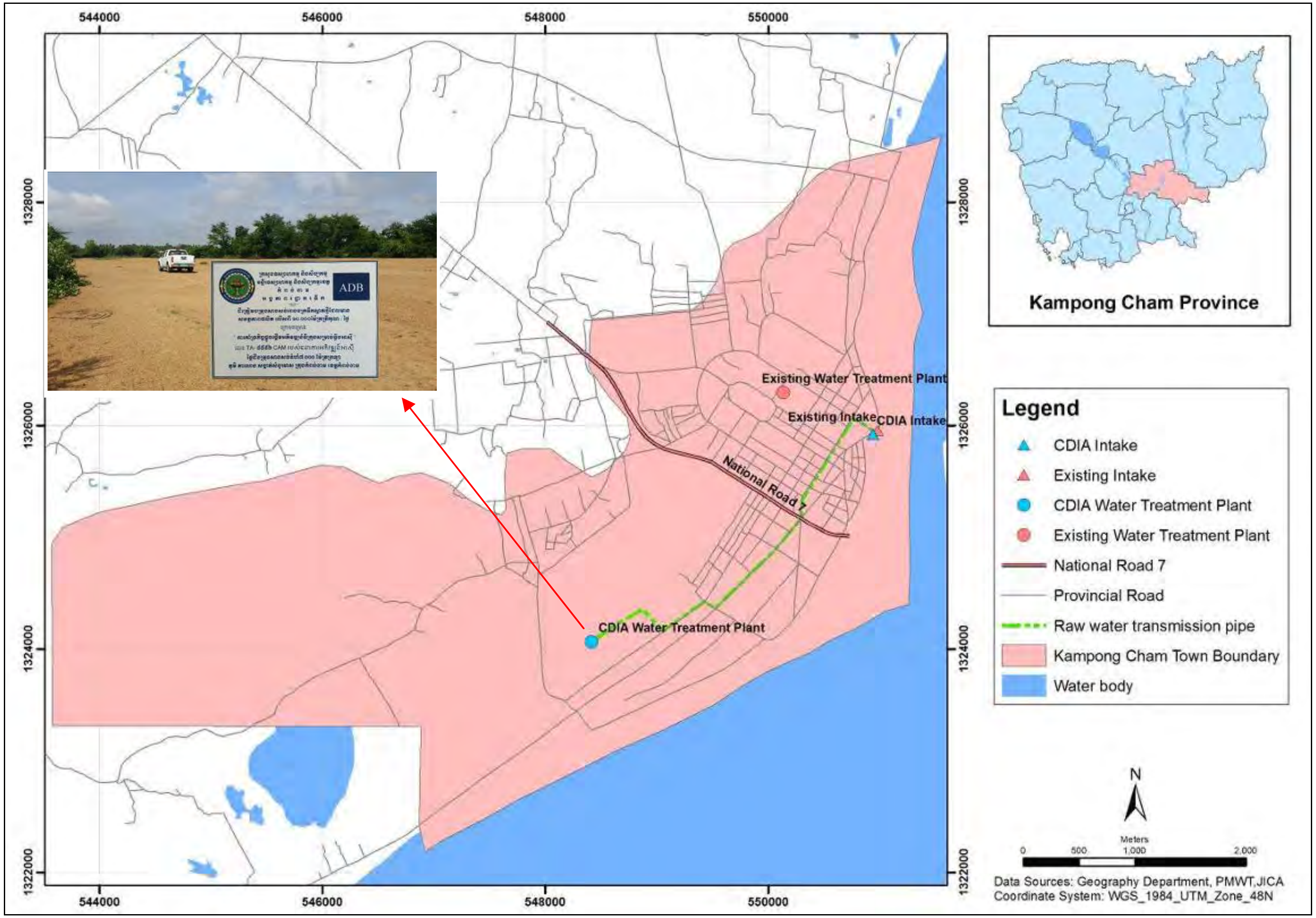


Figure 14. New WTP site location



6.4.2 Water Quality

55. Based on the National Drinking Water Standards of Cambodia, 2015, there are 27 parameters considered for urban water supply as shown in Table 26. Results from water quality monitoring of the Mekong source conducted by JICA (July 2012) and Kampong Cham Waterworks (June 2016) are provided in Table 27. The Mekong River water is considered safe and appropriate for a raw water source to the proposed new WTP.

Table 26. Drinking Water Quality Standard of Cambodia

Parameters	Unit	Allowable maximum value
Micro-organism		
E-coli	MPN/100ml	0
Chemical		
Aluminum (Al)	mg/l	0.2
Ammonia (NH3)	mg/l	1.5
Arsenic (As)	mg/l	0.05
Barium (Ba)	mg/l	0.7
Cadmium (Cd)	mg/l	0.003
Chloride (Cl-)	mg/l	250
Residue chlorine	mg/l	0.1-1.0
Chromium (Cr)	mg/l	0.05
Cupper (Cu)	mg/l	1
Fluoride (F)	mg/l	1.5
Total Hardness (CaCO3)	mg/l	300
Iron (Fe)	mg/l	0.3
Lead (Pb)	mg/l	0.01
Manganese (Mn)	mg/l	0.1
Mercury (Hg)	mg/l	0,001
Nitrate (NO3)	mg/l	50
Nitrite (NO2)	mg/l	3
Sodium (Na)	mg/l	250
Sulphate (PO4)	mg/l	250
Zinc (Zn)	mg/l	3
Physical		
Color	TCU	5
pH		6.5-8.5
Total Dissolved Solid (TDS)	mg/l	800
Conductivity	NCU	5
Taste		Acceptable
Odor		Acceptable

Source: National Drinking Water Standard 2015

Table 27. Water Quality Monitoring

N	Parameter	Unit	PWQS of Cambodia	WHO guideline values, 2011	Results 30/9/2016		Results 31/10//2016	
					Raw water	Treated water	Raw water	Treated water
1	Color	TCU	5	15	185	0.85	155	0.96
2	Turbidity	NTU	5	5	155	0.70	120	0.15
3	Residual chlorine	mg/l	0.2-0.5		-	0.60	-	0.65
4	pH		6.5-8.5	6.5-8.5	7.4	7.1	7.5	7
5	Total dissolved solids	mg/l	800	1000	85	191	84	130
6	Manganese (Mn)	mg/l	0.1	0.01			0.1	0
7	Sulfate (SO_4)	mg/l	250	250			0	0
8	Hydrogen Sulfide (H_2S)	mg/l	0.05				0	0
9	Chloride (Cl)	mg/l	250	250			-	21.2
10	Iron (Fe)	mg/l	0.3	0.3			0.003	0
11	Arsenic	µg/l	<50				0	0
12	Ammonia (NH3-N)	mg/L	1.5	1.5			0.2	0.005
13	E. Coli	MPN/100 ml	0	0			-	0
14	Total Coliform	MPN/100 ml	0	0			-	0
15	Alkalinity (as CaCo3)	mg/l			67	85	88	80
16	Conductivity	µs/cm			170	240	168	260
17	Organic Carbons	mg/l					-	0

Note: PWQS – Potable Water Quality Standards of Cambodia, MIH, Sept. 2015
Source: WQ results obtained from Kampong Cham Waterworks, Nov.2016

56. The data in Table 27. indicate that turbidity, color, iron, and E.Coli are well treated by the existing conventional treatment plant and the results meet both the National and WHO drinking water standards.

57. Based on the JICA Phase II results, toxic substances such as heavy metals and cyanide were either not reported or below the water quality limits, and the Mekong River water is considered safe for the raw water source to JICA's Phase II WTP and also for the proposed WTP.

58. Sodium hypochlorite powder is selected for disinfection as it is available locally, and safer to handle than chlorine gas.

59. Drying beds are considered from treatment of sludge, since sufficient area is available at the WTP site.

60. The proposed WTP system will distribute the water to the customers via a booster pump station.

6.4.3 WTP process design criteria

61. The design criteria for the WTP is presented in the following table:

Table 28. Design criteria for WTP preliminary design

Facility	Component
Distribution design capacity	11,600 m3/d
Receiving Well	Retention Time: T=2.8 min in dry season (T≥1.5 min)
Rapid Mixing Tank	Retention Time: T=2 min (2 <T< 5 min)
Flocculation Tank	Contact Time: 30 min
Sedimentation Tank	Horizontal-Flow Sedimentation Basin Surface Loading: Q/A=20.0 mm/min (15-30 mm/min) Mean Velocity: V=0.14 m/min (below 0.40 m/min)
Rapid Sand Filter	Filtration Speed: V=126.6 m/day (120-150 m/day) Maximum filtration speed (N-1): 10m/h Backwash: 20m/h with 5min air + 10 min air and water + 10 min water
Backwash	Backwash pumps and blowers
Chlorination	Sodium hypochorite powder Contact time: 30 min Treatment rate: 5 mg/l
Clearwater Reservoir	Retention Time: T= 3 hours
Sludge Drying Bed (Lagoon)	Sludge quantity production: to be determined
Chemical Feeding Facility	Alum ,Lime, and chlorine

6.5 Distribution network

6.5.1 Network Philosophy

62. Piped water supply shall be installed along roads in populated areas and areas where there is a potential for further housing development. The pipe distribution network is designed to supply the proposed extended service areas and also to fulfill supply to the existing service areas during the period 2019 to 2040.

63. Three options for the distribution network have been modelled and analysed:

Option 1 uses booster pumps from the clearwater reservoir, as shown in Figure 15. Treated water from the WTP will be pumped directly to the consumers without any elevated water towers or additional booster pumps along the network.

Option 2 uses booster pumps at clearwater reservoir with additional booster pumps at certain locations along the network towards the Phnom Pros area, as shown in Figure 16. This is required due to high elevation differences and a 35km distance between the WTP and the Phnom Pros distribution areas. Use of booster stations alone will minimize the land acquision required for an elevated water tank.

Option 3 uses booster pumps at the clearwater revervoir with additional booster pumps at certain locations along the network, and also provides for an elevated water tower in the Pnhom Pros area as shown in Figure 17. The water tower stores water whenever the consumption is less than normal or the residual pressure allows the water into the tanks, and provides additional storage for the downstream area.

Figure 15. Booster pump supplying directly the distribution network

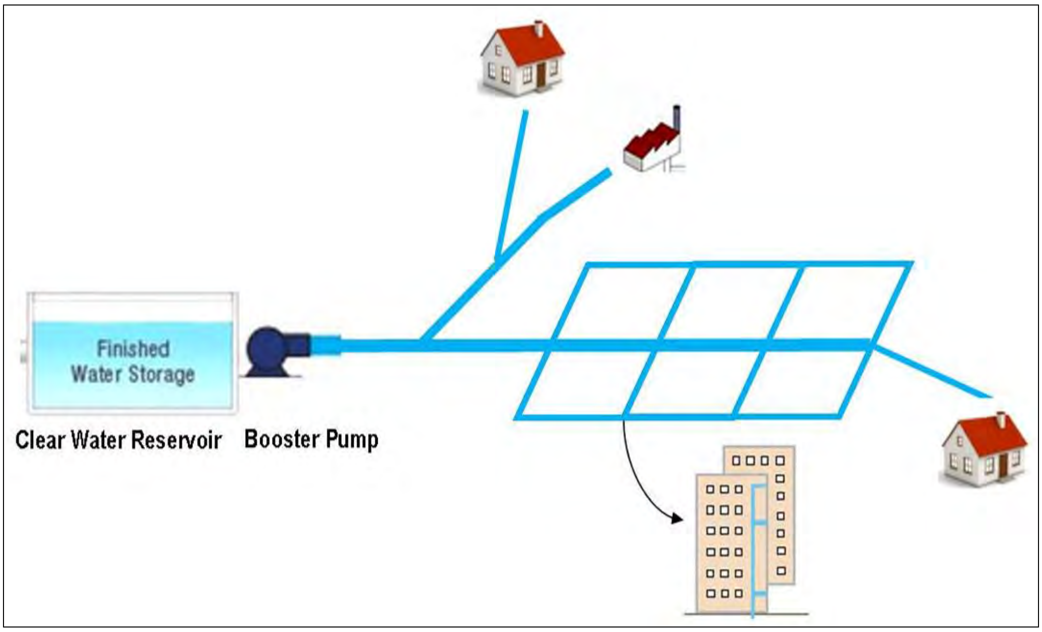


Figure 16. Two sets of booster pumps at the WTP and midway supplying the distribution network

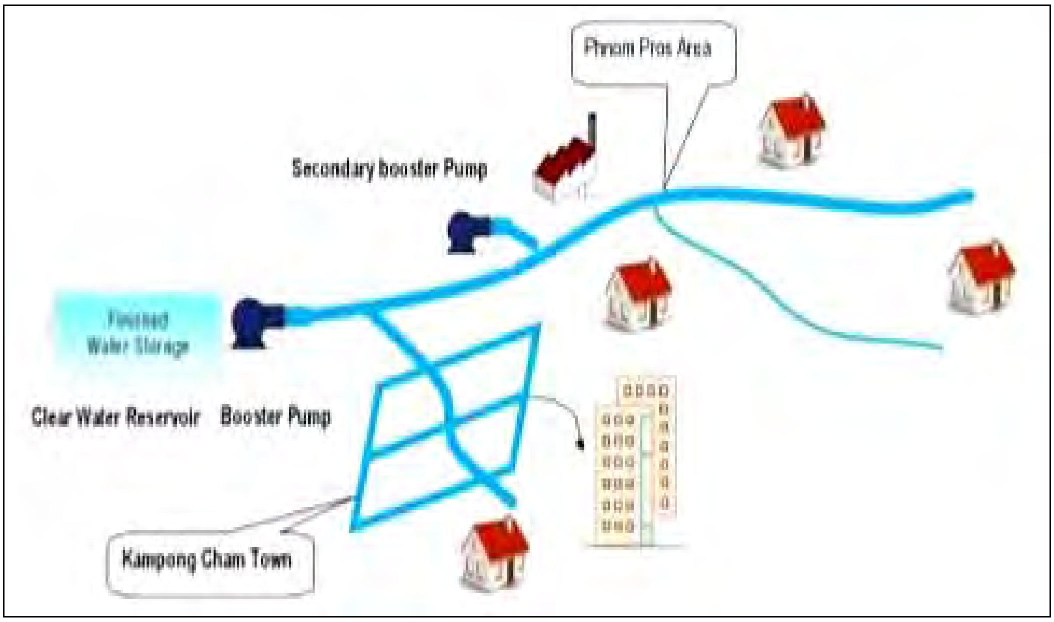
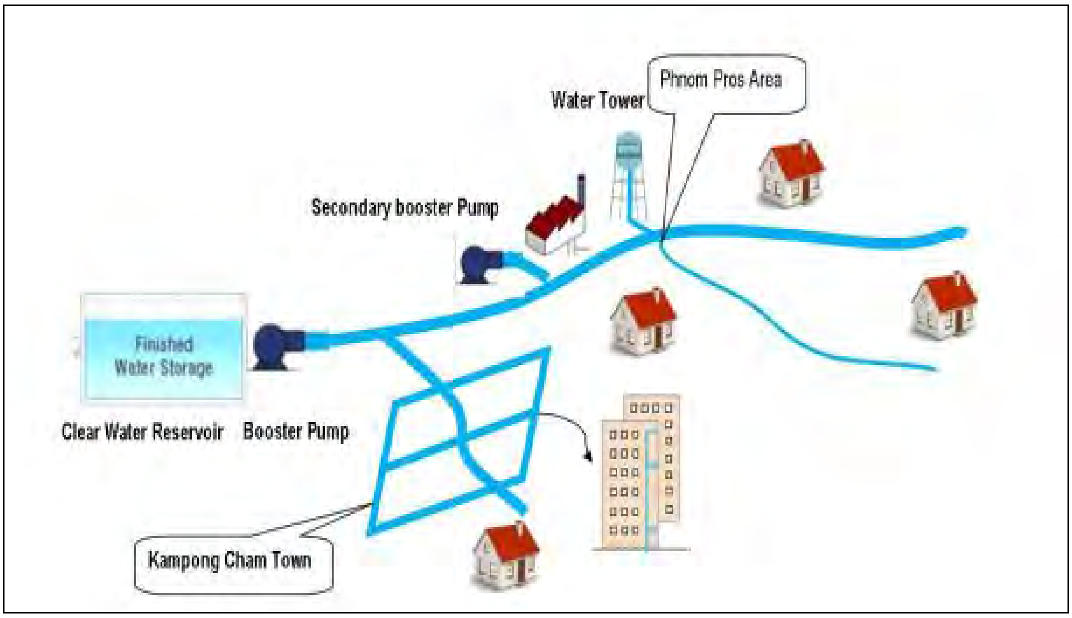


Figure 17. Booster pumps at the WTP and midway supplying the distribution network and elevated water tower at Phnom Pros area



6.5.2 Hourly peak factor

64. The proposed expanded coverage areas are towards the outer suburbs of Kampong Cham, serving more people by the system. In general, the hourly peak factor is related to number of water consumers or amount of total water demand, the larger the number of consumer or larger water demand having a smaller peak value. A hourly peak factor in the range 1.5-3.0 is typically used on urban water supply projects. In the document from JICA Phase II "Design Criteria for Water Supply Facilities" published by the Japan Waterworks Association, the peak hourly factor was determined by the following equation:

$$K = 2.6002 \times (Q/24)^{-0.0628}$$

Where *K – hourly factor*

Q – Daily supply amount

Table 29. Hourly factor for Kampong Cham

Description	Phase I (2020-2025)	Phase II (2026-2040)
Total daily water demand	11,058 m3/d	27,178 m3/d
Hourly factor: K	1.77	1.67

65. The pipe design is based on a 20 year project design period, providing a pipe of sufficient diameter to transport water for 2040 water demand. Two hydraulic modelling scenarios (EPAnet) have been developed: Scenario 1 from year 2020 to 2025 (Phase I), and Scenario 2 from year 2026 to 2040 (Phase II).

6.5.3 Pipe design criteria

66. Pipelines can be divided into raw water pipe, clear water main, main distribution pipeline, secondary distribution pipeline, and house connections. Under the preliminary design, only layouts of raw water pipeline and main distribution pipelines are considered. The secondary distribution and house connection pipelines will be addressed in the detailed design stage following full topo survey and detailed resettlement studies.

67. The most popular pipe material used in urban water supply in Cambodia is the cost-effective high density polyethylene (HDPE) for all distribution networks including trunk lines. HDPE is rigid and tough and it has better stress crack resistance and higher impact resistance than Polyvinyl Chloride (PVC). Less joints are required for HDPE pipe, so longer lengths can be laid at one time as compared to uPVC. For the pumped raw water pipeline, ductile cast iron (DCI) is more durable and practical under higher-pressure situations, particularly for the above-ground sections near intake and WTP inlet.

6.5.4 Network design criteria

68. Some important design criteria are described below :

- Distribution system fed by booster pump.
- Water consumption used in design is in 2025 is 140 lpcd and in 2040 is 155 lpcd
- The modelled water velocity ranges from 0.3 to 1.6 m/s to avoid settlement of particles inside the pipe.
- The minimum and maximum pressure in the network is 50m and 8m respectively.
- Assumed physical water loss 15% of total demand.
- Population growth in each area is calculated using: $P_t = P_0 * (1 + e)^t$
- The roughness calculation is based on the Hazen-William formula:

Formula $V = kCAR^{0.63}S^{0.54}$

Where $V: discharge\ in\ section\ (m/s)$

$C: roughnedd\ coefficient$

$R: hydaulic\ radius\ (m)$

$S: friction\ slope\ (m/m)$

$k: factor\ depent\ on\ unit$

- The selected pipe diameters follow HDPE catalogue, with coefficient roughness C=150.
- Water demand is estimated on average water consumption. It is increased depending on the period of the day, and an hourly peak factor of 1.77 and 1.67 was adopted for determining pipe size in 2025 and 2040 respectively.
- Two pipelines along the main roads have been considered, to avoid damaging the road with multiple crossings.
- Air valves and washout valves will be provided at high and low points.
- Ultimately it is recommended that rider mains[9] will be provided both sides of road from which domestic connections will be made.

[9] A small (usually 50mm) pipe from which domestic connections are made. It is usually capped every 500m or so and has one cross connnection to the distribution main. The purpose is to minimise road crossings and minimize fittings on the main.

6.6 Pumping stations design criteria

69. Three pump stations are proposed, a raw water pump at the intake, a clear water pump inside the WTP grounds and a booster station on the network to Phnom Pros area.

6.6.1 Intake pump station

70. The intake pump should have sufficient capacity and provide sufficient head to meet the requirement of treatment plant, and the pump type should preferably be of the same type as the pumps that have been used at the intake of the JICA Phase II project, to be compatible with existing operation and maintenance requirements.

6.6.2 Clearwater pump station

71. The pumps for the clear water pump station at the WTP shall have the same head as the pumps at the JICA Phase II WTP, to maintain equal water pressure in the network supplied from the two separate WTP's. Pumps are to be installed with capacity to serve year 2025 initially, with floor space reserved for additional pumps to meet further capacity for year 2040 demand.

6.6.3 Distribution pump along network

72. The topographical survey results reveals that the natural ground level in the proposed expansion area toward the Phnom Pros area (along national road No.7) is higher than WTP site by approximately 35m. Booster pumps in the network to serve this area are therefore required.

7 Preliminary Design

7.1 Intake

7.1.1 Intake Structure

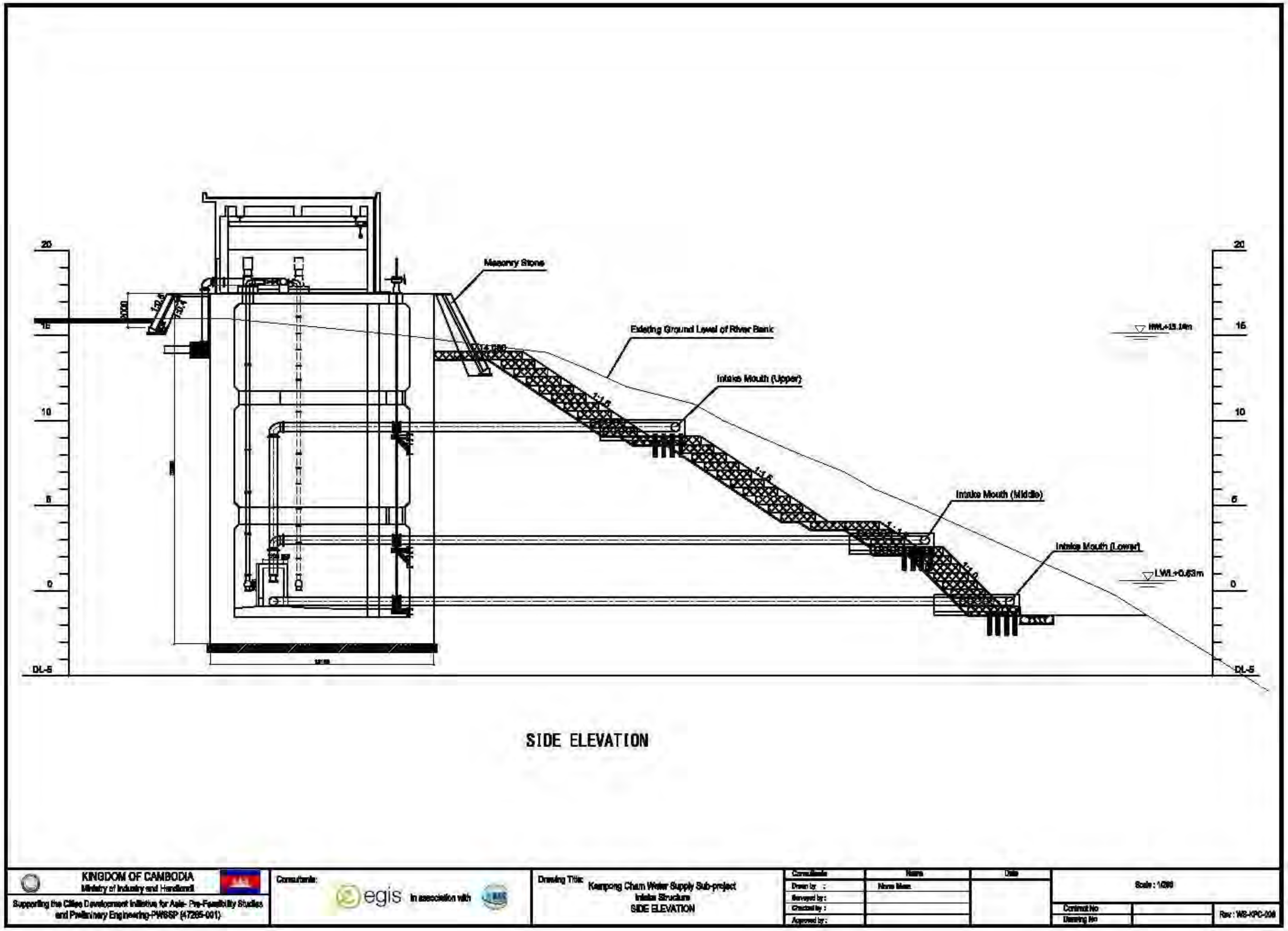
73. The intake structure has been sized for the full 2040 capacity, and reserved space for additional future pumps and associated equipment has been included. The main design elements of the intake are ;

- The intake structure dimensions are 13.40m x 12.95m x 15.70m deep
- The pump room dimensions (W*L*H) are 9.60m x 13.30m x 6.00m.
- RC roof slab will be provided.
- Travelling crane will be installed for pump lifting.
- A full geotechnical soil investigation is required around the intake site.
- Support for the pipe entry and multi-level access platforms with access ladders are to be installed.
- A generator room is to be constructed adjacent to the pump room.
- A RC concrete ramp and steps should be provided if necessary.
- A drainage sump is required.

74. The structural design of the proposed intake works is similar to the existing facilities (JICA 2), and follows international best practice and standards for reinforced concrete. The intake structure is to be constructed in the river bank, and the structural design should be considered for stability in sliding, overturning, uplifting and slope protection (gabion mattress, concrete or stone pavement, rip-rap, wooden stakes or sheet piles).

75. A typical section of the proposed intake structure is shown in Figure 18.

Figure 18. Typical section of intake structure



7.1.2 Intake Pumps

7.1.2.1 Intake pump for 2025, (Q=11,600m3/d)

76. The pump design, for 24 hour operation, includes water treatment loss through the pump, assumed as approximately 5% of the total water demand. Therefore, the water flow rate for pump specification was determined as:

$$\mathbf{Q_p = Q_{total} \times 1.05}$$

Where: Q_{total} = 11,058 m3/d (for Year 2025)

Therefore, the total required capacity for the intake pump is Q_p = 11,610m3/d. **Q=11,600m3/d** is adopted. The number of pumps is determined as 1 duty plus 1 standby, each with capacity Qp = 135 l/s.

Diameter of suction pipe

77. From pump calculations included in Annex 2, the diameter of the suction pipe is approximately 350mm with a water velocity of 1.40m/s.

Diameter of discharge pipe

78. The diameter of discharge pipe is approximately 300mm with water velocity 1.90m/s. The raw water transmission main pipe is calculated as having diameter 350mm with velocity 1.40m/s.

Pumping Head

79. The pumping head is the sum of the static head, entry losses, friction loss in the suction pipe, discharge losses, and other loss from fittings as described in equation below:

$$H_t = H_{stat} + h_{ent} + h_{fs} + h_{fd} + \sum h_m$$

80. The total dynamic head (Ht) has been calculated at 41m; see detailed calculation in Annex 2

Type of pump

81. The total dynamic head required is over 10m (Ht = 41m). A vertical turbine pump is designed for this higher suction lift and an example layout is shown in Figure 19. The motor sits on top of the well, with the impellers at the bottom connected by a long vertical shaft.

82. Both vertical turbine pumps and submersible pumps have been presented as options for the intake pumps at the DFR Workshop. MIH commented that submersible pumps are not to be considered due to previous problems experienced by various waterworks (spare parts not available easily). The existing water intake pumps under the JICA 2 project are vertical turbine pumps, and waterworks officials are hesitant to use different types of pumps both to maintain familiarity and for ease of sourcing spares and maintenance.

83. Horizontal split-casing pumps have been suggested as an alternative. They may be easier to maintain, but do not currently exist in the Cambodian water sector and are not suitable for pumping with high suction lift.

84. JICA Phase II booster pump systems are already utilizing a Frequency Inverter system for energy savings and to improve efficiency. Under this subproject the variable frequency inverter (VFI) system will also be adopted in all of the pumping systems.

Figure 19. Typical Vertical Turbine Pump arrangement

7.1.2.2 Intake pump for 2040 (Q=17,000m3/d)

85. From the same calculations as described above it can determined that for year 2040 :

- The total capacity of the intake pump is Qp = 17,000m3/d or 197 l/s. The number of pumps is 1 duty and 1 standby.
- The diameter of the suction pipe is approximately 350mm with water velocity 1.73m/s.
- The diameter of the discharge pipe is approximately 300mm with velocity 2.35m/s.
- The raw water transmission main pipe is of diameter 400mm with velocity 1.30m/s.

Pumping Head

86. The pumping head is the sum of the static head, entry losses, friction loss in the suction pipe, discharge losses, and other loss from fittings as described in equation below:

$$H_t = H_{stat} + h_{ent} + h_{fs} + h_{fd} + \sum h_m$$

The total dynamic head (H_t) has been derived and H_t=36m, see detail calculation in Annex 2.

Type of pump

87. For the same reasons described for the Intake pumps above, the vertical turbine pump is selected. The intake pump characteristics for 2025 and 2040 are summarized in the table below.

Table 30. Summary of intake pump capacity and characteristics

Description	Year 2025	Year 2040
Total Pumping capacity	135 l/s	197 l/s
Pump Total head	Ht = 41 m	Ht = 36 m
Pump type	Vertical Turbine	Vertical Turbine
Number of pump	1-Duty +1 Standby Q=135 l/s each pump	1-Duty additional Q=197 l/s (1)
Total Power	75 kw	180 kw
Suction pipe	Φ350 mm	Φ350 mm
Discharge pipe	Φ300mm	Φ300mm
Transmission pipe	Φ350mm; L = 4Km	Φ400mm; L = 4Km
Transformer 22KV/0.4KV, 3 phases 4 wire, Outdoor Type	160kVA	315kVA
Generator 400 V/230 V, 3 phases 4 wire, 50Hz, Silent Type	110kVA	275kVA
MV Pole Concrete for Incoming Overhead Line 22KV	2-pcs; voltage=22KV; AL-70mm2	

7.2 Water Treatment Plant

88. The required design capacity of the proposed WTP has to provide both sufficient water to serve the JICA Phase II coverage area after year 2019, and to provide additional capacity for the proposed extended coverage area under this project to 2025 water demand. However, the design has taken account of the full required WTP capacity for 2040 water demand.

89. The preliminary design dimensions of the different structures proposed for detailed design and construction at the WTP site are summarized below.

Table 31. Preliminary components of WTP (11,600m3/d) in 2025

Facility/Equipment		Component
Water Treatment Plant	Receiving Well	Reinforced Concrete Structure Internal Dimension: 1.60m width x 4.10m length x 4.50m depth Volume: V=29.5 m^3
	Rapid Mixing Tank	Reinforced Concrete Structure Gravitational force mixing using a weir Internal Dimension: 1.60m width X 1.50m length X 3.88m depth Volume: V=9.3 m^3
	Flocculation Basin	Reinforced Concrete Structure Slow Mixing Method: Vertical channel bands flocculator Internal Dimension: 6.90 m width x 3.25 m length x 4.50 m height + 3.46 m average effective depth Quantity: 3
	Sedimentation Basin	Reinforced Concrete Structure Horizontal-Flow Sedimentation Basin Supernatant Collecting System: Collecting Trough + Submerged Orifice Internal Dimension: 6.90 m width x 21.50 m length x 3.99 m average depth Quantity: 3

Facility/Equipment		Component
	Rapid Sand Filter	Reinforced Concrete Structure Internal Dimension: 2.50 m width x 7.00 m length Quantity: 6 Filter Sand Thickness: 100 cm Underdrain System: Porous Filter Bed Method Filtration Rate: V=120.5 m/day (120-150 m/day) Flow Control: Lower Part Control Method Backwash Method: Simultaneous Backwash Method by Air and Water
	Treated Water Reservoir	Reinforced Concrete Structure using Flat Slab Structure Quantity: 2 Effective Volume: V=2,500 m3 (1,250 m3 x 2) Effective Water Depth:H=3.80 m (3-6 m) Detention Time: T=3 hours Internal Dimension: 10.40 m width x 32.00 m length x 4.50m height Chlorine dosing weir at inlet
	Wastewater Basin	Reinforced Concrete Structure Quantity: 2 Effective Volume: V=211 m3 (105.5 m3 X 2) Internal Dimension: 4.00 m width x 11.00 m length x 5.60m height + 2.40m depth
	Sludge Drying Bed (Lagoon)	Reinforced Concrete Structure Quantity: 4 Effective Area: A=790 m^2
	Chemical Feeding Facility	Alum, Lime: at Administration Building Chlorine: Chlorine Feeding House (Floor Area 61.3m^2)
	Emergency Generator	450 KVA (Soundproof type, Equipped with Internal Water Tank) At Administration Building
	Administration Building	Reinforced Concrete Structure, Total Floor Area: 588 m2 Customer service and laboratory, administration and technical services, meeting and staff rooms

90. The chemical dosing system uses aluminum sulfate for coagulant, lime for pH and alkalinity adjustment, and chlorine (powdered or granule sodium hypochlorite) for disinfection.

91. It is proposed to use powdered sodium or calcium hypochlorite for chlorination. The existing older central WTP already uses this. The newer WTP recently constructed under JICA Phase II utilises a gaseous chlorine system under a negative pressure, but this carries a safety risk for the WTP and surrounding community, and is not available in Cambodia. All gas refills must be imported.

92. Replacement of gaseous chlorine systems in WTP's across Cambodia was a key recommendation of the 2013 PPTA for the Urban Water Supply & Sanitation project[10], because of the failure of all the gas chlorine system installed under a previous 2006 ADB implementation project[11]. It is not known why it was installed under JICA projects.

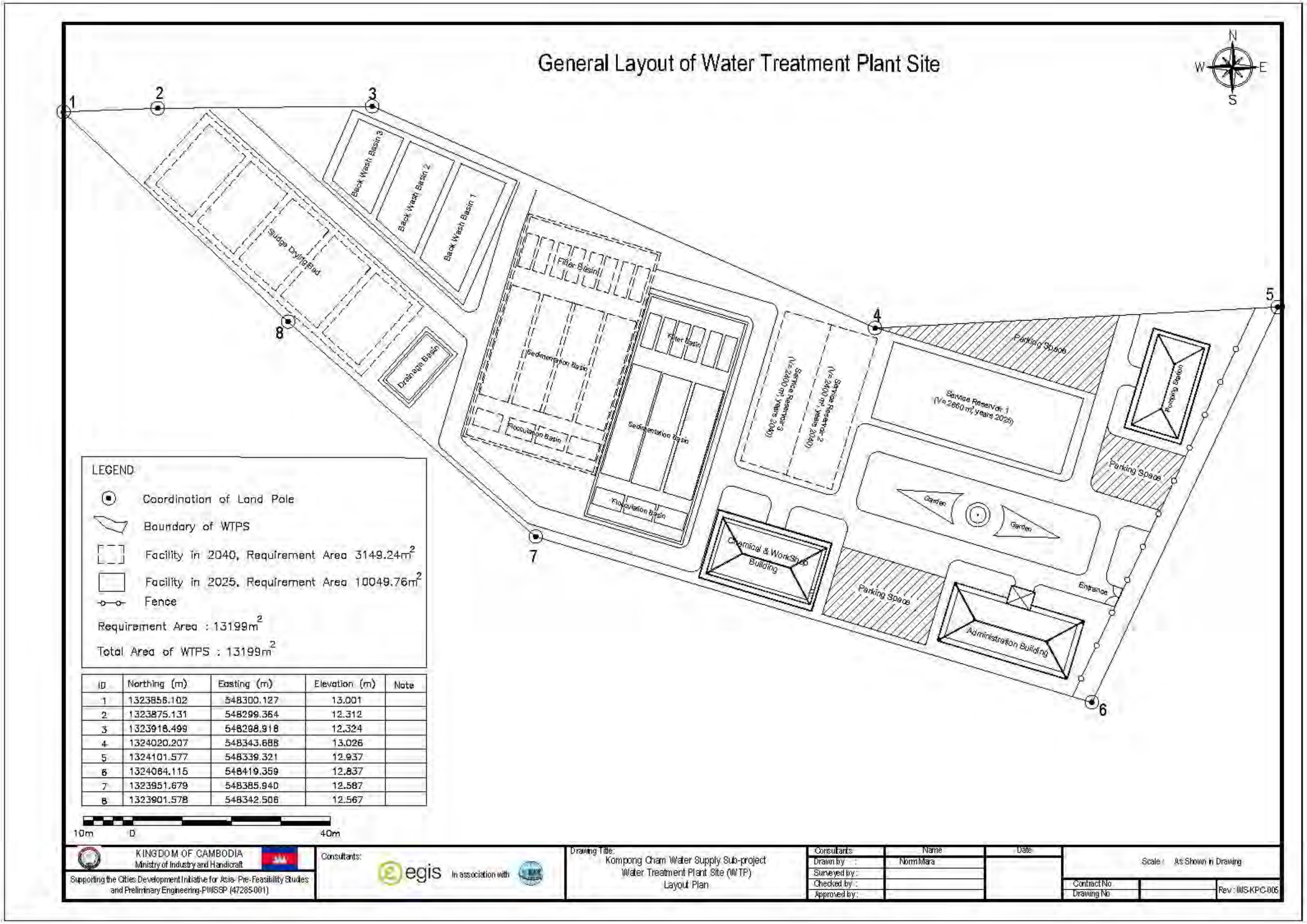
93. There have been some reported difficulties with using powdered source chlorine – dust and blocking of dosing lines – but these can be easily mitigated by wearing masks, eye protection and gloves, and by having outlet pipes from mixing and constant head tanks at least 20cm above the tank base, to ensure settled insoluble matter does not enter the dosing pipelines.

[10] ADB PPTA: TA-8125-CAM
[11] Metering & dosing equipment corroding and being repaired with hazardous "home made" temporary fixes.

94. Total land requirement is approximately 0.9 ha for WTP Phase-I (including service building and workshop) with a further 0.4 ha required for WTP Phase-II.

95. The preliminary layout of the proposed water treatment plant (WTP) is shown on Figure 20. The two proposed phases of the WTP are shown in the layout: Phase 1 is demonstrated by solid-line and Phase 2 by dash-line. The service building, pump station, chemical/workshop building, and other items common to both phases are included in Phase 1 construction.

Figure 20. Conceptual layout of proposed WTP



ID	Northing (m)	Easting (m)	Elevation (m)	Note
1	1323856.102	548300.127	13.001	
2	1323875.131	548299.364	12.312	
3	1323918.499	548298.918	12.324	
4	1324020.207	548343.688	13.026	
5	1324101.577	548339.321	12.937	
6	1324064.115	548419.359	12.837	
7	1323951.679	548385.940	12.587	
8	1323901.578	548342.506	12.567	

7.3 Distribution system

7.3.1 Clear water Reservoir Preliminary Design

96. The clear water reservoir capacity was calculated considering the hourly peak factor, which is 1.77 and 1.67 in year 2025 and 2040 respectively. Usually the peak hour occurs between 6:00-8:00AM and 5:00-8:00PM, coinciding with time in and time out of school and working hours in Cambodia. Three hours' storage for the clear water reservoir was proposed for each phase. The sizing of the clear water reservoir are described in Table 32.

Table 32. Sizing of the clear water reservoir

Specifications	Phase I (2020-2025)	Phase II (2026-2040): Additional storage to Phase I
Hourly water demand (m^3/hr)	11,600/24 (483m^3/hr)	17,000/24 (708m^3/hr)
Hourly peak	1.77	1.67
Retention time (hour)	3	3
Stored water volume when WTP not operating (m^3)	1,449	2,124
Absorb peak hourly flow (m^3/d)	3 x (855 - 483) = 1,116	3 x (1,182 – 708) = 1,422
Total storage required (m^3/d)	2,565	3,546

Note: The clearwater reservoir would be reinforced concrete structure underground

7.3.2 Pump characteristics for Clearwater pumping

97. The capacity of the required booster pumps need to be designed to meet hourly peak demand. Two phases of the project have been planned and the two scenarios hydraulically modelled. Based on the EPAnet model results, the capacity of the booster pump can be described as follows:

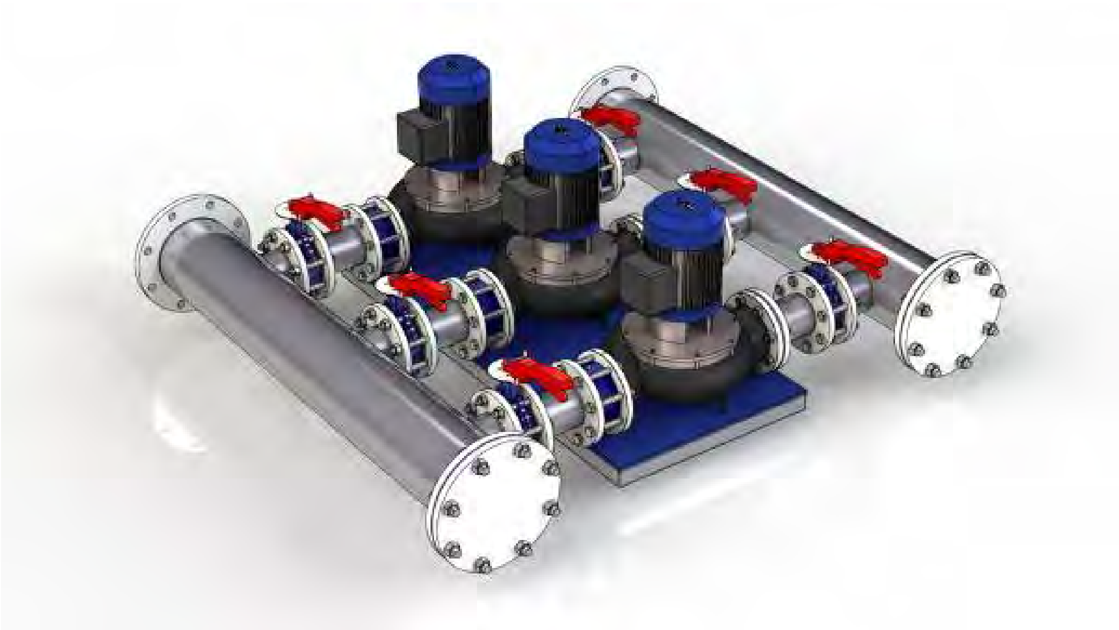
- **Phase 1** (2020-2025):
  Booster pump at WTP requires capacity of 814m³/h or 226 l/s, and total head 35m (36m for pump selection)

- **Phase 2** (2026-2040) :
  Booster pump at WTP requires total capacity of 1,076 m³/h or 299 l/s, and total head 35m (36m for pump selection)

98. Vertical in-line centrifugal pumps are the type of pump commonly used for distributing potable water to consumers in Cambodia. A packaged booster system is proposed for use at the WTP. The package booster system is factory designed for optimized pumping and simplified installation (smaller footprint, easy to install, lower installation cost, one electrical connection, no shaft/coupling alignments/adjustments, and integrated control. Figure 21. shows a typical layout. The benefits of a packaged booster system are :

- Saves Energy Costs: Efficient cascade control, application optimized software and pumps in the industry
- Single Source Responsibility: One manufacturer for pumps, motors, drives & control
- Plug-and-Pump: Easy to install and commission
- Easy to Operate: Large, clear, user friendly & advanced controls interface
- Reduce floor space: Space-saving complete solution

Figure 21. Typical packaged booster system



99. Packaged booster systems usually consist of two to six identical vertical in-line multi stage pumps connected in parallel and mounted on a common base frame provided with a control cabinet and all the necessary fittings. The pumps of the booster system can be removed without interfering with the pipework on either side of the manifolds.

100. It is possible to let one or more pumps function as standby pumps. A booster system with for example four pumps, one having the status of standby pump, will run like a booster system with three pumps, as the maximum number of pumps in operation is the total number of pumps minus the number of standby pumps. If a pump is stopped due to a fault, the standby pump will be cut in. This function ensures that the system can maintain the rated performance even if one of the pumps is stopped due to a fault. The status as standby pump alternates between all pumps of the same type, to ensure equal useage.

101. The proposed packaged booster system for pumping clearwater from the reservoir to distribution will have one common controller for all of the pumps. Many manufacturers make vertical inline centrifugal pumps and can provide packages.

102. Horizontal split case centrifugal pump has been considered for use in pumping clearwater to the network. However they require a larger footprint for installation and require regular inspection of the alignment of the horizontal shafts and associated bearings. Vertical in-line centrifugal pumps require a smaller footprint for installation and do not require constant inspection of alignment, having its drive shaft from the motor to the pump directly coupled removing the risk of mis-alignment even after a long period of operation.

7.3.2.1 Pumping Clearwater to Network for 2025

Pump capacity

103. The capacity of the pump selected should consider both the peak day factor (for seasonal demand variation) and the peak hourly factor, which varies depending on the type of service area (for example, peak hourly factor for urban area is normally less than peak factor for rural area). For Kampong Cham the daily peak factor selected is 1.2. and peak hourly factor has been calculated as 1.77 for 2025. Therefore, the required capacity of the booster pumps for 2025 is 226 l/s. Vertical in-line single stage pumps are selected, with 2 duty plus 1 standby.

Pumping head

104. The total head depends on design criteria such as head loss along pipe, elevation difference from WTP to end of pipe network, and the target pressure requirement. The total head for the booster pump at the WTP is estimated at 36m.

7.3.2.2 Pumping Clearwater to the network for 2040

Pump capacity

105. As above, the selected daily peak factor is 1.2 and peak hourly factor calculated as 1.67. The additional capacity of the booster pumps for 2040 is 299 l/s. The number of pumps selected is 2 additional duty pumps each of 165 l/s capacity. The booster pumps in year 2025 and 2040 at the clearwater tank were simulated with the same capacity to minimize number of pumps and power consumption. A VFD will be used to control the flows in 2040.

Pumping head

106. The total head depends on design criteria such as head loss along pipe, elevation difference from WTP to end of pipe network, and the target pressure requirement. The total head for booster pump at the WTP is estimated at 36m.

7.3.2.3 Summary of pumps for Clearwater pumping station

Table 33. Summary of pump parameters at WTP

Description	Year 2025	Year 2040
Total Pumping capacity	226 l/s	299 l/s
Pump Total head	Ht = 36 m	Ht = 36 m
Pump type	Vertical In-line Single Stage	Vertical In-line Single Stage
Number of pump	2-Duty +1 Stand-By Q=119 l/s each pump	2 additional Duty Q=165 l/s each pump
Total Power	110 kw	180 kw
Transformer 22KV/0.4KV, 3 phases 4 wire, Outdoor Type	500kVA	250kVA
Generator 400 V/230 V, 3 phases 4 wire, 50Hz, Silent Type	350kVA	250kVA
MV Pole Concrete for Incoming Overhead Line 22KV	18-pcs; Voltage=22KV; AL-150mm2	

7.3.3 Distribution network preliminary design

7.3.3.1 Pipe network

107. EPAnet software has been used to select diameter of pipeline following the design criteria. The diameter of pipelines have been designed for the total demand in 2040 in peak hour, with a booster pump in the network on the main line to the Phnom Pros area.

- **In Phase 1 (up to 2025):** Total water demand is 814m³/h (hourly peak factor 1.77).
- **In Phase II (up to 2040):** Total water demand is 1,076m³/h (hourly peak factor 1.67).

108. Three options for the distribution arrangement have been modelled, to investigate the optimization of the capital and operation costs. The results of the model runs are described below, and the comparisons of both options summarized.

109. The arrangement of the distribution network in Option 1 (with booster station at the WTP with water directly distributed to customers) is shown in Figure 15. The model results indicate that zero water pressure is available in the network at the proposed new extension area (due to higher ground level in the Phnom Pros area), even with a starting pressure at the WTP of 5 Bar. Thus this option is discounted as it can not provide water to the expansion area as proposed.

110. The arrangement of the distribution network in Option 2 (with booster station at the WTP and additional booster station further downstream in the network): The results are summarized in Table 34. This table indicates that by 2025 the main pipeline (from WTP to town) will require diameter 500mm and a length of 4,200m,and after 2025 one additional parallel pipeline with diameter of 560mm and length of 2,789m (to connect with the JICA II pipeline) will be required to maintain sufficient water pressure and flow to meet demand.

111. Figure 21. and Figure 23. show the pipe layout in 2025 and 2040.

Table 34. Summary of design of distribution network Option 2

No.	Results		
I.	**Pipe Diameter (mm) (*1)**	**Presure Normal (PN)**	**Pipe Length (m)**
1	63	PE100,PN 10	16,213.00
2	75	PE100,PN 10	2,485.00
3	90	PE100,PN 10	3,006.00
4	110	PE100,PN 10	5,394.00
5	125	PE100,PN 8	1,407.00
6	160	PE100,PN 8	4,736.00
7	200	PE100,PN 8	3,842.00
8	225	PE100,PN 8	1,576.00
9	250	PE100,PN 8	2,324.00
10	315	PE100,PN 8	2,605.00
11	450	PE100,PN 8	354.00
12	500	PE100,PN 8	7,631.00
13	560	PE100,PN 8	606.00
A. Sub-total -1			**52,179.00**
II.	**Additional pipe after year 2025**	**Presure Normal (PN)**	**Pipe Length (m)**
14	560mm	PE100,PN 8	2,789.00
B. Sub-total -2			**2,789.00**
Grand Total (I+II)			**54,968.00**
III.	**Total Pump Capacity in 2025**	**Flow (m3/h)**	**Head (m)**
15	At Water Treatment Plant	815	35
16	Booster pump along the PNW	488	28
IV.	**Total Pump Capacity Total in 2040**	**Flow (m3/h)**	**Head (m)**
17	At Water Treatment Plant	1,893	35
18	Booster pump along the PNW	662	32

Note: (*) Pipe diameter here follow pipe catalogue

112. The total pipe length in the model run for year 2025 is approximately 52km, along the existing road sides in the proposed expansion area. In addition to this 52km, pipe for supply to houses 2 or 3 rows back from the main roads with smaller diameters (63-110mm) account for a further 28km. In Year 2040 further pipelines with total length 40km (pipe diameter 63-560mm) will be needed in the coverage area to increase the served population up to 90-100% of the service area.

Table 35. Summary of Proposed Pipe design for option 2 distribution network

No.	Pipe Diameter	Presure Normal (PN)	Pipe Length	Additional pipe	Total pipe provision
	mm	PE100	m	m	m
1	63	PN 10	16,213	5,565	21,778.00
2	75	PN 11	2,485	5,564	8,049.20
3	90	PN 12	3,006	4,173	7,179.15
4	110	PN 13	5,394	2,782	8,176.10
5	125	PN 8	1,407	2,782	4,189.10
6	160	PN 9	4,736	1,391	6,127.05
7	200	PN 10	3,842	1,391	5,233.05
8	225	PN 11	1,576	2,782	4,358.10
9	250	PN 12	2,324	1,390	3,714.00
10	315	PN 13	2,605		2,605.00
11	450	PN 14	354		354.00
12	500	PN 15	7,631		7,631.00
13	560	PN 16	606		606.00
	Total:		**52,179**	**27,821**	**80,000**

Figure 22. Pipeline layout for 2025 under Option 2

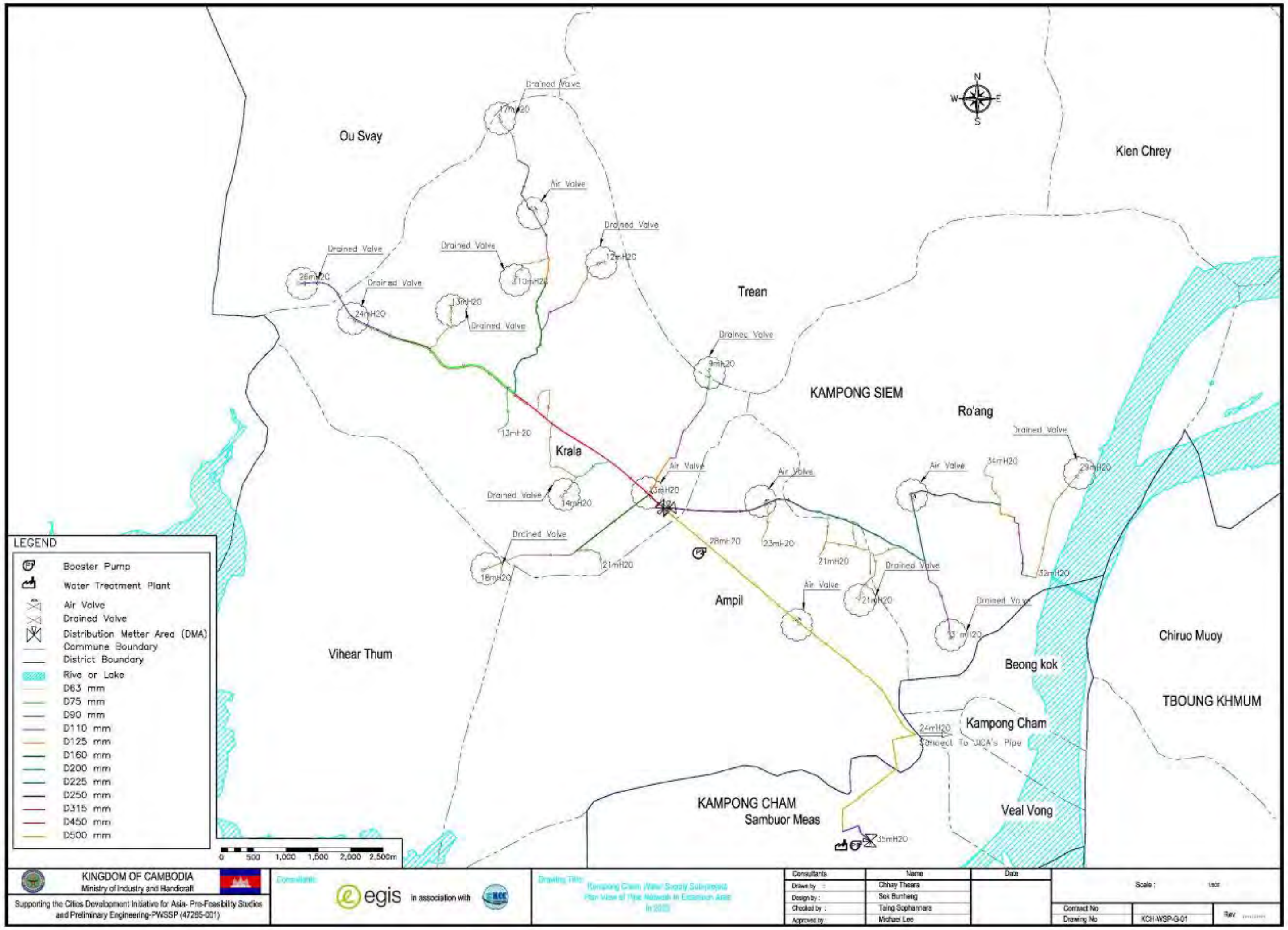
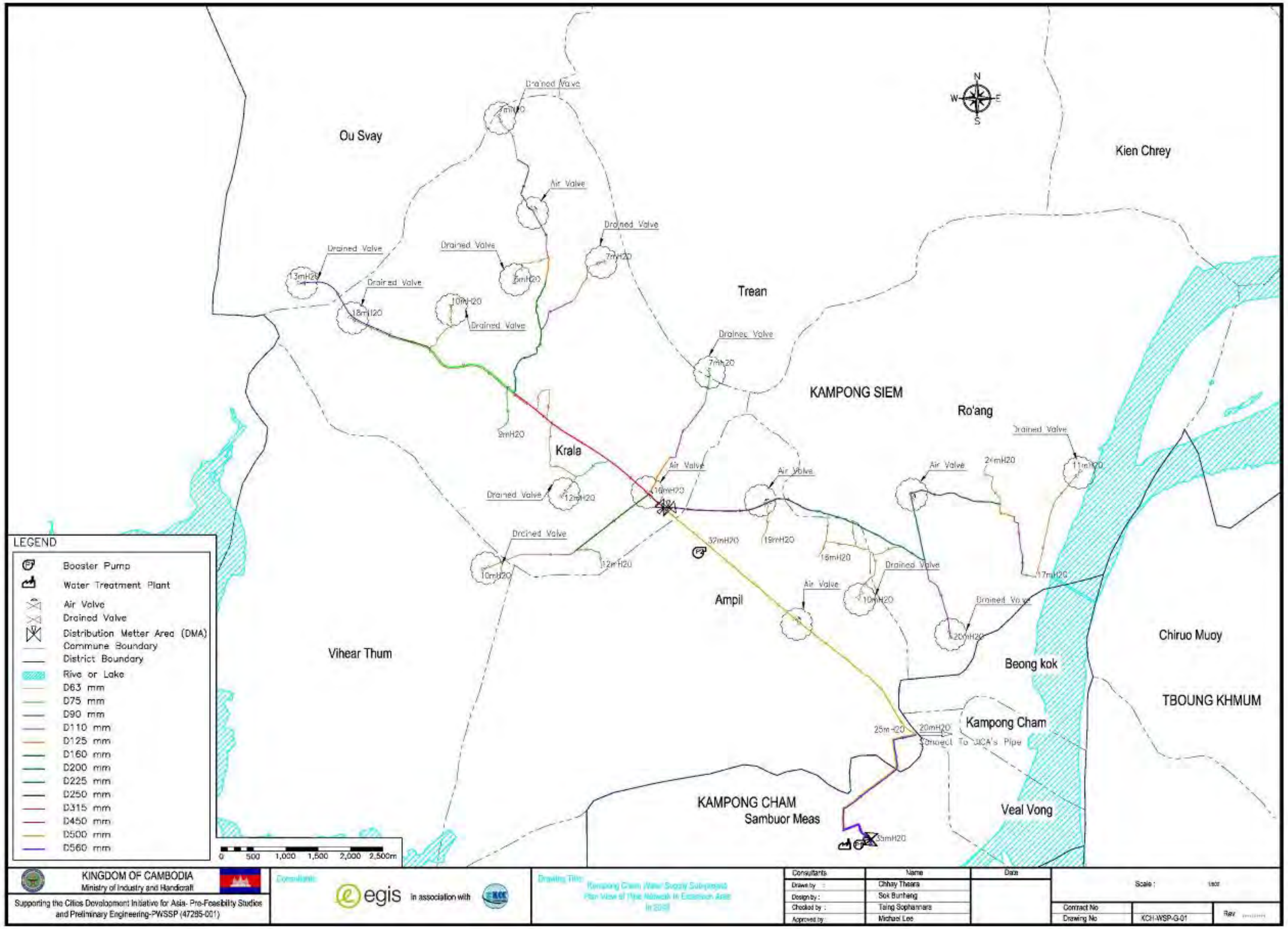


Figure 23. Pipeline layout by 2040 under Option 2



113. The arrangement of the distribution network in Option 3 (with booster station at the WTP and both an additional booster station further downstream in the network and an elevated water tower): From the model results the elevated water tower is ineffective without an additional booster station in the network, due to the natural ground level in the Phnom Pros area being 35m higher than the ground level at WTP. The Option 3 model results are described in the following tables and figures.

Table 36. Summary of design of distribution network Option 3

No.	Results		
I.	**Pipe Diameter (mm) (*1)**	**Presure Normal (PN)**	**Pipe Length (m)**
1	63	PE100,PN 10	15,477.00
2	75	PE100,PN 10	3,221.00
3	90	PE100,PN 10	4,316.00
4	110	PE100,PN 10	3,445.00
5	125	PE100,PN 8	2,297.00
6	160	PE100,PN 8	6,552.00
7	200	PE100,PN 8	2,542.00
8	250	PE100,PN 8	3,132.00
9	315	PE100,PN 8	1,793.00
10	355	PE100,PN 8	812.00
11	400	PE100,PN 8	355.00
12	450	PE100,PN 8	1,059.00
13	500	PE100,PN 8	7,178.00
A. Sub-total -1			**52,179.00**
II.	**Additional pipe after year 2025**	**Presure Normal (PN)**	**Pipe Length (m)**
14	560mm	PE100,PN 8	2,789.00
B. Sub-total -2			**2,789.00**
Grand Total (I+II)			**54,968.00**
III.	**Total Pump Capacity in 2025**	**Flow (m3/h)**	**Head (m)**
15	At Water Treatment Plant	815	35
16	Booster pump along the PNW	488	31
IV.	**Total Pump Capacity Total in 2040**	**Flow (m3/h)**	**Head (m)**
17	At Water Treatment Plant	1,893	35
18	Booster pump along the PNW	662	32
V.	**Water Tower**	**Volume (m³)**	**Hieght bottom of the tank (m)**
19	Water Tower near Phnom Pros (supply around 1 hour for hourly peak) (*2)	365	16

Note:

*(*1) Pipe diameters here follow pipe catalogue.*

*(*2) The location of water tower will be built at point in the Phnom Pros area where the ground level is 47.57masl*

The volume of water tower is 310 m³ with the height bottom of the tanks 15 m. It is designed for 24 hours storage.

Figure 24. Pipeline layout for 2025, option 3

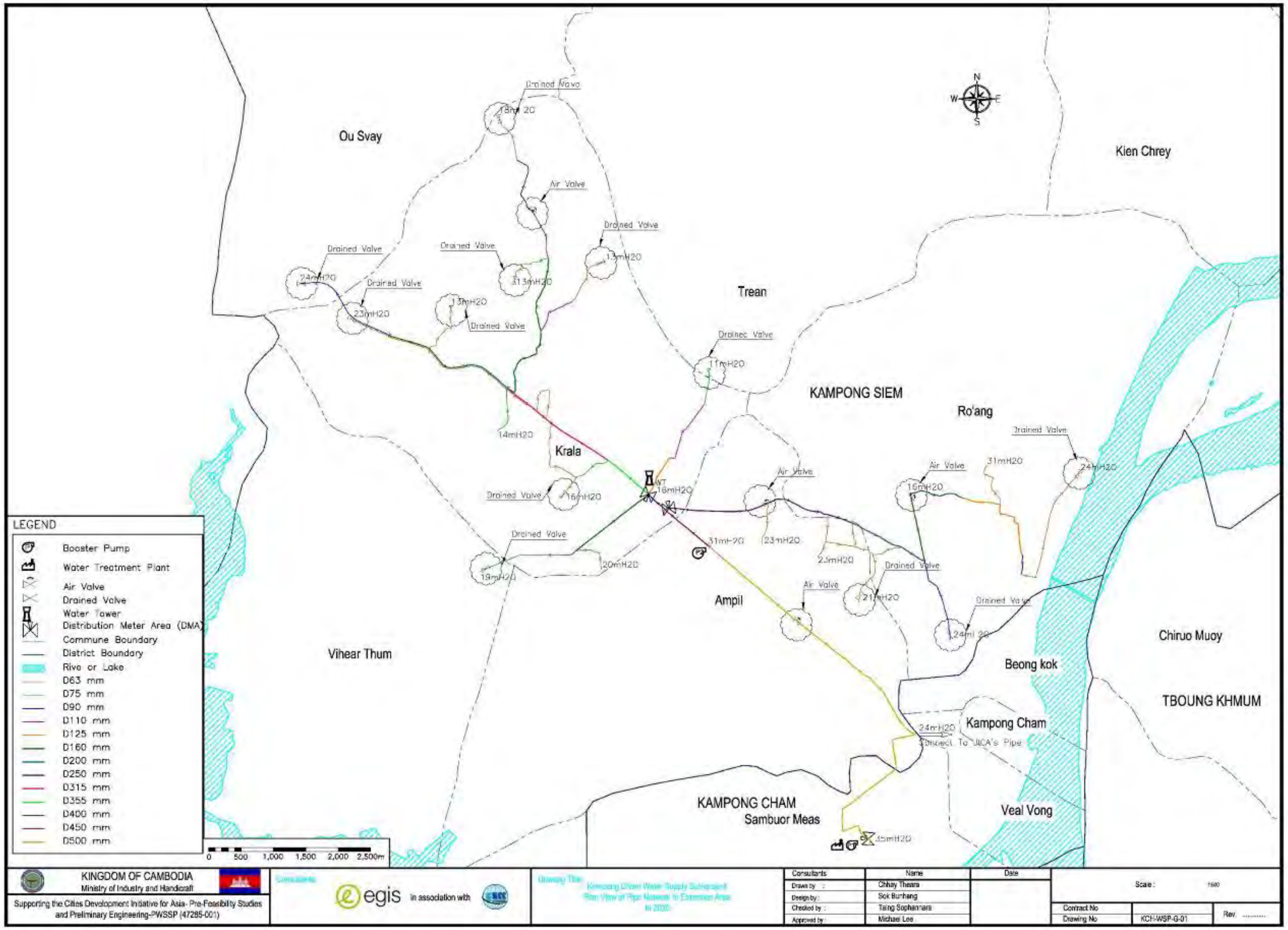
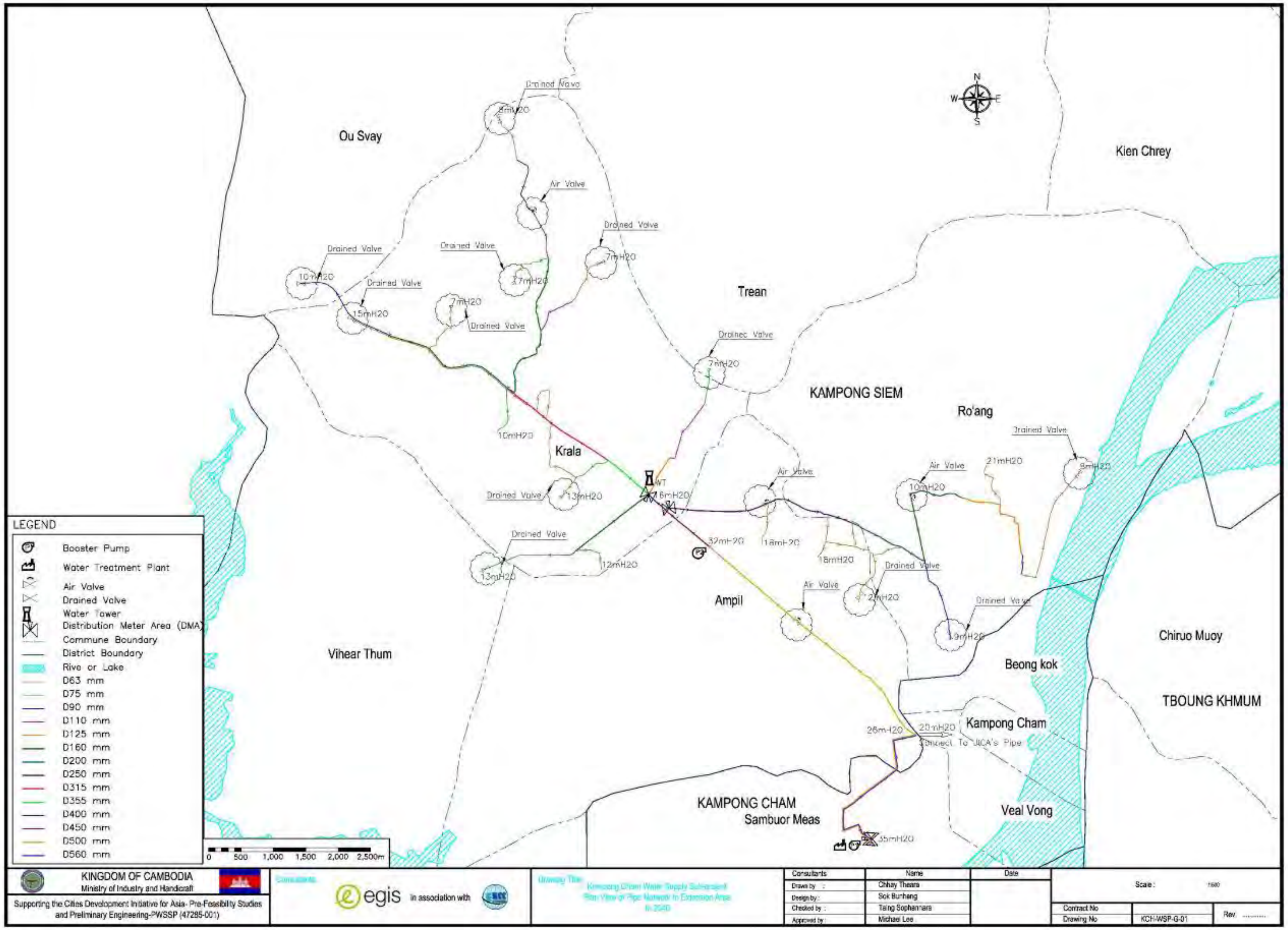
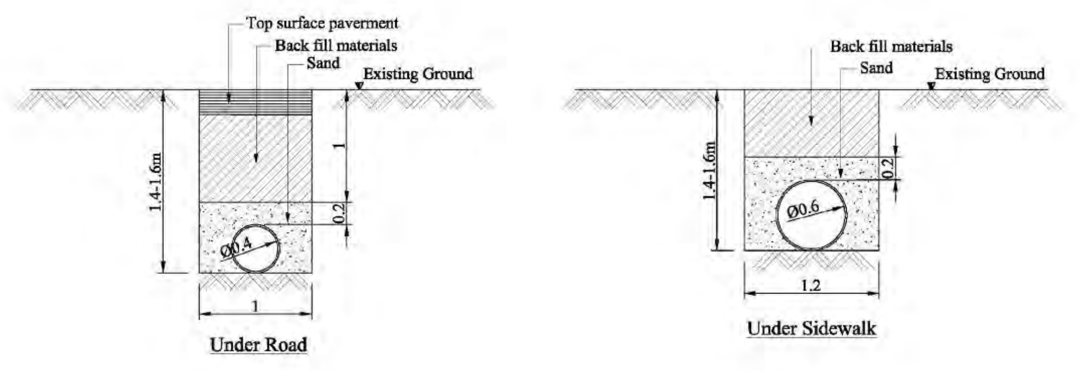


Figure 25. Pipeline layout for 2040, option 3



114. The typical detail of pipe trench and backfill material for the water transmission pipeline and main distribution line is shown below.

Figure 26. Typical detail for pipe trench and backfill



7.3.3.2 Pump along distribution network

A) Booster pump capacity in network for 2025

115. Due to the elevated ground level in the Phnom Pros area a booster pump is needed, and a small footprint booster pump system mid-way along the distribution network has been selected. The capacity of the pumps required for 2025 is 136l/s. The number of pumps selected is 2 duty and 1 standby, each with 68 l/s capacity. The total pump head required is approximately 29m.

B) Booster pump capacity in network for 2040

116. Likewise, the capacity of the booster pump for 2040 is 48l/s. The number of pumps selected is 1 duty 1 standby and the total pump head required is approximately 33m.

Table 37. Selection of booster pump capacity and characteristics

Description	Year 2025	Year 2040
Total Pumping capacity	136 l/s	48 l/s
Pump Total head	Ht = 29 m	Ht = 33 m
Pump type	Vertical In-line single stage	Vertical In-line single stage
Number of pump	2-Duty +1 Standby Q=68 l/s each pump	1-Duty Q=48 l/s pump
Total Power	60 kw	22 kw
Transformer 22KV/0.4KV, 3 phases 4 wire, Outdoor Type	100kVA	50kVA
Generator 400 V/230 V, 3 phases 4 wire, 50Hz, Silent Type	100kVA	35kVA
MV Pole Concrete for Incoming Overhead Line 22KV	2-pcs; Voltage=22KV; AL-70mm2	

Note: Packaged booster system: 1-set for 2025
Add one further pump to be incorporated in the first package of pumps for 2040
(Same pump set up/concept as in the booster pump at WTP.)

7.4 Summary of the project facilities

117. There are 2 project phases proposed. Phase 1 is proposed to be implemented within 2018-2020, with an estimated construction period of 2-2.5 years The facilities in Phase 1 are described in Table 38. They will provide safe water with sufficient capacity to serve 85-100% of people within the expansion areas on the outskirts of Kampong Cham and to some communes in Kampong Siem district. Thus the additional water demand in the original downtown service area from 2020 to 2025 will be covered.

118. Phase 2 is planned to provide safe water for the whole service area for target period 2026-2040. The facilities in Phase 2 are described in Table 39. They will serve 90-100% of the total population in the proposed expansion area on the outskirts of Kampong Cham and some communes in Kampong Siem district.

Table 38. Outline of Facilities in Phase 1 (2020-2025)

1)	Water source	Mekong river; HWL=15.18 m, LWL=0.63 m
2)	Intake Capacity	11,600 m3/d
	Pump at intake	• Total pump capacity: 135 l/s • Three pumps (vertical turbine pump) will be installed: 1 in duty and 1 standby • Total head 41m
3)	WTP Capacity	11,600 m3/d
	Booster pump at WTP	• Total pump capacity: 227 l/s • Three Booster pump will be installed: 2 in duty and 1 standby • Total head 36m (following pump selection)
	Treatment facilities	• Rapid Mixing • Flocculation • Sedimentation • Filtration • Disinfection
4)	Raw water transmission pipe (DIP)	Diameter : 350 mm ; Length : 4 km
5)	Distribution Networks	
	Pipe network	52 Km main pipes + 28 km of smaller pipes
	Booster pump along the distribution network	• Total pump capacity: 136 l/s • Three Booster pump will be installed: 2 in duty and 1 standby with capacity 68 l/s each pump • Total head 29m (following pump selection)
6)	Construction plan	2018-2020

Table 39. Outline of Facilities in Phase 2 (2026-2040)

1)	Water source	Mekong river; HWL=15.18 m, LWL=0.63 m
2)	Intake Capacity	17,000 m3/d
	Pump at intake	• Total pump capacity: 197 l/s • 1 additional dutypump will be installed • Total head: 36 m
3)	WTP Capacity	17,000m3/d
	Booster pump at WTP	• Total pump capacity: 300 l/s • Five Booster pumps at WTP will be installed: 3 on-duty • Total head 36m (following pump selection)
	Treatment facilities	• Rapid Mixing • Flocculation • Sedimentation • Filtration • Disinfection
4)	Raw water transmission	Diameter : 400 mm ; Length : 4 km

	pipe (DIP)	
5)	Distribution Network	
	Pipe networks	40 km
	Booster pump along the distribution network	• Total pump capacity : 48 l/s • One additional duty Booster pump at will be installed • Total head 33m (following pump selection)
6)	Construction plan	2028-2030

8 Preliminary Cost Estimates

119. The preliminary cost estimates for Phases 1 and 2 of the proposed subproject are presented in Table 41. and Table 42. , respectively. The unit rate for the WTP has been selected from available cost data from previous projects in Cambodia. Capital costs for WTP's rise with capacity with a logarithmic function. Ideally we would have ten or more recent completed WTP costs, but much data from the pre-2010 period has been lost. Recent previous water supply projects in Cambodia are Battambang and Kampong Cham (JICA, estimated), Mondul Kiri (JIAF, US$ 600/m3/d), Rattanakiri (private, US$ 215/m3/d) and Siem Reap (private). Data from Mondulkiri was not relevant as it was an imported package plant, data from Ratanakiri not relevant as the private plant was to a much lower standard than GoC plants, and Siem Reap cost data is not yet available as the contract has not been awarded yet. This left Battambang and Kampong Cham for reference as shown in Table 40. Thus the conservative unit rate cost adopted is US$ 450/m3/d for the WTP including pump, yard pipes and other associated facilities, including all labour.

Table 40. Historical cost data for WTP construction

Town	WTP capacity (m3/d)	Capital Cost ($)	Unit cost ($/(m3/d))
Battambang	22,000	10,000,000	456
Kampong Cham	11,500	8,104,000	705

120. Detailed engineering works and the project management and construction supervision (including O&M training) are not included in this unit rate as the engineering works are to be included in the Design-Build contract, and the project management and construction supervision works depend on the scope of works[12] and are proposed under the project implementation and assistant (PIAC) contract.

Table 41. Phase 1 (2020-2025) – Preliminary Cost Estimate

No.	Description	Unit	Capacity/ quantity	Unit rate	Amount US$
1	New WTP (Completed set including CW pumps)	m3/d	11,600	450	5,220,000
2	New Intake and equip with pump				-
	Intake structure	set	1	915,000	915,000
	Pumps (2 duty, 1 standby)	pcs	3	80,000	240,000
3	Feeding pipeline (DCI 350mm)	km	4	105,000	420,000
4	Pipe network	km	80	45,000	3,600,000
5	Network Booster pumsp(i)	set	3	50,000	150,000
6	Electrical facilities				-
	At treatment plant	set	1	300,000	300,000
	At intake	set	1	140,000	140,000
	At booster pump along network	set	3	140,000	420,000
7	Service and workshop building	set	1	200,000	200,000
8	Contingency 10%	LS			1,160,500
	Total				**12,765,500**

Note: (i) two duty and one standby

Table 42. Phase 2 (2026-2040) – Preliminary Cost Estimate

No.	Description	Unit	Capacity/ quantity	Unit rate	Amount US$
1	New WTP (Completed set including CW pumps)	m3/d	17,000	450	7,650,000
2	New Intake and equip with pump				-
	Intake structure	set	0	500,000	-
	Pumps (1 additional duty)	pcs	1	100,000	100,000
3	Feeding pipeline (DCI 400mm)	km	4	125,000	500,000
4	Pipe network	km	40	45,000	1,800,000
5	Network Booster pump(ii)	set	1	90,000	90,000
6	Electrical facilities				-
	At treatment plant	set	0	300,000	-
	At intake	set	0	140,000	-
	At booster pump along network	set	1	140,000	140,000
7	Service and workshop building	set	0	200,000	-
8	Contingency 10%	LS			1,028,000
	Total				**11,308,000**

Note: (ii) only one pump additional to the pump installed in 2025

[12] approximately 8% of the project cost according to ADB at the DFR Workshop)

121. Based on the current facility operations, preliminarily estimates for monthly O&M costs are presented in the the tables below for Year 2025 and 2040.

Table 43. Estimated monthly O&M cost

No.	Description	unit	Production Cost/m^3		Estimated Operating Cost/month	
			(Riel)	(US$)	(2025)	(2040)
1	Electrical	Kwh	313.74	0.078	22,769.99	56,924.99
2	Chemical		139.03	0.035		
	Powder chlorine	kg	16.79	0.004	1,218.55	3,046.38
	Gas Chlorine	kg	12.11	0.003	878.90	2,197.24
	lime	kg	19.45	0.005	1,411.60	3,529.01
	Alum	kg	37.66	0.009	2,733.21	6,833.03
	PAC	kg	53.02	0.013	3,847.98	9,619.95
3	Labour cost per month		209.73	0.052	15,221.36	38,053.41
4	Maintenance cost per month		268.36	0.067	19,476.50	48,691.24
5	Operating Cost of Booster system in the network	Kwh	313.74	0.078	22,769.99	56,924.99
	Total Monthly Production Cost (US$):				**90,328.09**	**225,820.22**

9 Conclusions

122. The Kampong Cham water supply subproject has been proposed with a conventional water treatment plant of design capacity of 11,600m3/d, to serve 85-100% of the urban population (100,117 people) in the expanded service area of area 35km^2 by 2025.

123. A water intake with a capacity of 11,600m3/d is proposed to withdraw raw water from the Mekong River, a 4km transmission line to the WTP, and 80km of distribution network are also included.

124. There are no permits required, except a water source allocation MOU, to be secured from the MoWRM for the raw water extraction from the Mekong River to the proposed WTP. The volume is up to 11,600m3/d by 2025.

125. Land commitment on the WTP and the Intake is confirmed.

126. The construction works of the proposed project will cause various impacts on the surroundings. Most of these impacts are short-term, reversible, localized and are easily mitigated. Negative impacts during construction from pipelaying works will include possible siltation and increased turbidity of nearby receiving bodies of water; generation of construction wastes; construction nuisances (i.e., noise and vibration); and traffic congestion. An environmental management plan is to be developed to mitigate the impacts. Coordination with the contractor and the local traffic management office should be done prior to start of construction, particularly on network extension. A set of environmental code of practices (ECOPs) may also be developed to address common negative impacts of construction works.

127. No specific construction problems with cost implications are expected from existing ground conditions or access.

128. Additional studies before launching a DB contract for WTP include geotechnical, D&B technical performance specifications

129. Detailed design of the distribution network for the Network (Works) Contract will be required.

Annex 1: Service area confirmation

ព្រះរាជាណាចក្រកម្ពុជា
ជាតិ សាសនា ព្រះមហាក្សត្រ

ក្រសួងឧស្សាហកម្ម និង សិប្បកម្ម
មន្ទីរឧស្សាហកម្ម និងសិប្បកម្មខេត្ត
កំ ព ង់ ចា ម
អ ង្គ ភា ព រ ដ្ឋា ក រ ទឹ ក
លេខ.....១១៥..........រកទ.កច

កំពង់ចាម,ថ្ងៃទី ១៩ ខែឧសភាឆ្នាំ ២០១៦

គោរពជូន

ឯកឧត្តមទេសរដ្ឋមន្ត្រី រដ្ឋមន្ត្រីក្រសួងឧស្សាហកម្ម និងសិប្បកម្ម

ជាទីគោរពដ៏ខ្ពង់ខ្ពស់ ។

តាមរយៈ ៖ លោកប្រធានមន្ទីរឧស្សហកម្ម និងសិប្បកម្មខេត្តកំពង់ចាម

កម្មវត្ថុ ៖ សំណើសុំប្រកាសអនុញ្ញាតអចិន្ត្រៃយ៍ ធ្វើអាជីវកម្មផ្គត់ផ្គង់ទឹកស្អាតរបស់អង្គភាពរដ្ឋាករទឹកខេត្តកំពង់ចាម ។

យោង ៖-ប្រកាសលេខ ៤៦១ MIH/2014 ចុះថ្ងៃទី ២៩ ខែឧសភា ឆ្នាំ២០១៤ របស់ក្រសួងឧស្សហកម្ម និងសិប្បកម្ម ។

-លិខិតស្នើសុំការអនុញ្ញាតកំណត់តំបន់ផ្ទៃសេវាកម្ម ផ្គត់ផ្គង់ទឹកស្អាត អង្គភាពរដ្ឋាករទឹកខេត្តកំពង់ចាម របស់មន្ទីរឧស្សហកម្ម និង សិប្បកម្មខេត្ត លេខ ៣៤៧ឧស.កច ចុះថ្ងៃទី ០៤ ខែធ្នូ ឆ្នាំ២០១៥ និង ចំណារឯកភាពលេខ ៥៧៨៨ កច ចុះថ្ងៃទី ០៨ ខែ ធ្នូ ឆ្នាំ ២០១៥ របស់ឯកឧត្តមអភិបាលនៃគណៈអភិបាលខេត្តកំពង់ចាម ។

តបតាមកម្មវត្ថុ និងយោងខាងលើ ខ្ញុំបាទសូមការអនុញ្ញាត គោរពជំរាបជូន **ឯកឧត្តមទេសរដ្ឋមន្ត្រី**មេត្តាជ្រាបដ៏ខ្ពង់ខ្ពស់ថា អង្គភាពរដ្ឋាករទឹកខេត្តកំពង់ចាម បានទទួលនូវជំនួយហិរញ្ញប្បទានឥតសំណងពីរាជរដ្ឋាភិបាលជប៉ុន ក្រោមគម្រោងពង្រីកប្រព័ន្ធផ្គត់ផ្គង់ទឹកស្អាត ខេត្តកំពង់ចាម-បាត់ដំបង ដែលនឹងត្រូវដាក់អោយដំណើរការនៅខែ មិថុនា ឆ្នាំ ២០១៦នេះ ។ អនុលោមតាមប្រកាសលេខ ៤៦១ MIH/2014 ចុះថ្ងៃទី ២៩ ខែឧសភាឆ្នាំ២០១៤ របស់ក្រសួងឧស្សហកម្ម និងសិប្បកម្ម បានតម្រូវអោយគ្រប់អង្គភាពរដ្ឋាករទឹកសាធារណៈ និង ឯកជនទាំងអស់ ត្រូវស្នើសុំប្រកាសអនុញ្ញាត ធ្វើអាជីវកម្មផ្គត់ផ្គង់ទឹកស្អាត ពីក្រសួង ដូចនេះខ្ញុំបាទសូមគោរពស្នើសុំប្រកាសអនុញ្ញាតអចិន្ត្រៃយ៍ពីក្រសួង ស្របតាមនីតិវិធីជាធរមាន ។

អាស្រ័យដូចបានជំរាបជូនខាងលើ សូម**ឯកឧត្តមទេសរដ្ឋមន្ត្រី** មេត្តាពិនិត្យ និងមេត្តាចេញប្រកាសអនុញ្ញាត ដល់អង្គភាពរដ្ឋាករទឹកខេត្តកំពង់ចាម បានដំណើរការស្របតាមនីតិវិធីក្រសួង ដោយសេចក្តីអនុគ្រោះ ។

សូម **ឯកឧត្តម** មេត្តាទទួលនូវសេចក្តីគោរពដ៏ខ្ពង់ខ្ពស់អំពីខ្ញុំបាទ ។

សូមជូនភ្ជាប់មកជាមួយនូវ

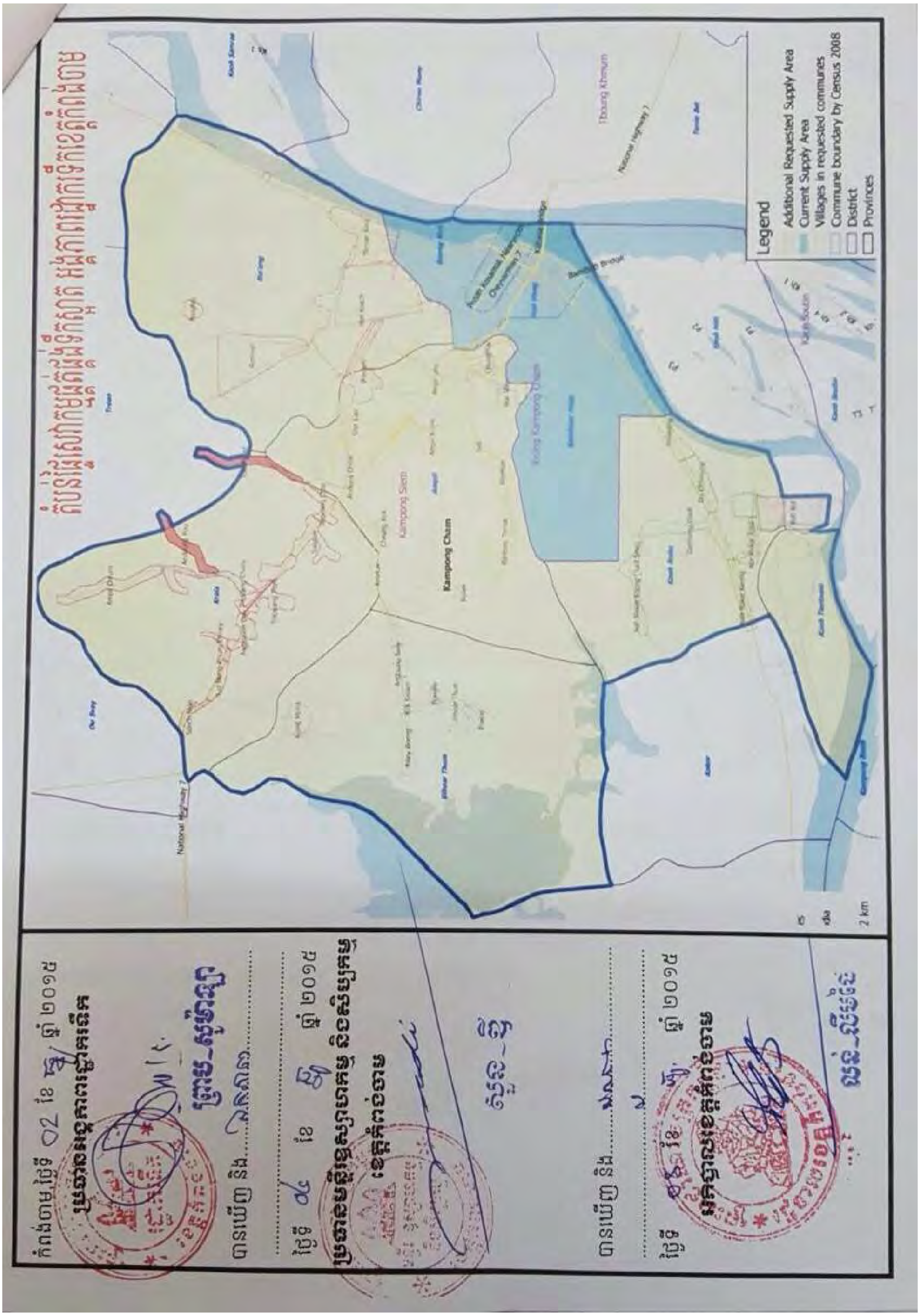
១.ផែនទីតំបន់សេវាកម្មចំនួន...០១ច្បាប់។ ២.សេចក្តីកំណត់ថ្លៃលក់ទឹកចំនួន....០១ច្បាប់។៣.សេចក្តីកំណត់ស្តីពីការយកថ្លៃនាឡិកាទឹកមិនប្រើប្រាស់ និងថ្លៃថែទាំនាឡិកាទឹកចំនួន...០១ច្បាប់ ។ ៤.សេចក្តីកំណត់ថ្លៃភ្ជាប់បណ្តាញចូលផ្ទះចំនួន...០១ច្បាប់ ។

លេខ...១៩៦......ឧស.កច
បានឃើញ និង សូមគោរពជូន
ឯកឧត្តមទេសរដ្ឋមន្ត្រី រដ្ឋមន្ត្រី
ក្រសួងឧស្សាហកម្ម និងសិប្បកម្ម
មេត្តាអនុញ្ញាតដោយសេចក្តីអនុគ្រោះ។
ថ្ងៃទី...១៩......ខែឧសភា...ឆ្នាំ ២០១៦
ប្រធានមន្ទីរឧស្សាហកម្ម និងសិប្បកម្មខេត្ត
កំពង់ចាម

ស៊ុន_ធី

ប្រធានអង្គភាពរដ្ឋាករទឹក

ព្រាប_សុម៉ាឡា


Legend
Additional Requested Supply Area
Current Supply Area
Villages in requested communes
Commune boundary by Census 2008
District
Provinces
Kampong Cham
National Highway 7
2 km

Annex 2: Pump and Electrical Calculations

1. Kampong Cham Sub-project - Proposed Pump and Electrical Design Capacity

A. Intake Pump

i. Capacity of pump intake in 2025

The pump design should include water loss during water treatment processing that is assumed approximately 5% of the total water demand (Q_{total}). Therefore, the water flow rate of pump intake was determined as below:

$$Q_p = Q_{total} \times 1.05 \qquad \textbf{Eq. 1}$$

Where: The total capacity in 2025 is Q_{total} = 11,057.60m^3/day

Therefore, the total capacity of the intake pump station at Kampong Cham is **Qp = 11,600m^3/day**.

And the number of pump unit in 2025 was determined (**1**) duty + (**1**) stand by with capacity of each pump **q_p = 135LPS**.

- **Diameter of suction pipe and discharge pipe**

We have the equation:

$$\phi = \sqrt{\frac{q_p \times 4}{v \times \pi}} \qquad \textbf{Eq. 2}$$

Where:
Φ : pipe diameter, mm
q_p : intake pump capacity of each pump, 0.135m^3/s
V : velocity in pipe (suction pipe and discharge pipe), m/s
π = 3.14

- **Diameter of suction pipe**

According to the **Eq.2**, the diameter of suction pipe is approximately **$\Phi_{suction}$ = 350mm** with **V_s = 1.40m/s**.

- **Diameter of discharge pipe**

According to the **Eq.2**, the diameter of discharge pipe is approximately **Φ_d =300mm** with velocity **V_d = 1.91m/s**.

The main pipe of raw water transmission is proposed **Φ_{main}=350mm** of diameter with **V_{main} = 1.40m/s**.

- **Pump head (H_t)**

The pumping head is the sum of the static head, entrance head, friction loss in the suction pipe, discharge loss, and other band loss as emphasizing in equation below:

$$H_t = H_{static} + h_{ent} + h_{fs} + h_{fd} + \Sigma h_m \qquad \textbf{Eq. 3}$$

Where:

H_t : total pump head, m

H_{static} : static head, m

h_{ent} : head loss of entrance bell, m

h_{fs} : friction loss of suction pipe, m

h_{fd} : friction loss of discharge pipe, m

h_m : friction loss by minors, m

- **Friction loss of suction pipe**

We have:

$$h_{f,s}=10.7\times\left(\frac{q_p}{C}\right)^{1.85}\times D^{-4.87}\times L_s \qquad \textbf{Eq. 4}$$

Where:

q_p : the capacity of each pump, 0.135m^3/s

C : the Hazen-Williams factor, 120

D : the diameter of the suction pipe, 350mm

Ls : the length of the suction pipe (column pipe), 15.21m

Therefore, the friction suction loss is $\mathbf{h_{fs}}$ **= 0.094m**.

- **Velocity suction head ($h_{v,s}$)**

We have:

$$h_{v,s}=\alpha\frac{v^2}{2g} \qquad \textbf{Eq. 5}$$

Where:

$h_{v,s}$: the velocity headloss, m
V : the velocity in suction pipe, 1.40m/s.
g : the earth's gravity, 9.81m2/s
α : velocity factor was assume 1

Therefore, the velocity suction head loss is $\mathbf{h_{vs}}$ **= 0.099m**.

- **Entrance head loss (h_{ent})**

We have:

$$h_{ent}=k\times h_{v,s} \qquad \textbf{Eq. 6}$$

Where: For bell mouth, k = 0.05

Therefore, the entrance headloss is $\mathbf{h_{ent}}$ **= 0.005m**.

- **Gate valve headloss** ($h_{g,v}$)

We have:

$$h_{g,v}=k\times h_{v,s} \qquad \textbf{Eq. 7}$$

Where: For the gate valve, k = 0.2

Therefore, the gate valve headloss is $\mathbf{h_{g,v}}$ **= 0.020m**.

- **Friction loss of discharge pipe (h_{fd})**

We have:

$$h_{f,d}=10.7\times\left(\frac{q_p}{C}\right)^{1.85}\times D^{-4.87}\times L_d \qquad \textbf{Eq. 8}$$

Where:

q_p : the capacity of each pump, 0.135m^3/s
C : the Hazen-Williams factor, 120
D : the diameter of the discharge pipe, 300mm
L_d : the length of discharge pipe (transmission pipe), 1.5m

Therefore, the friction discharge headloss is $\mathbf{h_{f,d} = 0.423m}$.

- **Friction loss of transmission pipe line (h_{ft})**

We have:

$$h_{ft}=10.7\times\left(\frac{q_p}{C}\right)^{1.85}\times D^{-4.87}\times L_t \qquad \textbf{Eq. 9}$$

Where:

q_p : the capacity of each pump, 0.135m^3/s
C : the Hazen-Williams factor, 120
D : the diameter of the main pipe, 350mm
L_t : the length of discharge pipe (transmission pipe), 4,000m

Therefore, the friction discharge headloss is $\mathbf{h_{f,d} = 23.713m}$.

- **Velocity discharge head (h_{vd})**

We have:

$$h_{v,d}=\alpha\frac{V_d^{\,2}}{2g} \qquad \textbf{Eq. 10}$$

Where:

$h_{v,d}$: the velocity headloss, m
V_d : the velocity in discharge pipe, 1.90m/s
g : the earth's gravity, 9.81m^2/s
α : velocity factor was assume 1

Therefore, the velocity discharge headloss is $\mathbf{h_{vd} = 0.184m}$.

- **Gate valve headloss at discharge**

We have:

$$h_{g,v}=k\times h_{v,d} \qquad \textbf{Eq. 11}$$

Where: For the gate valve, k = 0.2

Therefore, the gate valve headloss is $\mathbf{h_{gv} = 0.037m}$.

- **Bends headloss, h_b**

We have:

$$h_b = k \times h_{v,d}$$ **Eq. 12**

Where: We supposed that there are two bends with k = 0.25

Therefore, the bends headloss is $\mathbf{h_b = 0.092m}$.

- **Minor loss check valve (h_{chv})**

We have:

$$h_{chv} = k \times h_{v,d}$$ **Eq. 13**

Where: For check valve, k = 2.20

Therefore, the check valve headloss is $\mathbf{h_{chv} = 0.405m}$.

- **Total dynamic head (Ht)**

We have:

$H_t = H_{static} + h_{ent} + h_{fs} + h_{fd} + \Sigma h_m$

Where: the static head is h_{static} = 16.31m.

⇨ H_t = 16.31 + 0.005 + 0.094 + 24.137 + 0.837 = 41.38m.

Note: there are more bend along main transmission pipe line, we also used the model to run and complete the excel calculation sheet to find the total dynamic head approximately at 40.6m say 41m.

Therefore, the total dynamic head is $\mathbf{H_t}$ **= 41**.0m.

- **Type of pump**

A specialized centrifugal pumps designed to move water from a well or reservoir that is deep underground. Also known a deep well turbine pump or a line shaft turbine pump, it is one of two main types of turbine pumps, which are vertical turbine pumps and submersible turbine pumps.

- **Submersible pumps** have the electric motor located underwater at the bottom of the pump.
- **Vertical turbine pumps** have the motor located above ground, connected via a long vertical shaft to impellers at the bottom of the pump.

We have: H_t = 41m and the type of pump suitable for an installation in a sump with a vertical distance more than 10m just below the pump is a vertical turbine pump.

Therefore, the pump type will be used is type of vertical turbine pump.

ii. Capacity of intake pump for 2040

Same formula of calculation as described above is used to determine the pump capacity in year 2040 with the Q_t = 16,121m3/day.

Therefore, the total pump intake capacity in 2040 is approximately 17,000m3/day. One duty pump will be added in 2040 with capacity 166l/s. The 2025 1 duty + 1 standby pump will be kept for use in parallel. Thus the capacity of each pump will be 166l/s in 2040. The 2025 motor will be upgraded from 100ph to 125ph to handle the 166l/s pump. Additionally, the VFD will be used to help driving the VHS motor (vertical hollow shaft motor). Therefore, the total flow in 2040 will be 332l/s.

- **Diameter of suction pipe**

According to the equation **Eq.2**, the diameter of suction pipe (DIP) is approximately $\Phi_{suction}$ = 350mm with Vs = 1.73m/s.

- **Diameter of discharge pipe**

According to the equation **Eq.3**, the diameter of discharge pipe (DIP) is approximately Φ_d = 300mm with velocity V = 2.35m/s.

The main pipe (DIP) of raw water transmission is supposed Φ_{main}=400mm of diameter with V_d = 1.3m/s

- **Pumping Head**

The pumping head is the sum of the static head, entrance head, friction loss in the suction pipe, discharge loss, and other band loss as emphasizing in equation **Eq.4** to **Eq.12**.

We have:

$H_t = H_{static} + h_{ent} + h_{fs} + h_{fd} + \Sigma h_m$

Where: the static head is h_{static} = 16.31m

⇨ H_t = 16.31 + 0.008 + 0.160 + 18.535 + 1.280 = 35.09m

Note: there are more bend along the main transmission pipe line, we also used the model to run and complete the excel calculation sheet to find that the total dynamic head is approximately 35.90m say 36m.

Therefore, the total dynamic head (H_t) could be derided and resulting H_t = 36.

- **Type of pump**

We have: H_t = 36m > 15m

Therefore, the type of pump was chosen as the same in 2025 that is the vertical turbine pump type.

- **Summary the result of intake pump in KC**

Description	Year 2025	Year 2040
Total pump capacity	135 LPS	197 LPS
Total pump head	Ht = 41 m	Ht =36 m
Pump type	Vertical Turbine	Vertical Turbine
Number of pump	1 in duty and 1 stand by, capacity 135LPS/1pump	1 duty, capacity 166 LPS/1 pump (in addition to 2025)
Total power required	75kw	180kw
Column pipe (DIP)	Φ350 mm	Φ350mm
Discharge pipe (DIP)	Φ300mm	Φ300mm
Main pipe (HDPE)	Φ350mm, L = 4km	Φ400mm, L = 4km

B. Booster pump at water treatment plant (WTP) in 2025

The booster pump is the type of pump commonly used for distributing potable water to the consumers.

i. Capacity of distribution pump at WTP

The capacity of the pump should include peak hour factor that vary depend on area (for example, peak hour factor for urban area is normally less than peak factor for rural area). Therefore, the capacity of the booster pump for 2025 is **226LPS**. The number of pump was selected (**2) duty** pumps and (**1)** stand by with the capacity of each pump is **113LPS**.

ii. Pump head

The total head depends on criteria design such as head loss along pipe (leaking and friction), different level from WTP to end of pipe network, and pressure requirement. Therefore, the total head for booster pump at the WTP is **H = 36m**.

C. Booster Pump at WTP in 2040

The booster pump is the type of pump commonly used for distributing potable water to the consumers.

i. Capacity of distribution pump at WTP

The capacity of the booster pump in 2040 is **299 L/S**. Two (**2) additional duty** pumps will be added in 2040 and the other 2 duty pumps and 1 standby pump of 2025 will still be used in 2040, assuming that the motor will be upgraded. Therefore, the total number of pump is 5 pumps in which 4 duties + 1 standby with capacity of each pump in 2040 being 131LPS, so that the total pump capacity in 2040 is 525LPS. The pumps will not change capacity, but the motors will be upgraded from 75ph (55kw) in 2025 to 100ph (75kw) in 2040. Thus, the total power consumption in 2025 is 110kw and in 2040 is 300kw for 4 pumps. Moreover, the VFD will used to drive VHS motor.

ii. Pumping head

The total head depends on design criteria such as head loss along pipe (leaking and friction), different level from WTP to end of pipe network, and pressure requirement. Therefore, the total head for booster pump at the WTP is **H = 36m**.

- **Summary of the booster pump at the WTP in 2025 and 2040**

Description	Year 2025	Year 2040
Total pump capacity	226 LPS	299 LPS
Total pump head	Ht = 36m	Ht = 36m
Pump type	Vertical in-line single stage	Vertical in-line single stage
Number of pump	2 in duty and 1 stand by, capacity 114 LPS/1 pump	2 in duty, capacity 131 LPS/1 pump
Total power required	110 kw	75kw

D. Pump *for* distribution along network for 2025

According to the land issue and engineering concept, other booster pumps along the pipe line have been proposed for this project. The capacity of the pump along the pipe line for 2025 is 136 **LPS**. The number of pump selected is (**2)** duty pumps and (**1)** stand by with the capacity of each pump being **68 LPS**. The total pump head was determined to be around **29m**.

E. Pump for distribution along network for 2040

According to the land issue and engineering concept, other booster pumps along the pipe line have been proposed for this project. The capacity of the pump along the pipe line for 2040 is **48 LPS**. The number of pump was selected is (**1)** duty pump. The total pump head was determined to be around **33m**.

- **Summary**

Description	Year 2025	Year 2040
Total pump capacity	136 LPS	48 LPS
Total pump head	Ht = 29m	Ht = 33m
Pump type	Vertical in-line single stage	Vertical in-line single stage
Number of pump	2 in duty and 1 stand by, and capacity 68 LPS/pump	1 in duty with capacity 48 LPS
Total Power required	60kw	30kw

Note: The pump in 2040 is the same pump as in 2025. Therefore, the total pump is 3 duties + 1 standby with pump capacity ***being*** 68lps. The 2 duty ***pumps*** will be running in full load (for flow 68lps) and another duty will be running for flow 48lps; for this case, VFD will be used to control the flow.

F. Sump drainage pump for 2025 & 2040

The capacity for pump drainage for both 2025 and 2040 have been determined to be approximately **20 LPS** or **72m^3/h**. The number of pump selected is (**1**) duty pumps. The total pump head is about **H_t = 22m**. The diameter of discharge pipe is **Φ = 100mm**. The power required is **15kw**. Sump pump will be utilized only during maintenance/dewatering of the sump at the intake. Thus, the capacity will not change.

Annex:

<u>The vibration frequencies drive (VFD)</u>:

A better approach for maintaining the desired water pressure is to regulate the speed of the pump motor with a variable frequency drive (VFD). VFDs control the speed of AC induction motors by controlling the frequency and voltage supplied to the motor. While mechanical devices can be installed at a pump's output to adjust flow, a VFD regulates flow by adjusting the motor/pump speed. This approach is much better for a number of reasons:

- Uses a minimum amount of energy
- Reduces motor starting current
- Provides a degree of motor protection
- Cuts the wear and tear on the motor
- Simplifies the design of the flow control system
- Provides extensive diagnostics
- Reduces the required maintenance

When a VFD is used, the controlled motor's efficiency is optimized and runs at maximum efficiency regardless of the required flow and corresponding motor/pump speed. The VFD input current rises linearly with respect to output power because the VFD can slowly ramp the pump up to speed. As a result, the typical six to seven times motor rated current seen with an across-the-line started motor is nonexistent with a VFD.

As a result, the negative impact of frequent start/stop cycles is greatly reduced because the VFD limits the motor's inrush current, which and prevents the motor's thermal rises that are inherent with across-the-line starting.

Basic VFDs provide phase loss detection and motor thermal overload protection. Advanced pump-specific VFDs offer features such as loss of prime detection, detection of a pump in a no-flow (deadhead) condition, low/high-pressure level detection, broken pipe protection and pump over cycle protection. These factors make VFDs superior to mechanical devices for regulating pump flow, and other features of VFDs allow direct implementation in complex pump control applications.

Annex 3: Pipe calculations

I. Piped Distribution Network

Pipe water supply shall be installed along the road where priority for potential household. We design pipe distribution network to supply in extension area and also to fill up the existing area for the project duration.

It base on:

1. Data of population in each area and population grow.
2. Elevation profile of existing ground.
3. Water demand that include water consumption from domestic, commercial place, institutional place etc.

- Criteria design
    - Type of distribution system by using booster pump and water tower.
    - Water consumption in 2025 is 150 Ltr/pop/d and in 2040 is 160 Ltr/pop/d
    - The minimum residual pressure at the end of network is around 8 mH2O.
    - The water velocity range from 0.3 to 1.65 m/s to avoid particles sedimentation inside the pipe.
    - Assume Technical Water loss 15% of total demand.
    - Population grow base on each area

      Formula: $\boldsymbol{P_t = P_0 * (1+e)^t}$

      Where: P_t: Future population for project duration

      P_o: Population in current year/ *starter year*

      e : Population grow (%)

      t : Project *duration* (*year*)
    - The roughness calculation is based on Hazen-William formula

      Formula $\boldsymbol{V = kCAR^{0.63}S^{0.54}}$

      Where V: *discharge in section* (m/s)

      C: *roughnedd coefficient*

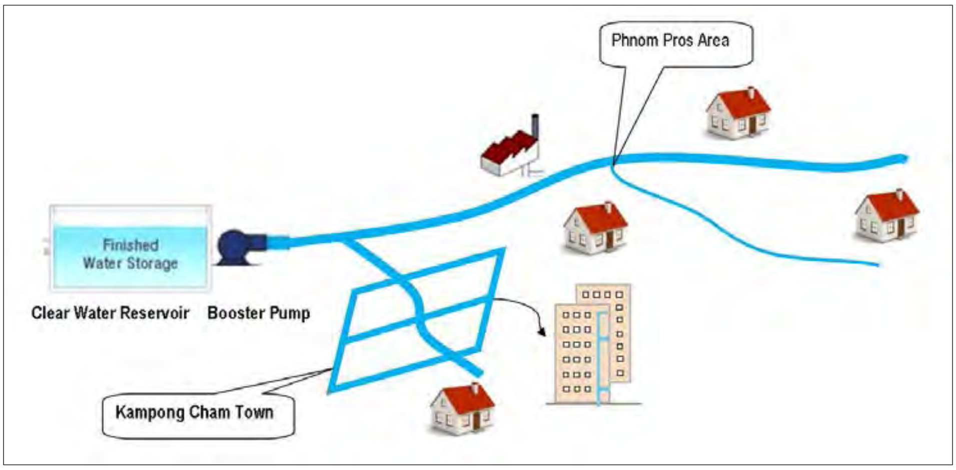
      R: *hydaulic radius* (m)

      S: *friction slope* (m/m)

      k: *factor depent on unit*
    - The pipe diameter follow catalogue of HDPE which coefficient roughness C=150.
    - Water demand that estimate normally in average water consumption. In practice it will be increase whenever they use at the same time. Peak hour 1.77 in year 2025 and 1.67 in year 2040.
    - Daily peak factor is 1.2 (k')
    - Total Water demand (TWD) for each node to design piped network in hourly peak to calculated by demand of each node (DN) multiple with hourly peak factor (k) and daily peak factor (k')

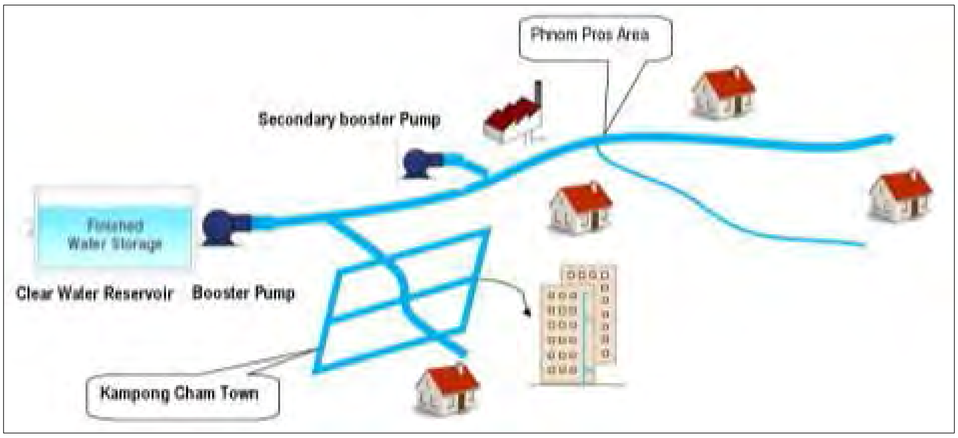
      Formula $WD = DN \times k \times k'$
    - Air valve and drained will provided in higher and lower of the existing ground level.
    - Distribution Meter Area (DMA) will be provided for main block in extension area and connection to existing pipeline.
    - There have three options for pipe network design:
        - **Option 1:** We use booster from water treatment plant to distribute directly to consumption place (household, institutional building, commercial, industrial place…)
        - **Option 2:** We use booster from the Water treatment plant to distribute along the pipeline and we need additional pump along the network to get enough pressure at end of piped network.
        - **Option 3:** We use booster from the Water treatment plant to distribute along the pipeline and we need additional pump along the network to convey the water into water tower near Phnom Pros area.

Figure 1: Option 1_Schema of distribution pipe network (Pump only)



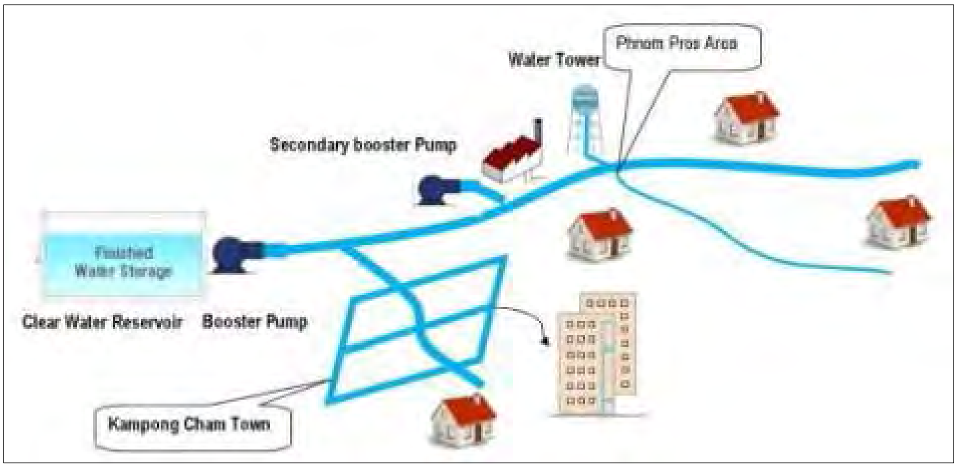
Option 1: we use booster pump to distribute the water for 24 hours and the total head will vary depend on the consumption of water's user.

Figure 2: Option 2_Schema of distribution pipe network (Pump +Pump)



Option 2: we have used a booster pump to distribute the water for 24 hours and the total head will vary depend on the consumption of water's user. In this option there are two clear water pumping stations, one at the water treatment plant and the second along the network.

Figure 3: Option 3_Schema of distribution pipe network (Pump +Pump + WT)



Option 3: we use the booster pump and two water towers for distribution 24 hours. In this option, we install the water tower to stock the water from the pipeline whenever the consumptions less than normal users or the residual pressure can allow the water into the tanks. And it will continue to distribute the water in the downstream area.

II. Data input

Table 1: Estimation of Water Demand in 2025

No.	Commune	Village	Pop_Total	Coverage (%)		Est_pop	Pop_growth_rate	Pop_design in	Unit Water cons.	Water deamand (Ltr/d)		Water loss	Total_water deamand		
				Domestic	Big Cons.		(%)	2025	(Ltr/Pop/d)	Dosmectic	Big Cons.	(%)	(Ltr/d)	(m3/d)	(m3/hour)
1	Krala	Andoung Pou	749	85%	-	637	2.50	815	150	122,245	-	15%	140,582	140.58	5.86
2		Ampil Chrum	884	85%	-	751	2.50	962	150	144,278	-	15%	165,920	165.92	6.91
3		Angkuonh Dei	674	90%	8.5%	607	2.50	776	150	116,475	9,900	15%	145,332	145.33	6.06
4		Thmei	677	90%	-	609	2.50	780	150	116,993	-	15%	134,542	134.54	5.61
5		Tuol Popel	859	85%	-	730	2.50	935	150	140,198	-	15%	161,228	161.23	6.72
6		Trakuon	1179	100%	11.5%	1179	2.50	1,509	150	226,383	26,034	15%	290,280	290.28	12.09
7		Trapeang Char	925	90%	13.4%	833	2.50	1,066	150	159,851	21,420	15%	208,461	208.46	8.69
8		Trapeang Chrey	1162	80%	8.0%	930	2.50	1,190	150	178,495	14,280	15%	221,691	221.69	9.24
9		Trapeang Ruessei	692	90%	16.0%	623	2.50	797	150	119,585	19,134	15%	159,527	159.53	6.65
10		Sdach Non	452	100%	-	452	2.50	579	150	86,790	-	15%	99,808	99.81	4.16
11		Trapeang Thma	649	96%	-	623	2.50	798	150	119,632	-	15%	137,576	137.58	5.73
12		Trapeang Tras	830	90%	12.0%	747	2.50	956	150	143,433	17,212	15%	184,742	184.74	7.70
13		Tuol Beng	986	100%	10.0%	986	2.50	1,262	150	189,325	18,932	15%	239,495	239.50	9.98
14	Ampil	Ampil Kraom	1342	85%	16.0%	1141	2.00	1,391	150	208,576	33,372	15%	278,240	278.24	11.59
15		Ampil Leu	4733	55%	1.0%	2603	2.00	3,173	150	475,984	4,760	15%	552,855	552.86	23.04
16		Andoung Chraoh	2776	100%	14.5%	2776	2.00	3,384	150	507,589	73,600	15%	668,368	668.37	27.85
17		Cheung Kouk	718	90%	4.3%	646	2.00	788	150	118,157	5,022	15%	141,656	141.66	5.90
18		Krala	1554	90%	8.3%	1399	2.00	1,705	150	255,733	21,098	15%	318,355	318.36	13.26
19		Sralau	903	100%	2.0%	903	2.00	1,101	150	165,113	3,302	15%	193,677	193.68	8.07
20	Ro-Ang	Prasam	2049	90%	-	1844	2.00	2,248	150	337,192	-	15%	387,771	387.77	16.16
21		Thma Koul	2526	60%	4.0%	1516	2.00	1,848	150	277,126	11,085	15%	331,443	331.44	13.81
22		Veal Khsach	1182	85%	-	1005	2.00	1,225	150	183,709	-	15%	211,265	211.26	8.80
		Total	28,501			23,538		29,286	3,300	4,392,862	279,152		5,372,815	5,373	223.87

Table 2: Estimation of Water Demand in 2040

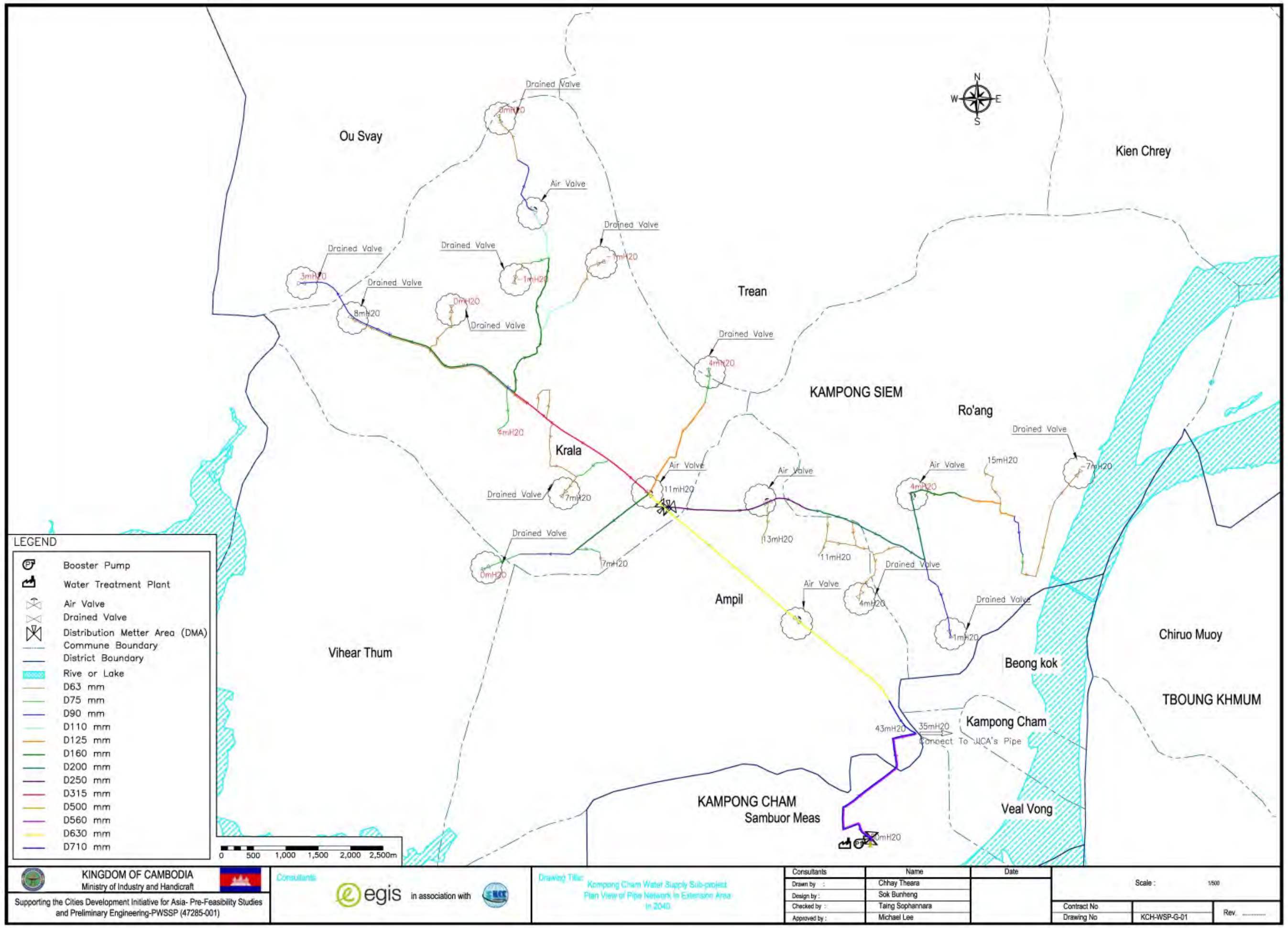
No.	Commune	Village	Pop_Total	Coverage (%)		Est_pop	Pop_growth_rate	Pop_design in	Unit Water cons.	Water deamand (Ltr/d)		Water loss	Total_water deamand		
				Dosmectic	Big Cons.		(%)	2040	(Ltr/Pop/d)	Dosmectic	Big Cons.	(%)	(Ltr/d)	(m3/d)	(m3/hour)
1	Krala	Andoung Pou	749	85%	-	637	2.50	1,180	160	188,850	-	15%	217,178	217.18	9.05
2		Ampil Chrum	884	85%	-	751	2.50	1,393	160	222,889	-	15%	256,322	256.32	10.68
3		Angkuonh Dei	674	90%	7.4%	607	2.50	1,125	160	179,936	13,383	15%	222,317	222.32	9.26
4		Thmei	677	90%	-	609	2.50	1,130	160	180,737	-	15%	207,848	207.85	8.66
5		Tuol Popel	859	85%	-	730	2.50	1,354	160	216,585	-	15%	249,073	249.07	10.38
6		Trakuon	1179	100%	10.9%	1179	2.50	2,186	160	349,728	38,252	15%	446,176	446.18	18.59
7		Trapeang Char	925	90%	13.0%	833	2.50	1,543	160	246,945	32,196	15%	321,012	321.01	13.38
8		Trapeang Chrey	1162	80%	7.1%	930	2.50	1,723	160	275,748	19,647	15%	339,705	339.70	14.15
9		Trapeang Ruessei	692	90%	14.0%	623	2.50	1,155	160	184,742	25,864	15%	242,197	242.20	10.09
10		Sdach Non	452	100%	-	452	2.50	838	160	134,077	-	15%	154,189	154.19	6.42
11		Trapeang Thma	649	96%	-	623	2.50	1,155	160	184,813	-	15%	212,535	212.53	8.86
12		Trapeang Tras	830	90%	10.5%	747	2.50	1,385	160	221,583	23,266	15%	281,577	281.58	11.73
13		Tuol Beng	986	100%	8.8%	986	2.50	1,828	160	292,478	25,592	15%	365,781	365.78	15.24
14	Ampil	Ampil Kraom	1342	85%	12.0%	1141	2.00	1,871	160	299,430	35,932	15%	385,666	385.67	16.07
15		Ampil Leu	4733	55%	1.0%	2603	2.00	4,271	160	683,319	6,833	15%	793,675	793.67	33.07
16		Andoung Chraoh	2776	100%	12.8%	2776	2.00	4,554	160	728,692	92,908	15%	944,840	944.84	39.37
17		Cheung Kouk	718	90%	4.3%	646	2.00	1,060	160	169,626	7,209	15%	203,360	203.36	8.47
18		Krala	1554	90%	8.3%	1399	2.00	2,295	160	367,128	30,288	15%	457,029	457.03	19.04
19		Sralau	903	100%	2.0%	903	2.00	1,481	160	237,035	4,741	15%	278,042	278.04	11.59
20	Ro-Ang	Prasam	2049	90%	-	1844	2.00	3,025	160	484,071	-	15%	556,681	556.68	23.20
21		Thma Koul	2526	60%	3.5%	1516	2.00	2,487	160	397,840	13,924	15%	473,530	473.53	19.73
22		Veal Khsach	1182	85%	-	1005	2.00	1,648	160	263,731	-	15%	303,290	303.29	12.64
		Total	**28,501**			**23,538**		**40,687**	**3,520**	**6,509,984**	**370,034**		**7,912,021**	**7,912**	**329.67**

III. Result

We use Epanet software to design diameter of pipeline which follow to design criteria.

III.1. Result of option 1 (Use Only Booster Pump)

Figure 4: Layout plant- Piped Network of Kampong Cham In Extension Area In 2040



According to result above:

- The elevation difference between a water treatment plant and the highest level in Phnom Pros area is 35.16m.
- To distribute the water at the downstream of extension area need the residual pressure at least 16mH2O at highest of the Phnom pros area in the year 2040.
- The maximum of total head of booster pump is 50m it cannot increase more due to it will effect to other accessory of water users near this water supply station, affected to the existing piped network(pipe characteristic not respond to high pressure)…
- Main pipe diameter could not increase more according to the velocity will be smaller than criteria design in average time in the year 2040 and also in the year 2025. (Please find it in table of node and pipe description)
- We recommend using secondary pumping station along the network to convey the water to distribute at downstream area.

III.2. Result of option 2 (Pump + Pump)

Figure 5: Layout Plant-Piped Network of Kampong Cham In Extension Area in 2025

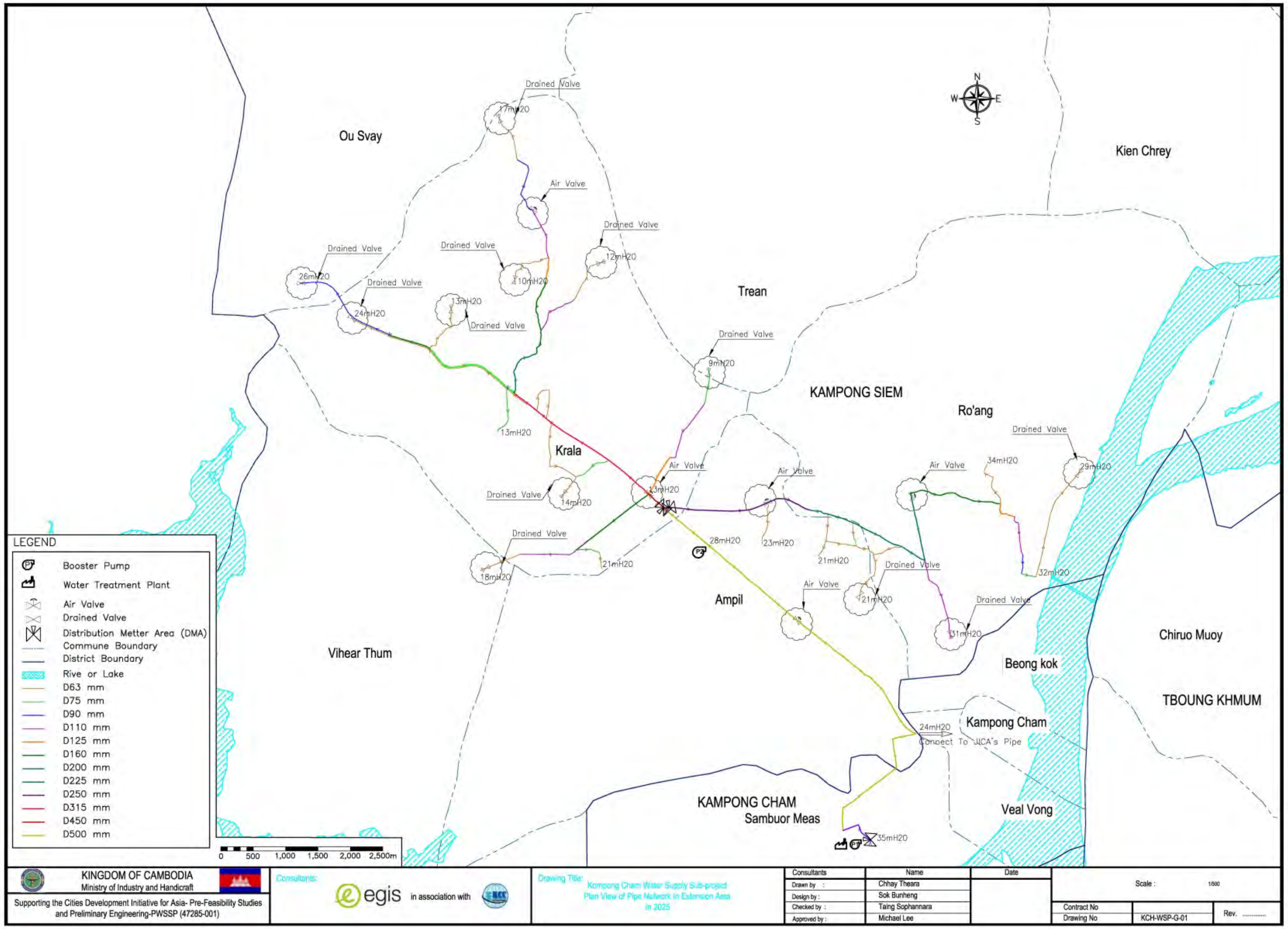


Figure 6: Layout plant- Piped Network of Kampong Cham In Extension Area In 2040

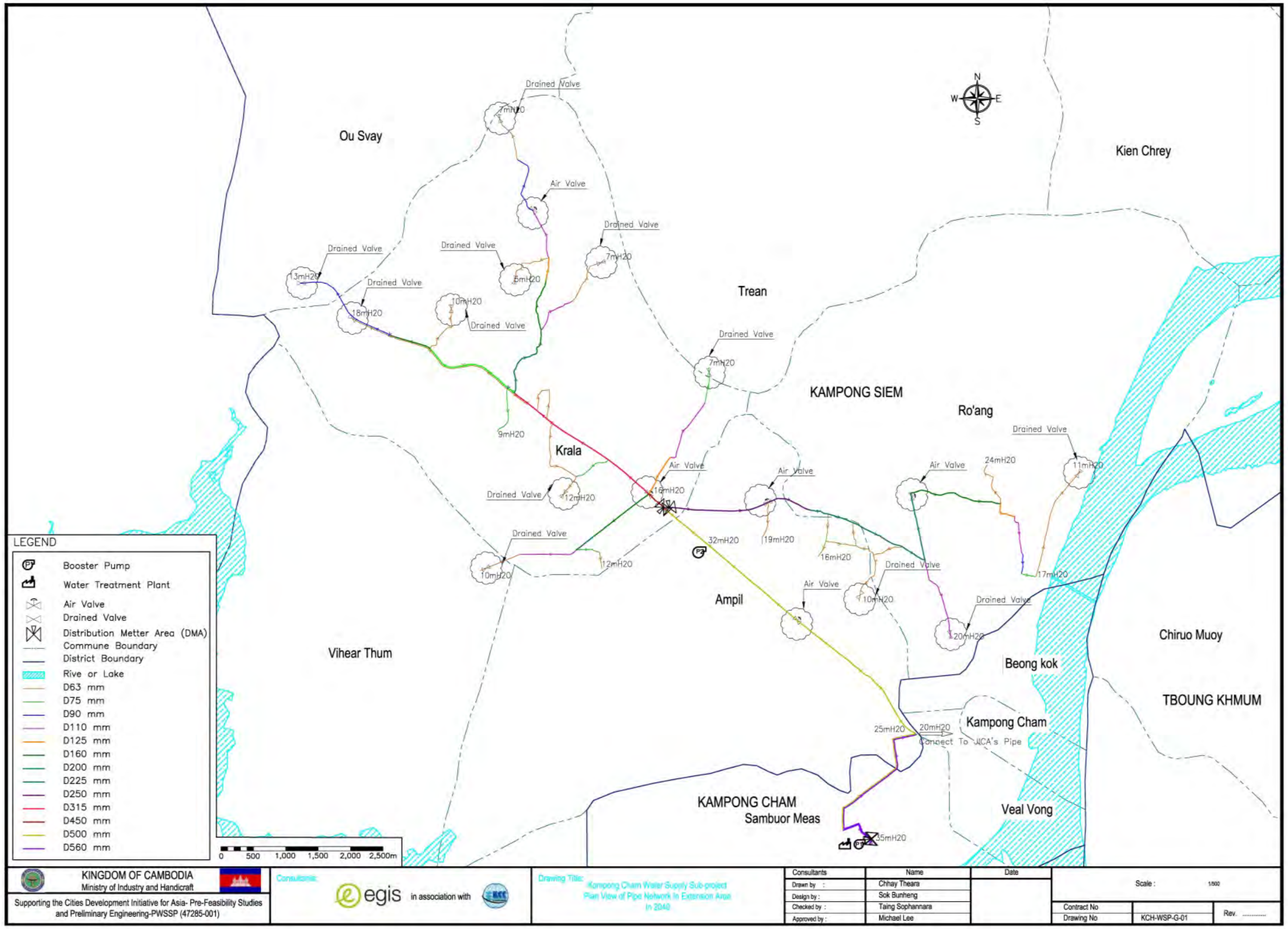


Table 3: Summaries data of accessory for main fitting in network

No.	Description	Unit	Quantity
1	**Distribution Meter Area (DMA)**		
1.1	DMA (HDPE) Ø560 mm	pcs	2
1.2	DMA (HDPE) Ø450 mm	pcs	1
1.3	DMA (HDPE) Ø250 mm	pcs	1
2	**Drained Valve**		
2.1	Drained Valve (HDPE) Ø63 mm	pcs	9
2.2	Drained Valve (HDPE) Ø75 mm	pcs	1
2.3	Drained Valve (HDPE) Ø90 mm	pcs	1
2.4	Drained Valve (HDPE) Ø110 mm	pcs	1
3	**Air Valve**		
3.1	Air Valve (HDPE) Ø500 mm	pcs	1
3.2	Air Valve (HDPE) Ø450 mm	pcs	1
3.3	Air Valve (HDPE) Ø250 mm	pcs	1
3.4	Air Valve (HDPE) Ø200 mm	pcs	1
3.5	Air Valve (HDPE) Ø110 mm	pcs	1

Table 4: Summaries data for facility of Pipe Network

No.	Summaries for Kampong Cham pipe water supply		
I.	**Pipe Diameter (mm) (*1)**	**Pressure Normal (PN)**	**Pipe Length (m)**
1	63	PE100,PN 10	16,213.00
2	75	PE100,PN 10	2,485.00
3	90	PE100,PN 10	3,006.00
4	110	PE100,PN 10	5,394.00
5	125	PE100,PN 8	1,407.00
6	160	PE100,PN 8	4,736.00
7	200	PE100,PN 8	3,842.00
8	225	PE100,PN 8	1,576.00
9	250	PE100,PN 8	2,324.00
10	315	PE100,PN 8	2,605.00
11	450	PE100,PN 8	354.00
12	500	PE100,PN 8	7,631.00
13	560	PE100,PN 8	606.00
A. Sub-total -1			**52,179.00**
II.	**Additional pipe after year 2025**	**Pressure Normal (PN)**	**Pipe Length (m)**
14	560mm	PE100,PN 8	2,789.00
B. Sub-total -2			**2,789.00**
Grand Total (I+II)			**54,968.00**
III.	**Total Pump Capacity in 2025**	**Flow (m3/h)**	**Head (m)**
15	At Water Treatment Plant	815	35
16	Booster pump along the PNW	488	28
IV.	**Total Pump Capacity Total in 2040**	**Flow (m3/h)**	**Head (m)**
17	At Water Treatment Plant	1,891	35
18	Booster pump along the PNW	662	32

Note: (*1) Pipe diameter here follow pipe catalogue.

According to Table result above:

- Diameter of pipeline was design for total demand in 2040 in peak hour.
    - **From 2015 to 2025,** the pipeline from the water treatment plant needs one pipeline along the road to supply for extension area and the current existing area. It needs 327m³/h (Total 815 m³/h for peak demand).
    - **After 2025**, the additional pipeline need to be installed one more pipeline along the road in parallel with the existing, it connect with JICA's pipeline around 2,789 m (see Figure below). It needs more 1,231 m³/h (Total 1,891 m³/h for peak demand) to supply at the existing area in the city.
- The capacity of booster pump need to be design in peak demand which device in 2 phases:
    - **Phase 1**: From 2015 to 2025, the capacity of booster pump at water treatment plant is 815m³/h; total head 35 mH2O. The capacity of booster pump along the pipeline is 488 m³/h; Total head 28 mH2O.
    - **Phase 2**: After 2025 to 2040, the capacity of booster pump at water treatment plant is 1,891 m³/h; Total head 35 mH2O. And the capacity of booster pump along the pipeline is 662 m³/h; Total head 32 mH2O.

III.1. Result of option 3 (Pump + Pump + Water Tower)

Figure 7: Layout Plant-Piped Network of Kampong Cham In Extension Area in 2025

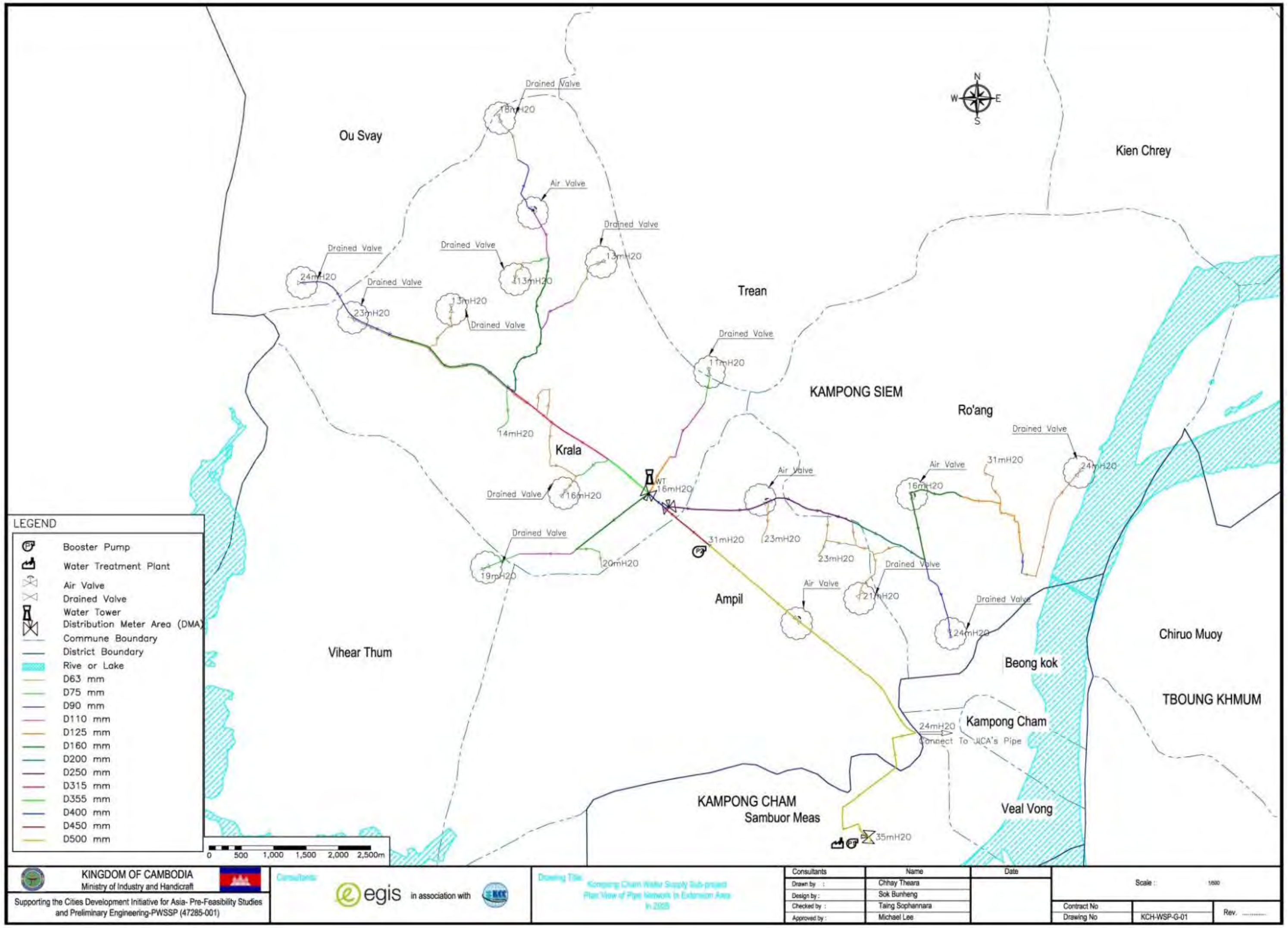


Figure 8: Layout plant- Piped Network of Kampong Cham In Extension Area In 2040

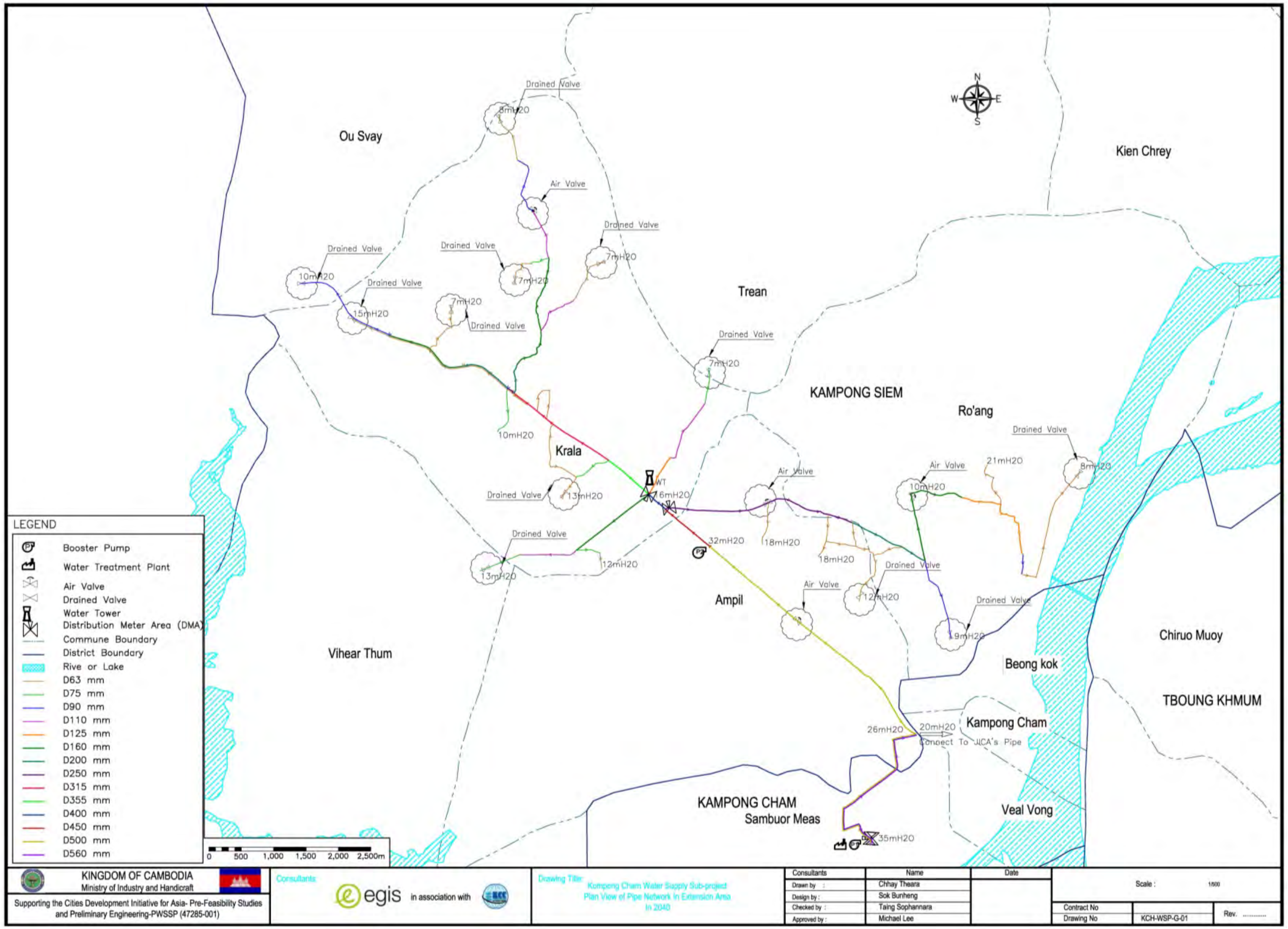


Table 5: Summaries data of accessory for main fitting in network

No.	Description	Unit	Quantity
1	**Distribution Meter Area (DMA)**		
1.1	DMA (HDPE) Ø560 mm	pcs	1
1.2	DMA (HDPE) Ø500 mm	pcs	1
1.2	DMA (HDPE) Ø450 mm	pcs	1
1.3	DMA (HDPE) Ø250 mm	pcs	1
2	**Drained Valve**		
2.1	Drained Valve (HDPE) Ø63 mm	pcs	8
2.2	Drained Valve (HDPE) Ø75 mm	pcs	2
2.3	Drained Valve (HDPE) Ø90 mm	pcs	2
3	**Air Valve**		
3.1	Air Valve (HDPE) Ø500 mm	pcs	1
3.2	Air Valve (HDPE) Ø250 mm	pcs	1
3.3	Air Valve (HDPE) Ø160 mm	pcs	1
3.4	Air Valve (HDPE) Ø110 mm	pcs	1

Table 6: Summaries data for facility of Pipe Network

No.	Summaries for Kampong Cham pipe water supply		
I.	**Pipe Diameter (mm) (*1)**	**Pressure Normal (PN)**	**Pipe Length (m)**
1	63	PE100,PN 10	15,477.00
2	75	PE100,PN 10	3,221.00
3	90	PE100,PN 10	4,316.00
4	110	PE100,PN 10	3,445.00
5	125	PE100,PN 8	2,297.00
6	160	PE100,PN 8	6,552.00
7	200	PE100,PN 8	2,542.00
8	250	PE100,PN 8	3,132.00
9	315	PE100,PN 8	1,793.00
10	355	PE100,PN 8	812.00
11	400	PE100,PN 8	355.00
12	450	PE100,PN 8	1,059.00
13	500	PE100,PN 8	7,178.00
A. Sub-total -1			**52,179.00**
II.	**Additional pipe after year 2025**	**Pressure Normal (PN)**	**Pipe Length (m)**
14	560mm	PE100,PN 8	2,789.00
B. Sub-total -2			**2,789.00**
Grand Total (I+II)			**54,968.00**
III.	**Total Pump Capacity in 2025**	**Flow (m3/h)**	**Head (m)**
15	At Water Treatment Plant	815	35
16	Booster pump along the PNW	488	31
IV.	**Total Pump Capacity Total in 2040**	**Flow (m3/h)**	**Head (m)**
17	At Water Treatment Plant	1,891	35
18	Booster pump along the PNW	662	32
V.	**Water Tower**	**Volume (m³)**	**Height bottom of the tank (m)**
19	Water Tower near Phnom Pros (supply around 1 hour for hourly peak) (*2)	365	16

Note : (*1) Pipe diameter here follow pipe catalogue.
(*2) The location of water tower will be built a t leg of Phnom Pros Area. Where the nature ground is 47.57m (m.s.l)

According to Table result above:

- Diameter of pipeline was design for total demand in 2040 in peak hour.
    - **From 2015 to 2025,** the pipeline from the water treatment plant needs one pipeline along the road to supply for extension area and the current existing area. It needs 327m³/h (Total 815 m³/h for peak demand).
    - **After 2025**, the additional pipeline need to be installed one more pipeline along the road in parallel with the existing, it connect with JICA's pipeline around 2,789 m (see Figure below). It needs more 1,231 m³/h (Total 1,891 m³/h for peak demand).
- The capacity of booster pump need to be design in peak demand which device in 2 phases:
    - **Phase 1**: From 2015 to 2025, the capacity of booster pump at water treatment plant is 815m³/h; total head 35 mH2O. The capacity of booster pump along the pipeline is 488 m³/h; Total head 31 mH2O.
    - **Phase 2**: After 2025 to 2040, the capacity of booster pump at water treatment plant is 1,891 m³/h; Total head 35 mH2O. And the capacity of booster pump along the pipeline is 662 m³/h; Total head 32 mH2O.
- The volume of water tower in Phnom pros area is 365 m³ with the height bottom of the tanks is 16 m. It is supply for 24 hours.

IV. Data from EPANET

IV.1. Option 1: Use Only Booster Pump

Figure 9: Network Map

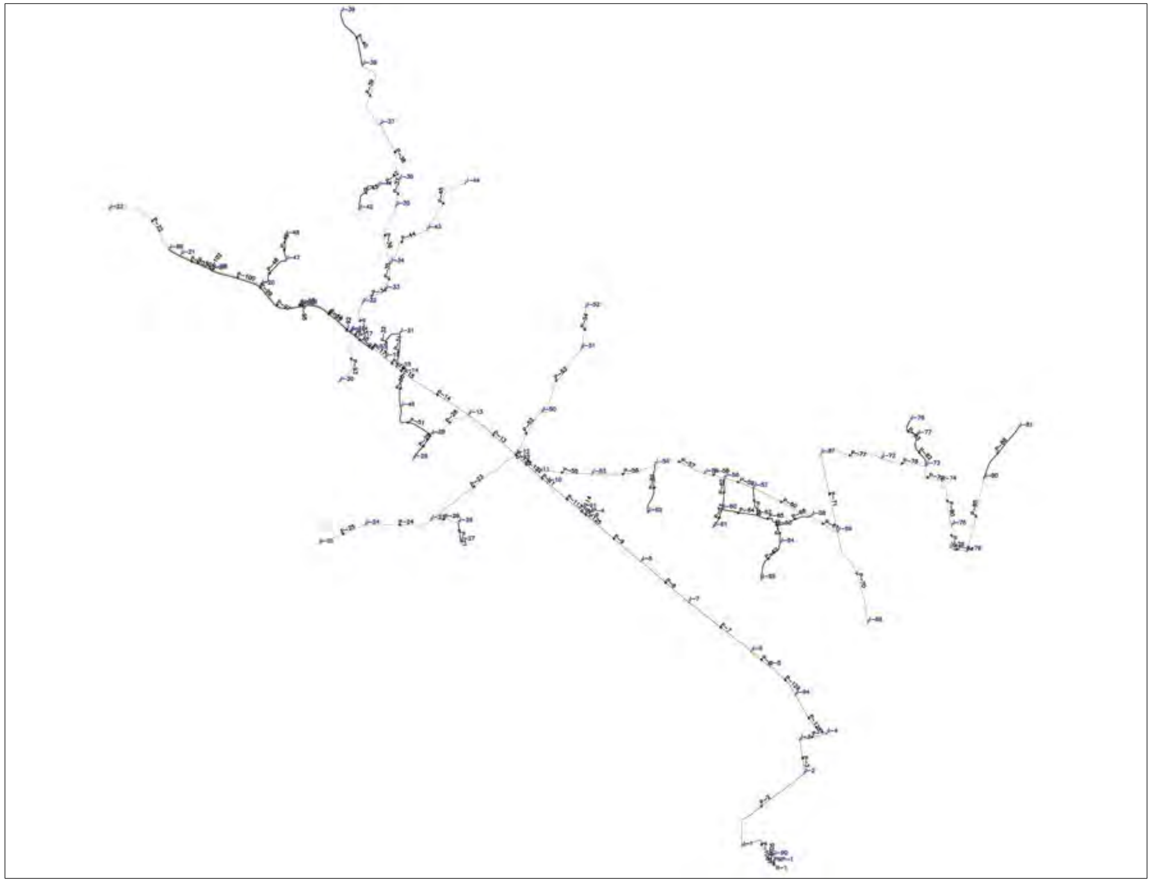


Table 7: Node data in 2025

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-1	13.95	0.0	48.00	48.00
J-2	14.01	0.0	47.00	48.00
J-3	14.95	0.0	46.00	47.00
J-4	17.95	4572.3	43.00	44.00
J-5	17.05	0.0	44.00	45.00
J-6	27.97	303.3	33.00	34.00
J-7	29.97	303.3	31.00	32.00
J-8	32.14	173.7	28.00	30.00
J-9	33.44	372.4	27.00	28.00
J-10	41.82	210.0	18.00	20.00
J-11	45.14	132.2	15.00	16.00
J-12	47.57	306.7	13.00	14.00
J-13	46.85	235.9	12.00	14.00
J-14	45.98	103.7	12.00	15.00
J-15	45.38	43.2	12.00	15.00
J-16	43.22	57.0	14.00	17.00
J-17	44.56	72.6	12.00	16.00
J-18	44.44	76.0	12.00	16.00
J-19	43.14	67.4	12.00	17.00
J-20	41.74	91.6	12.00	17.00
J-21	34.46	131.3	13.00	23.00
J-22	26.07	92.4	19.00	30.00
J-23	32.45	191.8	23.00	28.00
J-24	30.37	148.6	18.00	27.00
J-25	36.03	58.8	11.00	21.00
J-26	33.90	95.0	16.00	24.00
J-27	31.40	81.2	17.00	26.00
J-28	47.42	85.5	9.00	13.00
J-29	44.03	25.9	12.00	16.00
J-30	44.59	62.2	10.00	15.00
J-31	42.65	38.9	15.00	18.00
J-32	44.83	84.7	11.00	15.00
J-33	44.08	73.4	10.00	16.00
J-34	44.64	63.9	9.00	15.00
J-35	44.63	15.6	9.00	15.00
J-36	44.42	44.9	9.00	15.00
J-37	44.25	58.8	7.00	14.00
J-38	37.26	70.0	12.00	21.00
J-39	34.33	35.4	13.00	23.00
J-41	44.26	34.6	8.00	15.00
J-42	44.10	20.7	8.00	15.00
J-43	43.54	69.1	10.00	16.00
J-44	43.34	34.6	8.00	15.00
J-47	42.98	26.8	10.00	16.00
J-48	43.69	4.3	9.00	15.00
J-49	45.40	16.4	11.00	15.00
J-50	56.42	59.6	3.00	5.00
J-51	55.25	97.6	3.00	6.00
J-52	48.67	62.2	8.00	12.00
J-53	43.13	82.1	16.00	18.00
J-54	46.18	131.3	11.00	14.00
J-55	45.60	28.5	11.00	15.00
J-56	44.55	30.2	11.00	15.00

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m^3/day)	Peak Demand	Average Demand
J-57	40.36	30.2	13.00	19.00
J-58	31.93	20.7	19.00	27.00
J-59	27.15	259.2	23.00	31.00
J-60	35.14	30.2	17.00	24.00
J-61	34.41	7.8	18.00	24.00
J-62	35.54	30.2	16.00	23.00
J-63	32.47	35.4	17.00	25.00
J-64	31.90	45.8	16.00	25.00
J-65	30.53	38.0	16.00	26.00
J-66	19.69	219.5	19.00	34.00
J-67	37.09	46.7	13.00	21.00
J-72	24.43	14.7	24.00	33.00
J-73	20.62	51.8	25.00	36.00
J-74	18.83	105.4	26.00	37.00
J-75	15.62	87.3	24.00	39.00
J-76	15.54	34.6	22.00	38.00
J-77	19.75	14.7	26.00	37.00
J-78	18.18	7.8	27.00	38.00
J-79	17.08	32.0	19.00	36.00
J-80	17.54	32.0	15.00	35.00
J-81	17.82	15.6	15.00	34.00
J-82	33.46	67.4	20.00	26.00
J-83	43.62	25.1	12.00	17.00
J-84	44.19	121.8	12.00	16.00
J-85	43.03	121.0	12.00	17.00
J-86	43.02	14.7	12.00	17.00
J-87	34.73	125.3	17.00	24.00
J-88	34.83	98.5	17.00	24.00
J-89	31.84	38.9	19.00	26.00
J-90	12.41	0.0	50.00	50.00
J-91	33.44	0.0	27.00	28.00
J-92	47.56	0.0	13.00	14.00
J-94	17.34	0.0	43.00	44.00

Table 8: Pipe data in 2025

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-2	J-1	J-2	1381	656	19,558.00	0.67	11,049.70	0.38
P-3	J-2	J-3	453	656	19,558.00	0.67	11,049.70	0.38
P-4	J-3	J-4	369	656	19,558.00	0.67	11,049.70	0.38
P-6	J-5	J-6	318	656	11,465.00	0.39	6,477.40	0.22
P-8	J-7	J-8	825	656	10,391.50	0.36	5,870.90	0.20
P-9	J-8	J-9	975	656	10,084.10	0.35	5,697.20	0.20
P-11	J-10	J-11	211	656	9,053.30	0.31	5,114.90	0.18
P-90	R-1	PMP-1	32	656	19,558.00	0.67	11,049.70	0.38
P-105	PMP-1	J-90	28	656	19,558.00	0.67	11,049.70	0.38
P-106	J-90	J-1	606	656	19,558.00	0.67	11,049.70	0.38
P-113	J-91	J-10	647	656	9,425.00	0.32	5,324.80	0.18
P-128	J-4	J-94	672	656	11,465.00	0.39	6,477.40	0.22
P-129	J-94	J-5	521	656	11,465.00	0.39	6,477.40	0.22
P-151	J-9	J-91	3	656	9,425.00	0.32	5,324.80	0.18
P-7	J-6	J-7	1058	655	10,928.20	0.37	6,174.10	0.21
P-122	J-11	J-92	354	582	6,166.10	0.27	3,483.60	0.15
P-123	J-92	J-12	0	582	6,166.10	0.27	3,483.60	0.15
P-13	J-12	J-13	812	291	4,216.20	0.73	2,382.00	0.42
P-14	J-13	J-14	1018	291	3,638.70	0.63	2,055.70	0.36
P-15	J-14	J-15	128	291	3,388.90	0.59	1,914.60	0.33
P-16	J-15	J-16	194	291	3,271.40	0.57	1,848.30	0.32
P-17	J-16	J-17	453	291	3,142.70	0.55	1,775.50	0.31
P-55	J-11	J-53	756	231	2,653.30	0.73	1,499.00	0.41
P-56	J-53	J-54	863	231	2,508.00	0.69	1,417.00	0.39
P-57	J-54	J-55	704	231	2,156.30	0.60	1,218.20	0.34
P-18	J-17	J-18	123	185	1,942.20	0.84	1,097.30	0.47
P-19	J-18	J-19	764	185	1,371.40	0.59	774.80	0.34
P-58	J-55	J-56	284	185	2,105.80	0.91	1,189.70	0.51
P-59	J-56	J-57	402	185	1,927.40	0.83	1,088.90	0.47
P-60	J-57	J-58	859	185	1,758.30	0.76	993.40	0.43
P-61	J-58	J-59	408	185	1,630.20	0.70	921.00	0.40

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-71	J-59	J-67	1057	185	783.00	0.34	442.40	0.19
P-96	J-19	J-86	42	185	1,252.20	0.54	707.40	0.31
P-23	J-12	J-23	1421	148	1,018.50	0.69	575.40	0.39
P-33	J-17	J-32	469	148	1,072.00	0.73	605.70	0.41
P-34	J-32	J-33	363	148	922.20	0.62	521.00	0.35
P-35	J-33	J-34	381	148	792.20	0.54	447.60	0.30
P-36	J-34	J-35	815	148	495.50	0.34	279.90	0.19
P-37	J-35	J-36	354	148	468.00	0.32	264.40	0.18
P-77	J-67	J-72	847	148	700.40	0.47	395.70	0.27
P-97	J-86	J-20	647	148	1,008.10	0.68	569.50	0.39
P-100	J-20	J-88	667	148	790.90	0.54	446.90	0.30
P-52	J-12	J-50	676	115	388.40	0.43	219.50	0.24
P-53	J-50	J-51	1045	115	282.90	0.31	159.80	0.18
P-78	J-72	J-73	605	115	674.40	0.75	381.00	0.42
P-79	J-73	J-74	377	115	542.90	0.60	306.70	0.34
P-38	J-36	J-37	792	99	290.60	0.43	164.20	0.24
P-44	J-34	J-43	718	99	183.50	0.27	103.70	0.15
P-22	J-21	J-22	1237	81	163.60	0.36	92.40	0.21
P-24	J-23	J-24	890	81	367.00	0.82	207.40	0.46
P-39	J-37	J-38	968	81	186.60	0.41	105.40	0.23
P-70	J-59	J-66	1310	81	388.40	0.86	219.50	0.49
P-80	J-74	J-75	639	81	356.30	0.79	201.30	0.45
P-92	J-18	J-84	18	81	436.20	0.97	246.40	0.55
P-101	J-88	J-21	467	81	396.10	0.88	223.80	0.50
P-102	J-87	J-88	24	81	(220.50)	0.49	(124.60)	0.28
P-25	J-24	J-25	659	68	104.00	0.33	58.80	0.19
P-26	J-23	J-26	377	68	312.00	1.00	176.30	0.57
P-28	J-13	J-28	592	68	160.10	0.51	90.40	0.29
P-42	J-36	J-41	306	68	97.90	0.31	55.30	0.18
P-54	J-51	J-52	553	68	110.10	0.35	62.20	0.20
P-81	J-75	J-76	292	68	201.90	0.65	114.00	0.37
P-93	J-84	J-30	734	68	110.10	0.35	62.20	0.20
P-27	J-26	J-27	246	57	143.80	0.65	81.20	0.37

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-29	J-28	J-29	400	57	45.90	0.21	25.90	0.12
P-31	J-15	J-31	452	57	41.00	0.19	23.20	0.11
P-32	J-31	J-16	455	57	(27.80)	0.13	(15.70)	0.07
P-40	J-38	J-39	790	57	62.70	0.28	35.40	0.16
P-43	J-41	J-42	504	57	36.70	0.17	20.70	0.09
P-45	J-43	J-44	866	57	61.20	0.28	34.60	0.16
P-48	J-20	J-47	519	57	55.10	0.25	31.10	0.14
P-49	J-47	J-48	345	57	7.60	0.03	4.30	0.02
P-50	J-14	J-49	471	57	66.30	0.30	37.40	0.17
P-51	J-49	J-28	655	57	37.20	0.17	21.00	0.10
P-62	J-56	J-60	420	57	124.90	0.57	70.60	0.32
P-63	J-60	J-61	267	57	13.80	0.06	7.80	0.04
P-64	J-60	J-62	473	57	57.60	0.26	32.60	0.15
P-65	J-62	J-63	344	57	119.70	0.54	67.60	0.31
P-66	J-63	J-64	238	57	148.30	0.67	83.80	0.38
P-67	J-64	J-65	573	57	67.30	0.31	38.00	0.17
P-68	J-63	J-58	476	57	(91.40)	0.41	(51.60)	0.23
P-69	J-57	J-62	375	57	115.60	0.52	65.30	0.30
P-82	J-73	J-77	460	57	39.80	0.18	22.50	0.10
P-83	J-77	J-78	291	57	13.80	0.06	7.80	0.04
P-84	J-76	J-79	229	57	140.70	0.64	79.50	0.36
P-85	J-79	J-80	989	57	84.10	0.38	47.50	0.22
P-86	J-80	J-81	826	57	27.50	0.12	15.60	0.07
P-89	J-82	J-54	671	57	(119.30)	0.54	(67.40)	0.31
P-94	J-83	J-84	366	57	(44.30)	0.20	(25.10)	0.11
P-95	J-84	J-85	791	57	66.10	0.30	37.30	0.17
P-98	J-85	J-86	19	57	(218.10)	0.99	(123.20)	0.56
P-99	J-85	J-87	1329	57	70.10	0.32	39.60	0.18
P-103	J-87	J-89	637	57	68.80	0.31	38.90	0.18

Table 9: Node data in 2040

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-1	13.95	0.00	48.0	48.0
J-2	14.01	0.00	48.0	48.0
J-3	14.95	0.00	47.0	47.0
J-4	17.95	0.00	43.0	44.0
J-5	17.05	0.00	44.0	45.0
J-6	27.97	435.50	33.0	34.0
J-7	29.97	435.50	30.0	32.0
J-8	32.14	240.20	28.0	29.0
J-9	33.44	519.30	26.0	28.0
J-10	41.82	295.50	17.0	19.0
J-11	45.14	187.50	14.0	16.0
J-12	47.57	438.00	11.0	13.0
J-13	46.85	354.20	10.0	13.0
J-14	45.98	159.80	8.0	13.0
J-15	45.38	66.50	8.0	14.0
J-16	43.22	87.30	10.0	16.0
J-17	44.56	111.50	8.0	14.0
J-18	44.44	114.00	7.0	14.0
J-19	43.14	102.00	6.0	14.0
J-20	41.74	141.70	4.0	14.0
J-21	34.46	202.20	0.0	17.0
J-22	26.07	142.60	3.0	24.0
J-23	32.45	275.60	19.0	26.0
J-24	30.37	214.30	8.0	23.0
J-25	36.03	84.70	0.0	16.0
J-26	33.90	136.50	8.0	20.0
J-27	31.40	115.80	7.0	22.0
J-28	47.42	130.50	4.0	11.0
J-29	44.03	39.70	7.0	14.0
J-30	44.59	95.00	4.0	12.0
J-31	42.65	59.60	10.0	16.0
J-32	44.83	129.60	5.0	13.0
J-33	44.08	113.20	4.0	13.0
J-34	44.64	98.50	2.0	12.0
J-35	44.63	24.20	0.0	11.0
J-36	44.42	69.10	0.0	11.0
J-37	44.25	90.70	-3.0	10.0
J-38	37.26	108.90	0.0	15.0
J-39	34.33	54.40	0.0	17.0
J-41	44.26	52.70	-1.0	11.0
J-42	44.10	32.00	-1.0	11.0
J-43	43.54	107.10	2.0	12.0
J-44	43.34	53.60	-1.0	11.0
J-47	42.98	41.50	1.0	12.0
J-48	43.69	6.90	0.0	12.0
J-49	45.40	25.10	7.0	13.0
J-50	56.42	89.00	0.0	4.0
J-51	55.25	151.20	-1.0	4.0
J-52	48.67	96.80	4.0	10.0
J-53	43.13	117.50	13.0	17.0
J-54	46.18	188.40	7.0	13.0
J-55	45.60	40.60	6.0	13.0
J-56	44.55	43.20	5.0	13.0

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m³/day)	Peak Demand	Average Demand
J-57	40.36	43.20	7.0	16.0
J-58	31.93	29.40	12.0	23.0
J-59	27.15	371.50	15.0	27.0
J-60	35.14	43.20	10.0	21.0
J-61	34.41	10.40	11.0	21.0
J-62	35.54	43.20	9.0	20.0
J-63	32.47	51.00	8.0	21.0
J-64	31.90	65.70	5.0	21.0
J-65	30.53	55.30	4.0	21.0
J-66	19.69	315.40	1.0	27.0
J-67	37.09	67.40	4.0	17.0
J-72	24.43	21.60	14.0	29.0
J-73	20.62	73.40	13.0	31.0
J-74	18.83	150.30	13.0	32.0
J-75	15.62	124.40	8.0	32.0
J-76	15.54	49.20	4.0	30.0
J-77	19.75	21.60	13.0	31.0
J-78	18.18	10.40	15.0	33.0
J-79	17.08	45.80	0.0	28.0
J-80	17.54	45.80	-6.0	25.0
J-81	17.82	22.50	-7.0	25.0
J-82	33.46	97.60	13.0	23.0
J-83	43.62	38.00	7.0	14.0
J-84	44.19	185.80	7.0	14.0
J-85	43.03	184.90	5.0	14.0
J-86	43.02	22.50	6.0	14.0
J-87	34.73	190.90	8.0	20.0
J-88	34.83	151.20	8.0	20.0
J-89	31.84	59.60	8.0	22.0
J-90	12.41	0.00	50.0	50.0
J-91	33.44	0.00	26.0	28.0
J-92	47.56	0.00	11.0	13.0
J-94	17.34	0.00	44.0	45.0
J-95	13.95	0.00	46.0	48.0
J-96	14.01	0.00	41.0	46.0
J-97	14.95	0.00	39.0	44.0
J-98	17.95	17699	35.0	41.0
J-99	12.46	0.00	50.0	50.0
J-100	12.41	0.00	50.0	50.0

Table 10: Pipe data in 2040

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-2	J-1	J-2	1381	656	19,558.00	0.67	11,049.70	0.38
P-3	J-2	J-3	453	656	19,558.00	0.67	11,049.70	0.38
P-4	J-3	J-4	369	656	19,558.00	0.67	11,049.70	0.38
P-6	J-5	J-6	318	656	11,465.00	0.39	6,477.40	0.22
P-8	J-7	J-8	825	656	10,391.50	0.36	5,870.90	0.20
P-9	J-8	J-9	975	656	10,084.10	0.35	5,697.20	0.20
P-11	J-10	J-11	211	656	9,053.30	0.31	5,114.90	0.18
P-90	R-1	PMP-1	32	656	19,558.00	0.67	11,049.70	0.38
P-105	PMP-1	J-90	28	656	19,558.00	0.67	11,049.70	0.38
P-106	J-90	J-1	606	656	19,558.00	0.67	11,049.70	0.38
P-113	J-91	J-10	647	656	9,425.00	0.32	5,324.80	0.18
P-128	J-4	J-94	672	656	11,465.00	0.39	6,477.40	0.22
P-129	J-94	J-5	521	656	11,465.00	0.39	6,477.40	0.22
P-151	J-9	J-91	3	656	9,425.00	0.32	5,324.80	0.18
P-7	J-6	J-7	1058	655	10,928.20	0.37	6,174.10	0.21
P-122	J-11	J-92	354	582	6,166.10	0.27	3,483.60	0.15
P-123	J-92	J-12	0	582	6,166.10	0.27	3,483.60	0.15
P-13	J-12	J-13	812	291	4,216.20	0.73	2,382.00	0.42
P-14	J-13	J-14	1018	291	3,638.70	0.63	2,055.70	0.36
P-15	J-14	J-15	128	291	3,388.90	0.59	1,914.60	0.33
P-16	J-15	J-16	194	291	3,271.40	0.57	1,848.30	0.32
P-17	J-16	J-17	453	291	3,142.70	0.55	1,775.50	0.31
P-55	J-11	J-53	756	231	2,653.30	0.73	1,499.00	0.41
P-56	J-53	J-54	863	231	2,508.00	0.69	1,417.00	0.39
P-57	J-54	J-55	704	231	2,156.30	0.60	1,218.20	0.34
P-18	J-17	J-18	123	185	1,942.20	0.84	1,097.30	0.47
P-19	J-18	J-19	764	185	1,371.40	0.59	774.80	0.34
P-58	J-55	J-56	284	185	2,105.80	0.91	1,189.70	0.51
P-59	J-56	J-57	402	185	1,927.40	0.83	1,088.90	0.47
P-60	J-57	J-58	859	185	1,758.30	0.76	993.40	0.43
P-61	J-58	J-59	408	185	1,630.20	0.70	921.00	0.40

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-71	J-59	J-67	1057	185	783.00	0.34	442.40	0.19
P-96	J-19	J-86	42	185	1,252.20	0.54	707.40	0.31
P-23	J-12	J-23	1421	148	1,018.50	0.69	575.40	0.39
P-33	J-17	J-32	469	148	1,072.00	0.73	605.70	0.41
P-34	J-32	J-33	363	148	922.20	0.62	521.00	0.35
P-35	J-33	J-34	381	148	792.20	0.54	447.60	0.30
P-36	J-34	J-35	815	148	495.50	0.34	279.90	0.19
P-37	J-35	J-36	354	148	468.00	0.32	264.40	0.18
P-77	J-67	J-72	847	148	700.40	0.47	395.70	0.27
P-97	J-86	J-20	647	148	1,008.10	0.68	569.50	0.39
P-100	J-20	J-88	667	148	790.90	0.54	446.90	0.30
P-52	J-12	J-50	676	115	388.40	0.43	219.50	0.24
P-53	J-50	J-51	1045	115	282.90	0.31	159.80	0.18
P-78	J-72	J-73	605	115	674.40	0.75	381.00	0.42
P-79	J-73	J-74	377	115	542.90	0.60	306.70	0.34
P-38	J-36	J-37	792	99	290.60	0.43	164.20	0.24
P-44	J-34	J-43	718	99	183.50	0.27	103.70	0.15
P-22	J-21	J-22	1237	81	163.60	0.36	92.40	0.21
P-24	J-23	J-24	890	81	367.00	0.82	207.40	0.46
P-39	J-37	J-38	968	81	186.60	0.41	105.40	0.23
P-70	J-59	J-66	1310	81	388.40	0.86	219.50	0.49
P-80	J-74	J-75	639	81	356.30	0.79	201.30	0.45
P-92	J-18	J-84	18	81	436.20	0.97	246.40	0.55
P-101	J-88	J-21	467	81	396.10	0.88	223.80	0.50
P-102	J-87	J-88	24	81	(220.50)	0.49	(124.60)	0.28
P-25	J-24	J-25	659	68	104.00	0.33	58.80	0.19
P-26	J-23	J-26	377	68	312.00	1.00	176.30	0.57
P-28	J-13	J-28	592	68	160.10	0.51	90.40	0.29
P-42	J-36	J-41	306	68	97.90	0.31	55.30	0.18
P-54	J-51	J-52	553	68	110.10	0.35	62.20	0.20
P-81	J-75	J-76	292	68	201.90	0.65	114.00	0.37
P-93	J-84	J-30	734	68	110.10	0.35	62.20	0.20
P-27	J-26	J-27	246	57	143.80	0.65	81.20	0.37

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-29	J-28	J-29	400	57	45.90	0.21	25.90	0.12
P-31	J-15	J-31	452	57	41.00	0.19	23.20	0.11
P-32	J-31	J-16	455	57	(27.80)	0.13	(15.70)	0.07
P-40	J-38	J-39	790	57	62.70	0.28	35.40	0.16
P-43	J-41	J-42	504	57	36.70	0.17	20.70	0.09
P-45	J-43	J-44	866	57	61.20	0.28	34.60	0.16
P-48	J-20	J-47	519	57	55.10	0.25	31.10	0.14
P-49	J-47	J-48	345	57	7.60	0.03	4.30	0.02
P-50	J-14	J-49	471	57	66.30	0.30	37.40	0.17
P-51	J-49	J-28	655	57	37.20	0.17	21.00	0.10
P-62	J-56	J-60	420	57	124.90	0.57	70.60	0.32
P-63	J-60	J-61	267	57	13.80	0.06	7.80	0.04
P-64	J-60	J-62	473	57	57.60	0.26	32.60	0.15
P-65	J-62	J-63	344	57	119.70	0.54	67.60	0.31
P-66	J-63	J-64	238	57	148.30	0.67	83.80	0.38
P-67	J-64	J-65	573	57	67.30	0.31	38.00	0.17
P-68	J-63	J-58	476	57	(91.40)	0.41	(51.60)	0.23
P-69	J-57	J-62	375	57	115.60	0.52	65.30	0.30
P-82	J-73	J-77	460	57	39.80	0.18	22.50	0.10
P-83	J-77	J-78	291	57	13.80	0.06	7.80	0.04
P-84	J-76	J-79	229	57	140.70	0.64	79.50	0.36
P-85	J-79	J-80	989	57	84.10	0.38	47.50	0.22
P-86	J-80	J-81	826	57	27.50	0.12	15.60	0.07
P-89	J-82	J-54	671	57	(119.30)	0.54	(67.40)	0.31
P-94	J-83	J-84	366	57	(44.30)	0.20	(25.10)	0.11
P-95	J-84	J-85	791	57	66.10	0.30	37.30	0.17
P-98	J-85	J-86	19	57	(218.10)	0.99	(123.20)	0.56
P-99	J-85	J-87	1329	57	70.10	0.32	39.60	0.18
P-103	J-87	J-89	637	57	68.80	0.31	38.90	0.18

IV.2. Option 2: Use Booster Pump and secondary pumping station at network

Figure 10: Network Map

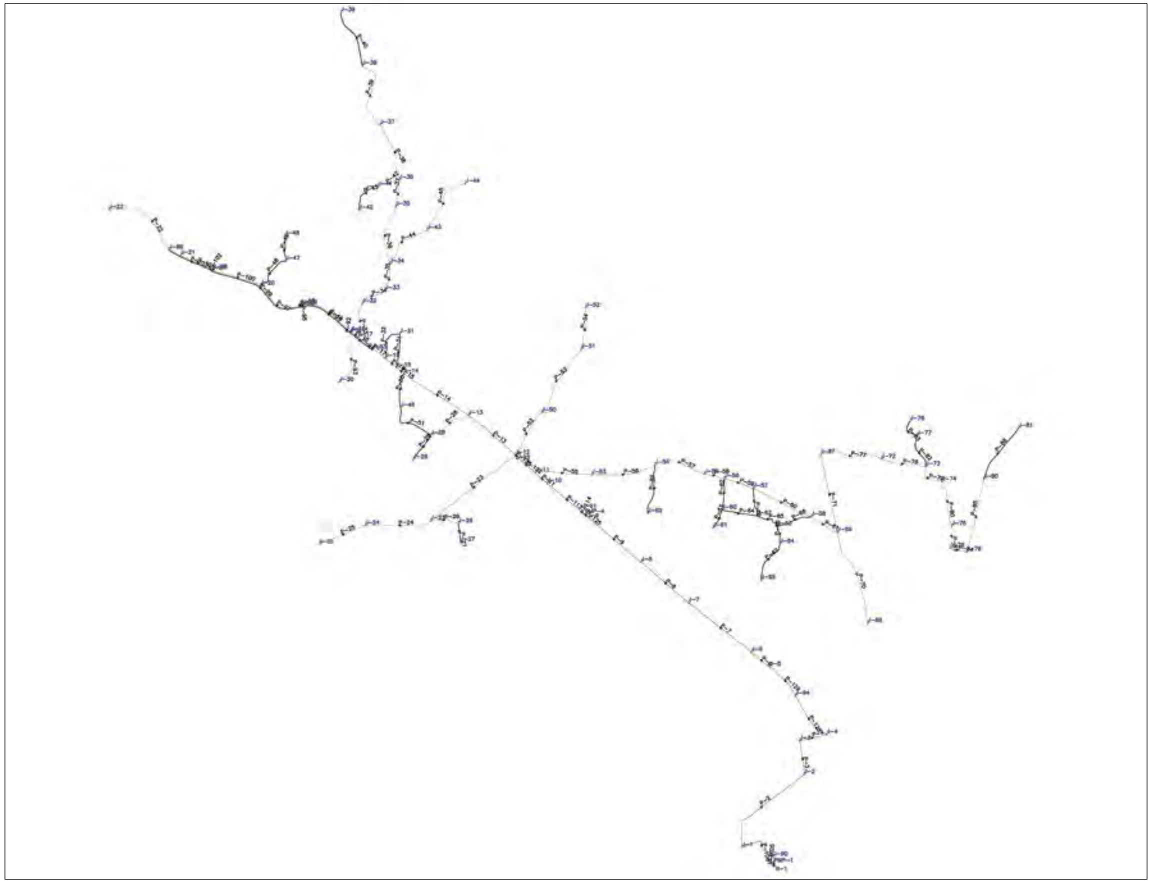


Table 11: Node data in 2025

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-1	13.95	0.0	33.00	24.00
J-2	14.01	0.0	32.00	23.00
J-3	14.95	0.0	31.00	22.00
J-4	17.95	3812.6	27.00	19.00
J-5	17.05	0.0	28.00	19.00
J-6	27.97	253.0	17.00	8.00
J-7	29.97	253.0	15.00	6.00
J-8	32.14	144.9	12.00	3.00
J-9	33.44	310.6	11.00	2.00
J-10	41.82	175.1	16.00	16.00
J-11	45.14	110.4	13.00	13.00
J-12	47.57	255.3	11.00	11.00
J-13	46.85	196.5	11.00	11.00
J-14	45.98	86.3	12.00	12.00
J-15	45.38	35.8	12.00	12.00
J-16	43.22	47.6	14.00	14.00
J-17	44.56	60.4	13.00	13.00
J-18	44.44	63.0	13.00	13.00
J-19	43.14	56.2	14.00	14.00
J-20	41.74	76.5	15.00	15.00
J-21	34.46	109.8	21.00	21.00
J-22	26.07	77.1	28.00	28.00
J-23	32.45	159.8	25.00	25.00
J-24	30.37	124.2	25.00	25.00
J-25	36.03	49.0	19.00	19.00
J-26	33.90	79.3	22.00	22.00
J-27	31.40	67.4	24.00	24.00
J-28	47.42	70.9	10.00	10.00
J-29	44.03	21.5	13.00	13.00
J-30	44.59	51.9	12.00	12.00
J-31	42.65	32.2	15.00	15.00
J-32	44.83	70.6	12.00	12.00
J-33	44.08	61.5	13.00	13.00
J-34	44.64	53.3	12.00	12.00
J-35	44.63	12.8	12.00	12.00
J-36	44.42	37.4	12.00	12.00
J-37	44.25	48.8	12.00	12.00
J-38	37.26	58.6	18.00	18.00
J-39	34.33	29.3	21.00	21.00
J-41	44.26	28.8	12.00	12.00
J-42	44.10	17.3	12.00	12.00
J-43	43.54	57.9	13.00	13.00
J-44	43.34	28.9	13.00	13.00
J-47	42.98	22.4	13.00	13.00
J-48	43.69	3.7	13.00	13.00
J-49	45.40	13.7	12.00	12.00
J-50	56.42	49.8	1.00	1.00
J-51	55.25	81.6	2.00	2.00
J-52	48.67	52.2	9.00	9.00
J-53	43.13	68.2	15.00	15.00
J-54	46.18	109.2	11.00	11.00
J-55	45.60	23.7	12.00	12.00
J-56	44.55	24.9	12.00	12.00

Node Label	Elevation (m)	Base Demand (m³/day)	Residual Pressure (m) Peak Demand	Residual Pressure (m) Average Demand
J-57	40.36	24.9	16.00	16.00
J-58	31.93	17.0	24.00	24.00
J-59	27.15	215.9	29.00	29.00
J-60	35.14	24.9	21.00	21.00
J-61	34.41	6.2	22.00	22.00
J-62	35.54	24.9	21.00	21.00
J-63	32.47	29.4	23.00	23.00
J-64	31.90	38.1	23.00	23.00
J-65	30.53	31.9	24.00	24.00
J-66	19.69	183.1	33.00	33.00
J-67	37.09	39.0	18.00	18.00
J-72	24.43	12.4	31.00	31.00
J-73	20.62	43.2	34.00	34.00
J-74	18.83	88.2	35.00	35.00
J-75	15.62	72.8	37.00	37.00
J-76	15.54	28.7	37.00	37.00
J-77	19.75	12.4	35.00	35.00
J-78	18.18	6.2	36.00	36.00
J-79	17.08	26.6	35.00	35.00
J-80	17.54	26.6	34.00	34.00
J-81	17.82	13.3	33.00	33.00
J-82	33.46	56.4	23.00	23.00
J-83	43.62	20.8	13.00	13.00
J-84	44.19	101.7	13.00	13.00
J-85	43.03	100.5	14.00	14.00
J-86	43.02	12.3	14.00	14.00
J-87	34.73	104.2	21.00	21.00
J-88	34.83	82.3	21.00	21.00
J-89	31.84	32.7	24.00	24.00
J-90	12.41	0.0	35.00	26.00
J-91	33.44	0.0	25.00	25.00
J-92	47.56	0.0	11.00	11.00
J-94	17.34	0.0	28.00	19.00

Table 12: Pipe data in 2025

Pipe Label	Start Node Node Label	Stop Node Node Label	Length (m)	Diameter (mm)	Peak demand Flow (m³/day)	Peak demand Velocity (m/s)	Average Demand Flow (m³/day)	Average Demand Velocity (m/s)
P-2	J-1	J-2	1381	462	16,486.30	1.14	9,210.20	0.64
P-3	J-2	J-3	453	462	16,486.30	1.14	9,210.20	0.64
P-4	J-3	J-4	369	462	16,486.30	1.14	9,210.20	0.64
P-6	J-5	J-6	318	462	9,661.40	0.82	5,397.40	0.46
P-7	J-6	J-7	1058	462	9,208.20	0.79	5,144.30	0.44
P-8	J-7	J-8	825	416	8,755.10	0.75	4,891.10	0.42
P-9	J-8	J-9	975	416	8,495.30	0.72	4,746.00	0.40
P-11	J-10	J-11	211	416	7,626.10	0.65	4,260.40	0.36
P-13	J-12	J-13	812	291	3,550.90	0.62	1,983.70	0.35
P-14	J-13	J-14	1018	291	3,065.30	0.53	1,712.50	0.30
P-15	J-14	J-15	128	291	2,854.90	0.50	1,594.90	0.28
P-16	J-15	J-16	194	291	2,757.30	0.48	1,540.40	0.27
P-17	J-16	J-17	453	291	2,649.30	0.46	1,480.00	0.26
P-18	J-17	J-18	123	185	1,637.80	0.71	915.00	0.40
P-19	J-18	J-19	764	185	1,156.80	0.50	646.30	0.28

P-22	J-21	J-22	1237	81	137.60	0.31	76.90	0.17
P-23	J-12	J-23	1421	148	859.90	0.58	480.40	0.32
P-24	J-23	J-24	890	81	310.90	0.69	173.70	0.39
P-25	J-24	J-25	659	68	88.20	0.28	49.20	0.16
P-26	J-23	J-26	377	68	262.90	0.84	146.90	0.47
P-27	J-26	J-27	246	57	120.60	0.55	67.40	0.31
P-28	J-13	J-28	592	68	134.50	0.43	75.10	0.24
P-29	J-28	J-29	400	57	38.70	0.18	21.60	0.10
P-31	J-15	J-31	452	57	34.20	0.16	19.10	0.09
P-32	J-31	J-16	455	57	(23.00)	0.10	(12.80)	0.06
P-33	J-17	J-32	469	148	903.20	0.61	504.60	0.34
P-34	J-32	J-33	363	148	776.40	0.53	433.70	0.29
P-35	J-33	J-34	381	148	666.60	0.45	372.40	0.25
P-36	J-34	J-35	815	148	416.00	0.28	232.40	0.16
P-37	J-35	J-36	354	148	392.80	0.27	219.50	0.15
P-38	J-36	J-37	792	99	244.40	0.36	136.50	0.20
P-39	J-37	J-38	968	81	157.70	0.35	88.10	0.20
P-40	J-38	J-39	790	57	52.60	0.24	29.40	0.13
P-42	J-36	J-41	306	68	82.00	0.26	45.80	0.15
P-43	J-41	J-42	504	57	30.90	0.14	17.30	0.08
P-44	J-34	J-43	718	99	154.70	0.23	86.40	0.13
P-45	J-43	J-44	866	68	51.00	0.23	28.50	0.13
P-48	J-20	J-47	519	57	46.40	0.21	25.90	0.12
P-50	J-14	J-49	471	57	6.20	0.03	3.50	0.02
P-51	J-49	J-28	655	57	55.70	0.25	31.10	0.14
P-52	J-12	J-50	676	115	31.00	0.14	17.30	0.08
P-53	J-50	J-51	1045	115	327.90	0.36	183.20	0.20
P-54	J-51	J-52	553	68	238.20	0.26	133.10	0.15
P-55	J-11	J-53	756	231	92.80	0.30	51.80	0.17
P-56	J-53	J-54	863	231	2,231.70	0.62	1,246.80	0.34
P-57	J-54	J-55	704	231	2,109.50	0.58	1,178.50	0.33
P-58	J-55	J-56	284	185	1,814.10	0.50	1,013.50	0.28
P-59	J-56	J-57	402	185	1,772.40	0.77	990.10	0.43
P-60	J-57	J-58	859	185	1,622.90	0.70	906.60	0.39
P-61	J-58	J-59	408	185	1,481.10	0.64	827.40	0.36
P-62	J-56	J-60	420	57	1,373.30	0.59	767.20	0.33
P-64	J-60	J-62	473	57	104.60	0.47	58.50	0.27
P-65	J-62	J-63	344	57	10.80	0.05	6.00	0.03
P-66	J-63	J-64	238	57	49.00	0.22	27.40	0.12
P-67	J-64	J-65	573	57	101.10	0.46	56.50	0.26
P-68	J-63	J-58	476	57	125.30	0.57	70.00	0.32
P-69	J-57	J-62	375	57	57.20	0.26	32.00	0.14
P-70	J-59	J-66	1310	81	(76.80)	0.35	(42.90)	0.19
P-71	J-59	J-67	1057	185	96.90	0.44	54.20	0.25
P-77	J-67	J-72	847	148	327.90	0.73	183.20	0.41
P-78	J-72	J-73	605	115	658.80	0.28	368.10	0.16
P-79	J-73	J-74	377	115	589.20	0.40	329.20	0.22
P-80	J-74	J-75	639	81	567.60	0.63	317.10	0.35
P-81	J-75	J-76	292	67	457.80	0.51	255.70	0.28
P-82	J-73	J-77	460	57	300.00	0.67	167.60	0.37
P-83	J-77	J-78	291	57	170.10	0.55	95.00	0.30
P-84	J-76	J-79	229	68	32.50	0.15	18.10	0.08
P-85	J-79	J-80	989	57	10.80	0.05	6.00	0.03
P-86	J-80	J-81	826	57	119.10	0.54	66.50	0.30
P-89	J-82	J-54	671	57	71.10	0.32	39.70	0.18
P-90	R-1	PMP-1	32	462	23.20	0.11	13.00	0.06
P-92	J-18	J-84	18	81	(100.50)	0.46	(56.20)	0.25
P-93	J-84	J-30	734	68	16,486.30	1.14	9,210.20	0.64

P-94	J-83	J-84	366	57	368.10	0.82	205.60	0.46
P-95	J-84	J-85	791	57	92.80	0.30	51.80	0.17
P-96	J-19	J-86	42	185	(37.10)	0.17	(20.70)	0.09
P-97	J-86	J-20	647	148	55.70	0.25	31.10	0.14
P-98	J-85	J-86	19	57	1,056.30	0.46	590.10	0.26
P-99	J-85	J-87	1329	57	851.70	0.58	475.80	0.32
P-100	J-20	J-88	667	148	(182.90)	0.83	(102.20)	0.46
P-101	J-88	J-21	467	81	59.20	0.27	33.10	0.15
P-102	J-87	J-88	24	81	667.70	0.45	373.00	0.25
P-103	J-87	J-89	637	57	334.10	0.74	186.60	0.42
P-105	PMP-1	J-90	28	462	(186.70)	0.42	(104.30)	0.23
P-106	J-90	J-1	606	462	58.80	0.27	32.80	0.15
P-113	J-91	J-10	647	416	16,486.30	1.14	9,210.20	0.64
P-114	PMP-4	J-91	2	416	16,486.30	1.14	9,210.20	0.64
P-122	J-11	J-92	354	416	7,940.00	0.68	4,435.80	0.38
P-123	J-92	J-12	0	416	7,940.00	0.68	4,435.80	0.38
P-125	PMP-4	J-9	1	416	5,196.40	0.44	2,903.00	0.25
P-128	J-4	J-94	672	462	5,196.40	0.44	2,903.00	0.25
P-129	J-94	J-5	521	462	(7,940.00)	0.68	(4,435.80)	0.38

Table 13: Node data in 2040

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-1	13.95	0.00	33.0	26.0
J-2	14.01	0.00	30.0	25.0
J-3	14.95	0.00	28.0	24.0
J-4	17.95	158.90	24.0	20.0
J-5	17.05	0.00	24.0	21.0
J-6	27.97	10.50	13.0	10.0
J-7	29.97	10.50	11.0	8.0
J-8	32.14	6.00	8.0	5.0
J-9	33.44	12.90	6.0	4.0
J-10	41.82	7.30	19.0	18.0
J-11	45.14	4.60	16.0	14.0
J-12	47.57	10.70	13.0	12.0
J-13	46.85	8.20	13.0	12.0
J-14	45.98	3.60	13.0	13.0
J-15	45.38	1.50	14.0	13.0
J-16	43.22	2.00	16.0	15.0
J-17	44.56	2.50	14.0	14.0
J-18	44.44	2.60	14.0	14.0
J-19	43.14	2.30	15.0	15.0
J-20	41.74	3.20	16.0	16.0

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-21	34.46	4.60	19.0	22.0
J-22	26.07	3.20	26.0	30.0
J-23	32.45	6.70	25.0	26.0
J-24	30.37	5.20	25.0	27.0
J-25	36.03	2.10	18.0	21.0
J-26	33.90	3.30	20.0	23.0
J-27	31.40	2.80	21.0	25.0
J-28	47.42	3.00	11.0	11.0
J-29	44.03	0.90	14.0	14.0
J-30	44.59	2.20	13.0	13.0
J-31	42.65	1.30	16.0	16.0
J-32	44.83	3.00	13.0	14.0
J-33	44.08	2.60	14.0	14.0
J-34	44.64	2.20	13.0	13.0
J-35	44.63	0.50	11.0	13.0
J-36	44.42	1.50	11.0	13.0
J-37	44.25	2.00	10.0	13.0
J-38	37.26	2.40	15.0	19.0
J-39	34.33	1.20	17.0	22.0
J-41	44.26	1.20	10.0	13.0
J-42	44.10	0.70	10.0	13.0
J-43	43.54	2.40	13.0	14.0
J-44	43.34	1.20	12.0	14.0
J-47	42.98	0.90	14.0	15.0
J-48	43.69	0.10	13.0	14.0
J-49	45.40	0.60	13.0	13.0
J-50	56.42	2.10	4.0	3.0
J-51	55.25	3.40	3.0	3.0
J-52	48.67	2.20	9.0	10.0
J-53	43.13	2.80	17.0	16.0
J-54	46.18	4.50	13.0	12.0
J-55	45.60	1.00	13.0	13.0
J-56	44.55	1.00	13.0	14.0
J-57	40.36	1.00	16.0	17.0
J-58	31.93	0.70	23.0	25.0
J-59	27.15	9.00	27.0	30.0
J-60	35.14	1.00	20.0	22.0
J-61	34.41	0.30	21.0	23.0
J-62	35.54	1.00	19.0	22.0
J-63	32.47	1.20	21.0	24.0
J-64	31.90	1.60	20.0	24.0
J-65	30.53	1.30	21.0	25.0
J-66	19.69	7.60	31.0	36.0
J-67	37.09	1.60	16.0	20.0
J-72	24.43	0.50	28.0	32.0
J-73	20.62	1.80	31.0	36.0
J-74	18.83	3.70	32.0	37.0
J-75	15.62	3.00	34.0	40.0
J-76	15.54	1.20	34.0	40.0
J-77	19.75	0.50	32.0	36.0
J-78	18.18	0.30	34.0	38.0
J-79	17.08	1.10	32.0	38.0
J-80	17.54	1.10	29.0	37.0
J-81	17.82	0.50	29.0	36.0

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m³/day)	Peak Demand	Average Demand
J-82	33.46	2.30	23.0	24.0
J-83	43.62	0.90	14.0	15.0
J-84	44.19	4.20	14.0	14.0
J-85	43.03	4.20	15.0	15.0
J-86	43.02	0.50	15.0	15.0
J-87	34.73	4.40	22.0	23.0
J-88	34.83	3.40	22.0	23.0
J-89	31.84	1.40	24.0	26.0
J-90	12.41	0.00	35.0	28.0
J-91	33.44	0.00	28.0	26.0
J-92	47.56	0.00	13.0	12.0
J-94	17.34	0.00	25.0	21.0
J-95	13.95	0.00	31.0	25.6
J-96	14.01	0.00	27.0	23.8
J-97	14.95	0.00	24.0	22.2
J-98	17.95	17699	20.0	18.8
J-99	12.46	0.00	35.0	27.9
J-100	12.41	0.00	35.0	27.9

Table 14: Pipe data in 2040

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-2	J-1	J-2	1,381	461.8	13,388.2	0.93	7,922.0	0.55
P-3	J-2	J-3	453	461.8	13,388.2	0.93	7,922.0	0.55
P-4	J-3	J-4	369	461.8	13,388.2	0.93	7,922.0	0.55
P-6	J-5	J-6	318	415.6	13,388.2	1.14	7,922.0	0.68
P-7	J-6	J-7	1,058	415.6	12,774.9	1.09	7,559.1	0.64
P-8	J-7	J-8	825	415.6	12,161.7	1.04	7,196.3	0.61
P-9	J-8	J-9	975	415.6	11,824.4	1.01	6,996.7	0.60
P-11	J-10	J-11	211	415.6	10,676.7	0.91	6,317.6	0.54
P-13	J-12	J-13	812	290.8	5,136.8	0.90	3,039.6	0.53
P-14	J-13	J-14	1,018	290.8	4,443.1	0.77	2,629.0	0.46
P-15	J-14	J-15	128	290.8	4,138.1	0.72	2,448.6	0.43
P-16	J-15	J-16	194	290.8	3,994.3	0.70	2,363.5	0.41
P-17	J-16	J-17	453	290.8	3,835.8	0.67	2,269.7	0.40
P-18	J-17	J-18	123	184.6	2,364.0	1.02	1,398.8	0.60
P-19	J-18	J-19	764	184.6	1,674.4	0.72	990.8	0.43
P-22	J-21	J-22	1,237	81.4	201.5	0.45	119.2	0.27
P-23	J-12	J-23	1,421	147.6	1,163.7	0.79	688.6	0.47
P-24	J-23	J-24	890	81.4	419.1	0.93	248.0	0.55
P-25	J-24	J-25	659	67.8	118.3	0.38	70.0	0.22
P-26	J-23	J-26	377	67.8	356.3	1.14	210.8	0.68
P-27	J-26	J-27	246	57.0	163.5	0.74	96.8	0.44
P-28	J-13	J-28	592	67.8	194.4	0.62	115.0	0.37
P-29	J-28	J-29	400	57.0	55.5	0.25	32.8	0.15
P-31	J-15	J-31	452	57.0	50.4	0.23	29.8	0.14
P-32	J-31	J-16	455	57.0	(34.3)	0.16	(20.3)	0.09
P-33	J-17	J-32	469	147.6	1,315.6	0.89	778.5	0.53
P-34	J-32	J-33	363	147.6	1,133.1	0.77	670.5	0.45
P-35	J-33	J-34	381	147.6	973.9	0.66	576.3	0.39

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-36	J-34	J-35	815	147.6	607.4	0.41	359.4	0.24
P-37	J-35	J-36	354	147.6	573.8	0.39	339.6	0.23
P-38	J-36	J-37	792	99.4	356.3	0.53	210.8	0.31
P-39	J-37	J-38	968	81.4	229.2	0.51	135.6	0.30
P-40	J-38	J-39	790	57.0	75.9	0.34	44.9	0.20
P-42	J-36	J-41	306	67.8	119.7	0.38	70.8	0.23
P-43	J-41	J-42	504	57.0	45.3	0.21	26.8	0.12
P-44	J-34	J-43	718	99.4	227.8	0.34	134.8	0.20
P-45	J-43	J-44	866	57.0	75.9	0.34	44.9	0.20
P-48	J-20	J-47	519	57.0	68.6	0.31	40.6	0.18
P-49	J-47	J-48	345	57.0	10.2	0.05	6.0	0.03
P-50	J-14	J-49	471	57.0	80.1	0.36	47.4	0.21
P-51	J-49	J-28	655	57.0	45.0	0.20	26.7	0.12
P-52	J-12	J-50	676	115.4	474.6	0.53	280.8	0.31
P-53	J-50	J-51	1,045	115.4	349.0	0.39	206.5	0.23
P-54	J-51	J-52	553	67.8	135.8	0.44	80.4	0.26
P-55	J-11	J-53	756	230.8	3,019.6	0.84	1,786.8	0.49
P-56	J-53	J-54	863	230.8	2,854.6	0.79	1,689.1	0.47
P-57	J-54	J-55	704	230.8	2,451.6	0.68	1,450.7	0.40
P-58	J-55	J-56	284	184.6	2,394.7	1.04	1,417.0	0.61
P-59	J-56	J-57	402	184.6	2,193.9	0.95	1,298.2	0.56
P-60	J-57	J-58	859	184.6	2,003.4	0.87	1,185.5	0.51
P-61	J-58	J-59	408	184.6	1,858.8	0.80	1,099.9	0.48
P-62	J-56	J-60	420	57.0	140.9	0.64	83.4	0.38
P-63	J-60	J-61	267	57.0	14.6	0.07	8.6	0.04
P-64	J-60	J-62	473	57.0	66.4	0.30	39.3	0.18
P-65	J-62	J-63	344	57.0	137.1	0.62	81.2	0.37
P-66	J-63	J-64	238	57.0	169.4	0.77	100.2	0.45
P-67	J-64	J-65	573	57.0	77.4	0.35	45.8	0.21
P-68	J-63	J-58	476	57.0	(103.8)	0.47	(61.4)	0.28
P-69	J-57	J-62	375	57.0	130.6	0.59	77.3	0.35
P-70	J-59	J-66	1,310	81.4	443.9	0.99	262.7	0.58
P-71	J-59	J-67	1,057	184.6	890.7	0.39	527.0	0.23
P-77	J-67	J-72	847	147.6	795.8	0.54	470.9	0.32
P-78	J-72	J-73	605	115.4	765.1	0.85	452.7	0.50
P-79	J-73	J-74	377	115.4	616.2	0.68	364.6	0.40
P-80	J-74	J-75	639	81.4	404.5	0.90	239.3	0.53
P-81	J-75	J-76	292	67.8	229.2	0.73	135.6	0.43
P-82	J-73	J-77	460	57.0	45.3	0.21	26.8	0.12
P-83	J-77	J-78	291	57.0	14.6	0.07	8.6	0.04
P-84	J-76	J-79	229	57.0	160.6	0.73	95.0	0.43
P-85	J-79	J-80	989	57.0	96.4	0.44	57.0	0.26
P-86	J-80	J-81	826	57.0	32.1	0.15	19.0	0.09
P-89	J-82	J-54	671	57.0	(137.3)	0.62	(81.2)	0.37
P-90	R-1	PMP-1	32	461.8	13,008.1	0.90	11,545.6	0.80
P-92	J-18	J-84	18	81.4	528.9	1.18	313.0	0.70
P-93	J-84	J-30	734	67.8	132.9	0.43	78.6	0.25
P-94	J-83	J-84	366	57.0	(54.0)	0.25	(32.0)	0.14
P-95	J-84	J-85	791	57.0	80.7	0.37	47.7	0.22
P-96	J-19	J-86	42	184.6	1,531.3	0.66	906.1	0.39
P-97	J-86	J-20	647	147.6	1,234.3	0.83	730.3	0.49
P-98	J-85	J-86	19	57.0	(265.0)	1.20	(156.8)	0.71

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-99	J-85	J-87	1,329	57.0	85.7	0.39	50.7	0.23
P-100	J-20	J-88	667	147.6	965.6	0.65	571.4	0.39
P-101	J-88	J-21	467	81.4	486.2	1.08	287.7	0.64
P-102	J-87	J-88	24	81.4	(266.2)	0.59	(157.5)	0.35
P-103	J-87	J-89	637	57.0	83.2	0.38	49.2	0.22
P-105	PMP-1	J-90	28	461.8	13,008.1	0.90	11,545.6	0.80
P-106	J-90	J-1	606	461.8	13,388.2	0.93	7,922.0	0.55
P-113	J-91	J-10	647	415.6	11,092.8	0.95	6,563.8	0.56
P-114	PMP-4	J-91	2	415.6	11,092.8	0.95	6,563.8	0.56
P-122	J-11	J-92	354	415.6	7,392.8	0.63	4,374.4	0.37
P-123	J-92	J-12	0	415.6	7,392.8	0.63	4,374.4	0.37
P-125	PMP-4	J-9	1	415.6	(11,092.8)	0.95	(6,563.8)	0.56
P-128	J-4	J-94	672	461.8	13,388.2	0.93	7,922.0	0.55
P-129	J-94	J-5	521	461.8	13,388.2	0.93	7,922.0	0.55
P-131	J-95	J-96	1347	517.2	25,636.0	1.41	15,169.2	0.84
P-132	J-96	J-97	449	517.2	25,636.0	1.41	15,169.2	0.84
P-133	J-97	J-98	359	517.2	25,636.0	1.41	15,169.2	0.84
P-136	R-1	PMP-5	39	461.8	13,008.1	0.90	11,545.6	0.80
P-140	J-99	J-95	615	517.2	25,636.0	1.41	15,169.2	0.84
P-143	PMP-5	J-100	23	461.8	13,008.1	0.90	11,545.6	0.80
P-146	J-90	J-99	19	517.2	25,636.0	1.41	15,169.2	0.84

IV.3. Option 3: Pump + Pump + Water Tower

Figure 11: Network Map

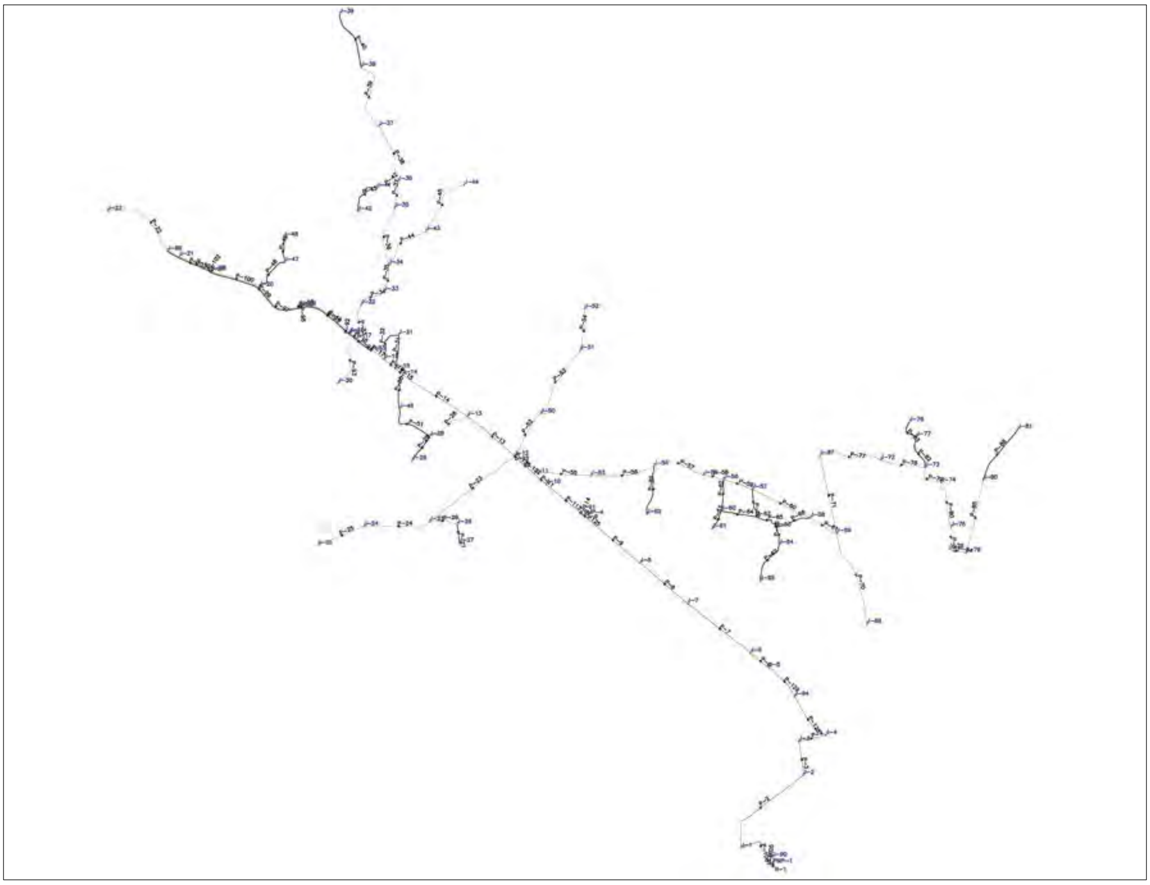


Table 15: Node data in 2025

Node Label	Elevation (m)	Base Demand (m^3/day)	Residual Pressure (m)	
			Peak Demand	Average Demand
J-1	13.95	0.0	33.00	26.00
J-2	14.01	0.0	30.00	25.00
J-3	14.95	0.0	28.00	24.00
J-4	17.95	3812.8	24.00	20.00
J-5	17.05	0.0	24.00	21.00
J-6	27.97	253.2	13.00	10.00
J-7	29.97	253.2	11.00	8.00
J-8	32.14	145.2	8.00	5.00
J-9	33.44	310.2	6.00	4.00
J-10	41.82	175.4	19.00	18.00
J-11	45.14	110.6	16.00	14.00
J-12	47.57	255.7	13.00	12.00
J-13	46.85	196.1	13.00	12.00
J-14	45.98	86.4	13.00	13.00
J-15	45.38	35.4	14.00	13.00
J-16	43.22	47.5	16.00	15.00
J-17	44.56	60.5	14.00	14.00
J-18	44.44	63.1	14.00	14.00
J-19	43.14	56.2	15.00	15.00
J-20	41.74	76.9	16.00	16.00
J-21	34.46	109.7	19.00	22.00
J-22	26.07	76.9	26.00	30.00
J-23	32.45	159.8	25.00	26.00
J-24	30.37	124.4	25.00	27.00
J-25	36.03	49.2	18.00	21.00
J-26	33.90	79.5	20.00	23.00
J-27	31.40	67.4	21.00	25.00
J-28	47.42	70.8	11.00	11.00
J-29	44.03	21.6	14.00	14.00
J-30	44.59	51.8	13.00	13.00
J-31	42.65	32.0	16.00	16.00
J-32	44.83	70.8	13.00	14.00
J-33	44.08	61.3	14.00	14.00
J-34	44.64	53.6	13.00	13.00
J-35	44.63	13.0	11.00	13.00
J-36	44.42	37.2	11.00	13.00
J-37	44.25	48.4	10.00	13.00
J-38	37.26	58.8	15.00	19.00
J-39	34.33	29.4	17.00	22.00
J-41	44.26	28.5	10.00	13.00
J-42	44.10	17.3	10.00	13.00
J-43	43.54	57.9	13.00	14.00
J-44	43.34	28.5	12.00	14.00
J-47	42.98	22.5	14.00	15.00
J-48	43.69	3.5	13.00	14.00
J-49	45.40	13.8	13.00	13.00
J-50	56.42	50.1	4.00	3.00
J-51	55.25	81.2	3.00	3.00
J-52	48.67	51.8	9.00	10.00
J-53	43.13	68.3	17.00	16.00
J-54	46.18	108.9	13.00	12.00
J-55	45.60	23.3	13.00	13.00
J-56	44.55	25.1	13.00	14.00

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m³/day)	Peak Demand	Average Demand
J-57	40.36	25.1	16.00	17.00
J-58	31.93	17.3	23.00	25.00
J-59	27.15	216.0	27.00	30.00
J-60	35.14	25.1	20.00	22.00
J-61	34.41	6.0	21.00	23.00
J-62	35.54	25.1	19.00	22.00
J-63	32.47	29.4	21.00	24.00
J-64	31.90	38.0	20.00	24.00
J-65	30.53	32.0	21.00	25.00
J-66	19.69	183.2	31.00	36.00
J-67	37.09	38.9	16.00	20.00
J-72	24.43	12.1	28.00	32.00
J-73	20.62	43.2	31.00	36.00
J-74	18.83	88.1	32.00	37.00
J-75	15.62	72.6	34.00	40.00
J-76	15.54	28.5	34.00	40.00
J-77	19.75	12.1	32.00	36.00
J-78	18.18	6.0	34.00	38.00
J-79	17.08	26.8	32.00	38.00
J-80	17.54	26.8	29.00	37.00
J-81	17.82	13.0	29.00	36.00
J-82	33.46	56.2	23.00	24.00
J-83	43.62	20.7	14.00	15.00
J-84	44.19	102.0	14.00	14.00
J-85	43.03	100.2	15.00	15.00
J-86	43.02	12.1	15.00	15.00
J-87	34.73	104.5	22.00	23.00
J-88	34.83	82.1	22.00	23.00
J-89	31.84	32.8	24.00	26.00
J-90	12.41	0.0	35.00	28.00
J-91	33.44	0.0	28.00	26.00
J-92	47.56	0.0	13.00	12.00
J-94	17.34	0.0	25.00	21.00

Table 16: Pipe data in 2025

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-2	J-1	J-2	1381	462	16,302.10	1.13	9,210.20	0.64
P-3	J-2	J-3	453	462	16,302.10	1.13	9,210.20	0.64
P-4	J-3	J-4	369	462	16,302.10	1.13	9,210.20	0.64
P-6	J-5	J-6	318	462	9,553.40	0.66	5,397.40	0.37
P-7	J-6	J-7	1058	462	9,105.30	0.63	5,144.30	0.36
P-8	J-7	J-8	825	462	8,657.30	0.60	4,891.10	0.34
P-9	J-8	J-9	975	462	8,400.30	0.58	4,746.00	0.33
P-11	J-10	J-11	211	462	7,540.90	0.52	4,260.40	0.29
P-13	J-12	J-13	812	291	3,511.20	0.61	1,983.70	0.35
P-14	J-13	J-14	1018	291	3,031.10	0.53	1,712.50	0.30
P-15	J-14	J-15	128	291	2,823.10	0.49	1,594.90	0.28
P-16	J-15	J-16	194	291	2,726.50	0.48	1,540.40	0.27
P-17	J-16	J-17	453	291	2,619.70	0.46	1,480.00	0.26

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-18	J-17	J-18	123	208	1,619.50	0.55	915.00	0.31
P-19	J-18	J-19	764	208	1,156.90	0.39	653.60	0.22
P-22	J-21	J-22	1237	81	136.10	0.30	76.90	0.17
P-23	J-12	J-23	1421	148	850.30	0.58	480.40	0.32
P-24	J-23	J-24	890	99	307.40	0.46	173.70	0.26
P-25	J-24	J-25	659	57	87.20	0.40	49.20	0.22
P-26	J-23	J-26	377	68	260.00	0.83	146.90	0.47
P-27	J-26	J-27	246	57	119.30	0.54	67.40	0.31
P-28	J-13	J-28	592	68	133.00	0.43	75.10	0.24
P-29	J-28	J-29	400	57	38.20	0.17	21.60	0.10
P-31	J-15	J-31	452	57	33.90	0.15	19.10	0.09
P-32	J-31	J-16	455	57	(22.70)	0.10	(12.80)	0.06
P-33	J-17	J-32	469	185	893.10	0.39	504.60	0.22
P-34	J-32	J-33	363	185	767.70	0.33	433.70	0.19
P-35	J-33	J-34	381	148	659.10	0.45	372.40	0.25
P-36	J-34	J-35	815	115	411.40	0.46	232.40	0.26
P-37	J-35	J-36	354	115	388.40	0.43	219.50	0.24
P-38	J-36	J-37	792	99	241.60	0.36	136.50	0.20
P-39	J-37	J-38	968	81	156.00	0.35	88.10	0.20
P-40	J-38	J-39	790	57	52.00	0.24	29.40	0.13
P-42	J-36	J-41	306	57	81.10	0.37	45.80	0.21
P-43	J-41	J-42	504	57	30.60	0.14	17.30	0.08
P-44	J-34	J-43	718	81	152.90	0.34	86.40	0.19
P-45	J-43	J-44	866	57	50.50	0.23	28.50	0.13
P-48	J-20	J-47	519	57	45.90	0.21	25.90	0.12
P-49	J-47	J-48	345	57	6.10	0.03	3.50	0.02
P-50	J-14	J-49	471	57	55.10	0.25	31.10	0.14
P-51	J-49	J-28	655	57	30.60	0.14	17.30	0.08
P-52	J-12	J-50	676	115	324.20	0.36	183.20	0.20
P-53	J-50	J-51	1045	99	235.50	0.35	133.10	0.20
P-54	J-51	J-52	553	68	91.80	0.29	51.80	0.17
P-56	J-53	J-54	863	231	2,085.90	0.58	1,178.50	0.33
P-57	J-54	J-55	704	231	1,793.80	0.50	1,013.50	0.28
P-58	J-55	J-56	284	185	1,752.60	0.76	990.10	0.43
P-59	J-56	J-57	402	185	1,604.70	0.69	906.60	0.39
P-60	J-57	J-58	859	185	1,464.50	0.63	827.40	0.36
P-61	J-58	J-59	408	185	1,358.00	0.59	767.20	0.33
P-62	J-56	J-60	420	57	103.50	0.47	58.50	0.27
P-63	J-60	J-61	267	57	10.70	0.05	6.00	0.03
P-64	J-60	J-62	473	57	48.40	0.22	27.40	0.12
P-65	J-62	J-63	344	57	99.90	0.45	56.50	0.26
P-66	J-63	J-64	238	57	123.90	0.56	70.00	0.32
P-67	J-64	J-65	573	57	56.60	0.26	32.00	0.14
P-68	J-63	J-58	476	57	(75.90)	0.34	(42.90)	0.19
P-69	J-57	J-62	375	57	95.90	0.43	54.20	0.25
P-70	J-59	J-66	1310	99	324.20	0.48	183.20	0.27
P-71	J-59	J-67	1057	185	651.50	0.28	368.10	0.16
P-77	J-67	J-72	847	148	582.70	0.39	329.20	0.22
P-78	J-72	J-73	605	148	561.20	0.38	317.10	0.21
P-79	J-73	J-74	377	115	452.70	0.50	255.70	0.28
P-80	J-74	J-75	639	99	296.70	0.44	167.60	0.25

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-81	J-75	J-76	292	81	168.20	0.37	95.00	0.21
P-82	J-73	J-77	460	57	32.10	0.15	18.10	0.08
P-83	J-77	J-78	291	57	10.70	0.05	6.00	0.03
P-84	J-76	J-79	229	68	117.80	0.38	66.50	0.21
P-85	J-79	J-80	989	57	70.30	0.32	39.70	0.18
P-86	J-80	J-81	826	57	22.90	0.10	13.00	0.06
P-89	J-82	J-54	671	57	(99.40)	0.45	(56.20)	0.25
P-90	R-1	PMP-1	32	462	16,302.10	1.13	9,210.20	0.64
P-92	J-18	J-84	18	81	351.00	0.78	198.30	0.44
P-93	J-84	J-30	734	68	91.80	0.29	51.80	0.17
P-94	J-83	J-84	366	57	(36.70)	0.17	(20.70)	0.09
P-95	J-84	J-85	791	57	42.10	0.19	23.80	0.11
P-96	J-19	J-86	42	208	1,057.50	0.36	597.40	0.20
P-97	J-86	J-20	647	208	860.30	0.29	486.00	0.17
P-98	J-85	J-86	19	57	(175.70)	0.80	(99.30)	0.45
P-99	J-85	J-87	1329	57	40.40	0.18	22.80	0.10
P-100	J-20	J-88	667	148	678.30	0.46	383.20	0.26
P-101	J-88	J-21	467	81	330.30	0.73	186.60	0.42
P-102	J-87	J-88	24	81	(202.70)	0.45	(114.50)	0.25
P-103	J-87	J-89	637	57	58.10	0.26	32.80	0.15
P-105	PMP-1	J-90	28	462	16,302.10	1.13	9,210.20	0.64
P-113	J-91	J-10	647	462	7,851.30	0.54	4,435.80	0.31
P-114	PMP-4	J-91	2	462	7,851.30	0.54	4,435.80	0.31
P-123	J-92	J-12	0	416	5,138.40	0.44	2,903.00	0.25
P-125	PMP-4	J-9	1	462	(7,851.30)	0.54	(4,435.80)	0.31
P-128	J-4	J-94	672	462	9,553.40	0.66	5,397.40	0.37
P-129	J-94	J-5	521	462	9,553.40	0.66	5,397.40	0.37
P-130	J-90	FCV-1	7	517	16,302.10	0.90	9,210.20	0.51
P-131	FCV-1	J-1	598	517	16,302.10	0.90	9,210.20	0.51
P-132	J-11	FCV-3	17	231	2,206.80	0.61	1,246.80	0.34
P-133	FCV-3	J-53	740	231	2,206.80	0.61	1,246.80	0.34
P-134	J-11	FCV-4	19	416	5,138.40	0.44	2,903.00	0.25
P-135	FCV-4	J-92	336	416	5,138.40	0.44	2,903.00	0.25

Table 17: Node data in 2040

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m³/day)	Peak Demand	Average Demand
J-1	13.95	0.00	33.0	26.2
J-2	14.01	0.00	30.0	25.1
J-3	14.95	0.00	28.0	23.8
J-4	17.95	0.00	25.0	20.6
J-5	17.05	0.00	23.0	20.6
J-6	27.97	435.50	12.0	9.5
J-7	29.97	435.50	8.0	6.8
J-8	32.14	240.20	5.0	4.1
J-9	33.44	519.30	2.0	2.3

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m³/day)	Peak Demand	Average Demand
J-10	41.82	295.50	23.0	19.3
J-11	45.14	187.50	19.0	15.9
J-12	47.57	438.00	16.0	13.3
J-13	46.85	354.20	15.0	13.1
J-14	45.98	159.80	13.0	13.1
J-15	45.38	66.50	14.0	13.6
J-16	43.22	87.30	15.0	15.6
J-17	44.56	111.50	13.0	13.9
J-18	44.44	114.00	13.0	13.9
J-19	43.14	102.00	13.0	14.6
J-20	41.74	141.70	13.0	15.7
J-21	34.46	202.20	9.0	18.6
J-22	26.07	142.60	13.0	25.2
J-23	32.45	275.60	24.0	25.6
J-24	30.37	214.30	21.0	25.8
J-25	36.03	84.70	10.0	18.1
J-26	33.90	136.50	13.0	20.4
J-27	31.40	115.80	12.0	21.5
J-28	47.42	130.50	9.0	10.6
J-29	44.03	39.70	12.0	13.7
J-30	44.59	95.00	9.0	12.4
J-31	42.65	59.60	16.0	16.0
J-32	44.83	129.60	12.0	13.3
J-33	44.08	113.20	12.0	13.8
J-34	44.64	98.50	10.0	12.7
J-35	44.63	24.20	9.0	12.2
J-36	44.42	69.10	7.0	11.8
J-37	44.25	90.70	4.0	10.7
J-38	37.26	108.90	7.0	15.9
J-39	34.33	54.40	7.0	17.8
J-41	44.26	52.70	5.0	11.0
J-42	44.10	32.00	5.0	10.9
J-43	43.54	107.10	10.0	13.3
J-44	43.34	53.60	7.0	12.4
J-47	42.98	41.50	11.0	13.9
J-48	43.69	6.90	10.0	13.2
J-49	45.40	25.10	12.0	13.0
J-50	56.42	89.00	5.0	3.6
J-51	55.25	151.20	3.0	3.3
J-52	48.67	96.80	7.0	8.9
J-53	43.13	117.50	19.0	16.9
J-54	46.18	188.40	13.0	12.8
J-55	45.60	40.60	12.0	12.7
J-56	44.55	43.20	11.0	13.1
J-57	40.36	43.20	13.0	16.4
J-58	31.93	29.40	17.0	23.2
J-59	27.15	371.50	20.0	27.3
J-60	35.14	43.20	16.0	20.7
J-61	34.41	10.40	16.0	21.4
J-62	35.54	43.20	14.0	19.8
J-63	32.47	51.00	14.0	21.5
J-64	31.90	65.70	11.0	20.6
J-65	30.53	55.30	10.0	21.2
J-66	19.69	315.40	20.0	31.7
J-67	37.09	67.40	9.0	16.9

Node Label	Elevation	Base Demand	Residual Pressure (m)	
	(m)	(m³/day)	Peak Demand	Average Demand
J-72	24.43	21.60	20.0	28.7
J-73	20.62	73.40	22.0	32.0
J-74	18.83	150.30	22.0	33.0
J-75	15.62	124.40	22.0	34.9
J-76	15.54	49.20	20.0	34.5
J-77	19.75	21.60	22.0	32.6
J-78	18.18	10.40	24.0	34.1
J-79	17.08	45.80	17.0	32.4
J-80	17.54	45.80	12.0	29.9
J-81	17.82	22.50	11.0	29.4
J-82	33.46	97.60	19.0	22.8
J-83	43.62	38.00	13.0	14.3
J-84	44.19	185.80	13.0	14.0
J-85	43.03	184.90	12.0	14.4
J-86	43.02	22.50	13.0	14.7
J-87	34.73	190.90	18.0	21.6
J-88	34.83	151.20	18.0	21.6
J-89	31.84	59.60	18.0	23.5
J-90	12.41	0.00	35.0	27.9
J-91	33.44	0.00	32.0	28.0
J-92	47.56	0.00	16.0	13.3
J-94	17.34	0.00	24.0	20.7
J-95	13.95	0.00	31.0	25.6
J-96	14.01	0.00	27.0	23.8
J-97	14.95	0.00	24.0	22.2
J-98	17.95	17699	20.0	18.8
J-99	12.46	0.00	35.0	27.9
J-100	12.41	0.00	35.0	27.9

Table 18: Pipe data in 2040

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-2	J-1	J-2	1,381	461.8	15,881.8	1.10	9,510.0	0.66
P-3	J-2	J-3	453	461.8	15,881.8	1.10	9,510.0	0.66
P-4	J-3	J-4	369	461.8	15,881.8	1.10	9,510.0	0.66
P-6	J-5	J-6	318	461.8	15,881.8	1.10	9,510.0	0.66
P-7	J-6	J-7	1,058	461.8	15,154.6	1.05	9,074.6	0.63
P-8	J-7	J-8	825	461.8	14,427.4	1.00	8,639.1	0.60
P-9	J-8	J-9	975	461.8	14,026.2	0.97	8,398.9	0.58
P-11	J-10	J-11	211	461.8	12,665.6	0.88	7,584.2	0.52
P-13	J-12	J-13	812	290.8	6,090.4	1.06	3,646.9	0.64
P-14	J-13	J-14	1,018	290.8	5,268.0	0.92	3,154.5	0.55
P-15	J-14	J-15	128	290.8	4,905.8	0.85	2,937.6	0.51
P-16	J-15	J-16	194	290.8	4,735.3	0.83	2,835.5	0.49
P-17	J-16	J-17	453	290.8	4,549.4	0.79	2,724.2	0.47
P-18	J-17	J-18	123	207.8	2,803.5	0.96	1,678.8	0.57
P-19	J-18	J-19	764	207.8	2,007.4	0.69	1,202.1	0.41
P-22	J-21	J-22	1,237	81.4	238.1	0.53	142.6	0.32
P-23	J-12	J-23	1,421	147.6	1,380.8	0.93	826.8	0.56
P-24	J-23	J-24	890	99.4	499.2	0.74	298.9	0.45
P-25	J-24	J-25	659	57.0	141.4	0.64	84.7	0.38

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m³/day)	(m/s)	(m³/day)	(m/s)
P-26	J-23	J-26	377	67.8	421.3	1.35	252.3	0.81
P-27	J-26	J-27	246	57.0	193.3	0.88	115.8	0.53
P-28	J-13	J-28	592	67.8	230.8	0.74	138.2	0.44
P-29	J-28	J-29	400	57.0	66.4	0.30	39.7	0.18
P-31	J-15	J-31	452	57.0	59.3	0.27	35.5	0.16
P-32	J-31	J-16	455	57.0	(40.2)	0.18	(24.1)	0.11
P-33	J-17	J-32	469	184.6	1,559.8	0.67	934.0	0.40
P-34	J-32	J-33	363	184.6	1,343.3	0.58	804.4	0.35
P-35	J-33	J-34	381	147.6	1,154.3	0.78	691.2	0.47
P-36	J-34	J-35	815	147.6	721.4	0.49	432.0	0.29
P-37	J-35	J-36	354	115.4	681.0	0.75	407.8	0.45
P-38	J-36	J-37	792	99.4	424.2	0.63	254.0	0.38
P-39	J-37	J-38	968	81.4	272.7	0.61	163.3	0.36
P-40	J-38	J-39	790	57.0	90.9	0.41	54.4	0.25
P-42	J-36	J-41	306	57.0	141.4	0.64	84.7	0.38
P-43	J-41	J-42	504	57.0	53.4	0.24	32.0	0.14
P-44	J-34	J-43	718	99.4	268.4	0.40	160.7	0.24
P-45	J-43	J-44	866	57.0	89.5	0.41	53.6	0.24
P-48	J-20	J-47	519	57.0	80.8	0.37	48.4	0.22
P-49	J-47	J-48	345	57.0	11.5	0.05	6.9	0.03
P-50	J-14	J-49	471	57.0	95.3	0.43	57.1	0.26
P-51	J-49	J-28	655	57.0	53.4	0.24	32.0	0.15
P-52	J-12	J-50	676	115.4	562.7	0.62	337.0	0.37
P-53	J-50	J-51	1,045	99.4	414.1	0.62	248.0	0.37
P-54	J-51	J-52	553	67.8	161.6	0.52	96.8	0.31
P-56	J-53	J-54	863	230.8	3,390.8	0.94	2,030.4	0.56
P-57	J-54	J-55	704	230.8	2,913.2	0.81	1,744.4	0.48
P-58	J-55	J-56	284	184.6	2,845.4	1.23	1,703.8	0.74
P-59	J-56	J-57	402	184.6	2,604.9	1.13	1,559.8	0.67
P-60	J-57	J-58	859	184.6	2,376.7	1.03	1,423.2	0.62
P-61	J-58	J-59	408	184.6	2,203.3	0.95	1,319.3	0.57
P-62	J-56	J-60	420	57.0	168.3	0.76	100.8	0.46
P-63	J-60	J-61	267	57.0	17.3	0.08	10.4	0.05
P-64	J-60	J-62	473	57.0	78.8	0.36	47.2	0.21
P-65	J-62	J-63	344	57.0	162.8	0.74	97.5	0.44
P-66	J-63	J-64	238	57.0	202.0	0.92	121.0	0.55
P-67	J-64	J-65	573	57.0	92.3	0.42	55.3	0.25
P-68	J-63	J-58	476	57.0	(124.4)	0.56	(74.5)	0.34
P-69	J-57	J-62	375	57.0	156.1	0.71	93.5	0.42
P-70	J-59	J-66	1310	99.4	526.7	0.79	315.4	0.47
P-71	J-59	J-67	1,057	184.6	1,056.2	0.46	632.4	0.27
P-77	J-67	J-72	847	147.6	943.6	0.64	565.1	0.38
P-78	J-72	J-73	605	147.6	907.6	0.61	543.5	0.37
P-79	J-73	J-74	377	115.4	731.5	0.81	438.0	0.48
P-80	J-74	J-75	639	99.4	480.5	0.72	287.7	0.43
P-81	J-75	J-76	292	81.4	272.7	0.61	163.3	0.36
P-82	J-73	J-77	460	57.0	53.4	0.24	32.0	0.14
P-83	J-77	J-78	291	57.0	17.3	0.08	10.4	0.05
P-84	J-76	J-79	229	67.8	190.5	0.61	114.0	0.37
P-85	J-79	J-80	989	57.0	114.0	0.52	68.3	0.31
P-86	J-80	J-81	826	57.0	37.5	0.17	22.5	0.10
P-89	J-82	J-54	671	57.0	(163.0)	0.74	(97.6)	0.44

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
P-90	R-1	PMP-1	32	461.8	15,146.4	1.05	13,604.5	0.94
P-92	J-18	J-84	18	81.4	605.6	1.35	362.6	0.81
P-93	J-84	J-30	734	67.8	158.7	0.51	95.0	0.30
P-94	J-83	J-84	366	57.0	(63.5)	0.29	(38.0)	0.17
P-95	J-84	J-85	791	57.0	73.2	0.33	43.8	0.20
P-96	J-19	J-86	42	207.8	1,837.2	0.63	1,100.1	0.38
P-97	J-86	J-20	647	207.8	1,494.0	0.51	894.6	0.31
P-98	J-85	J-86	19	57.0	(305.7)	1.39	(183.0)	0.83
P-99	J-85	J-87	1329	57.0	70.1	0.32	42.0	0.19
P-100	J-20	J-88	667	147.6	1,176.6	0.80	704.5	0.48
P-101	J-88	J-21	467	81.4	575.7	1.28	344.7	0.77
P-102	J-87	J-88	24	81.4	(348.4)	0.77	(208.6)	0.46
P-103	J-87	J-89	637	57.0	99.6	0.45	59.6	0.27
P-105	PMP-1	J-90	28	461.8	15,146.4	1.05	13,604.5	0.94
P-113	J-91	J-10	647	461.8	13,159.1	0.91	7,879.7	0.54
P-114	PMP-4	J-91	2	461.8	13,159.1	0.91	7,879.7	0.54
P-123	J-92	J-12	0	415.6	8,765.5	0.75	5,248.8	0.45
P-125	PMP-4	J-9	1	461.8	(13,159.1)	0.91	(7,879.7)	0.54
P-128	J-4	J-94	672	461.8	15,881.8	1.10	9,510.0	0.66
P-129	J-94	J-5	521	461.8	15,881.8	1.10	9,510.0	0.66
P-131	J-95	J-96	1347	517.2	29,557.4	1.63	17,699.0	0.98
P-132	J-96	J-97	449	517.2	29,557.4	1.63	17,699.0	0.98
P-133	J-97	J-98	359	517.2	29,557.4	1.63	17,699.0	0.98
P-136	R-1	PMP-5	39	461.8	15,146.4	1.05	13,604.5	0.94
P-143	PMP-5	J-100	23	461.8	15,146.4	1.05	13,604.5	0.94
P-146	J-90	J-99	19	517.2	29,557.4	1.63	17,699.0	0.98
P-147	J-100	J-90	9	461.8	-	0.00	-	0.00
P-148	R-1	PMP-6	37	461.8	15,146.4	1.05	-	0.00
P-149	PMP-6	J-90	30	461.8	15,146.4	1.05	-	0.00
P-151	J-90	FCV-1	11	517.2	15,881.8	0.87	9,510.0	0.52
P-	FCV-	J-1	595	517.2	15,881.8	0.87	9,510.0	0.52

Pipe Label	Start Node	Stop Node	Length	Diameter	Peak demand		Average Demand	
					Flow	Velocity	Flow	Velocity
	Node Label	Node Label	(m)	(mm)	(m^3/day)	(m/s)	(m^3/day)	(m/s)
152	1							
P-153	J-99	FCV-2	6	517.2	29,557.4	1.63	17,699.0	0.98
P-154	FCV-2	J-95	609	517.2	29,557.4	1.63	17,699.0	0.98
P-155	J-11	FCV-3	20	230.8	3,587.0	0.99	2,147.9	0.59
P-156	FCV-3	J-53	737	230.8	3,587.0	0.99	2,147.9	0.59
P-157	J-11	FCV-4	10	415.6	8,765.5	0.75	5,248.8	0.45
P-158	FCV-4	J-92	344	415.6	8,765.5	0.75	5,248.8	0.45

Appendix 3
Preliminary Design Report – Battambang Subproject

Appendix H5 Preliminary Design Report –Battambang Subproject

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1. Introduction

The CDIA TA subproject for Battambang wastewater aims to address the priority areas selected by MPWT and DPWT by designing and constructing a separated sewer network, installing free domestic connections, and providing a new WWTP to enable the WW load to be treated to national effluent quality standards. WW treatment for the wider urban area has been divided into two Phases with the first Phase addressing the main populated areas to 2040. The existing SAWA funded WWTP on a 5.8ha site is to be decommissioned, and a nearby new 6.6ha site developed into a conventional trickling filter based WWTP with pre-settlement. This will provide adequate treatment for Phase 1 to 2040 and provide space for expansion for Phase 2 at a later date. Several options for wastewater treatment capacity are discussed.

2. Current Situation

2.1 Existing system

Battambang currently has a 1,000m^3/d lagoon-based WWTP that was constructed in 1994 through SAWA funding, and was intended to serve 15,000 people over 89ha of the main downtown area on the west side of the river only. It consists of settlement/anaerobic ponds, facultative ponds and effluent disposal to wetlands and a fish pond. This is still operating but is now limited to 450m^3/d as the original pumps are in poor condition and operate only 10 hours per day. The existing WWTP layout is shown in Figure 2 and the service area in Figure 3.

The sewerage network in the town centre is combined and consists of concrete pipe from 800mm – 1500mm installed between the colonial period and recent years. All wastewater not connected to the downtown network flows via a series of open channels to the river and rice fields to the north of town. The DPWT have a good understanding of the system and records of existing pipe sizes, break points and flood prone areas.

The city has grown significantly since the previous WW scheme was installed and what was considered the city centre at that time was little changed from colonial times. The city centre today covers a much larger area, and Battambang has become the second largest city after Phnom Penh. Requirements for wastewater treatment have similarly grown.
The east side of the river has an ongoing ADB preliminary design study under the “GMS Southern Economic Corridor Towns Development Project”.

The existing WWTP layout consists of the three main stages of lagoon treatment process; anaerobic pond, facultative pond and maturation pond or wetland as shown in Figure 4. The existing WWTP covers an area of 5.8ha and lagoon components are as below:

Anaerobic pond: 2 ponds with 5m depth. A pump station was installed at the downstream end.
Facultative pond: 2 circular ponds in parallel, with an average diameter of 63m, and average depth 1.5m with deep fermentation pit (diameter 21m, and 3m depth) in each pond.
Maturation pond: 1 ponds with average depth of 1m and area of 0.5ha.
Wetland: Wetland area of 0.5ha with 70% of the area covered by vegetation - local species including Cyperus, Sesbania rostrata, water hyacinth and Phragmites.

Wastewater stabilisation ponds (WSP’s) or lagoon systems are shallow man-made basins into which wastewater flows and from which, after a retention time of several days (rather than several hours in conventional treatment process), a well-treated effluent is discharged. WSP systems comprise a series of ponds including anaerobic, facultative, and several maturation ponds. The advantage of WSP systems are simplicity, low cost, and high efficiency. If a suitable amount of *cheap land is available*, it is generally acknowledged that they should always be the first choice of technology in developing countries.

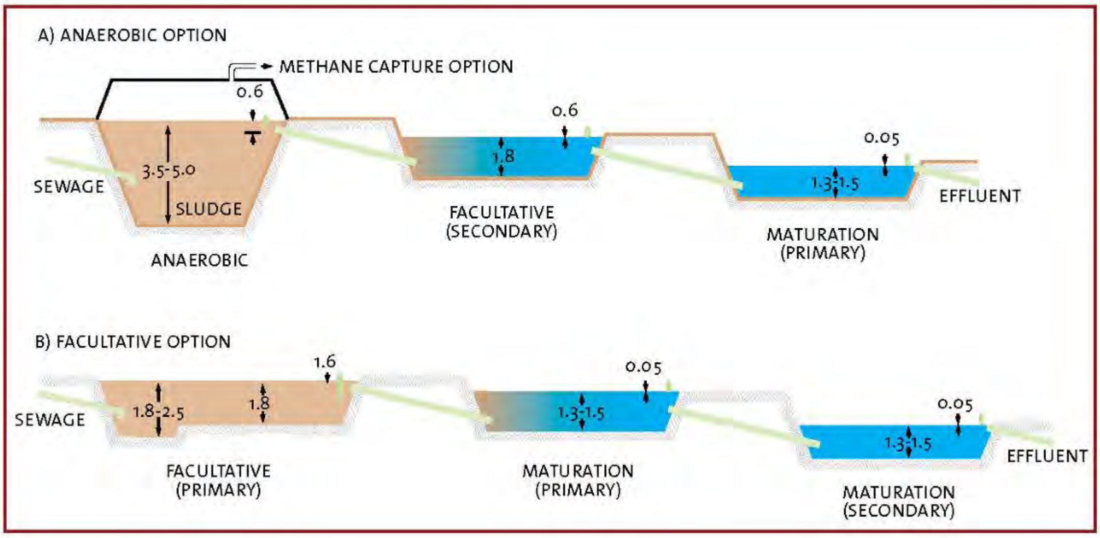
Anaerobic: A pond (normally at least 3-5m deep) where sewage is digested anaerobically (in the absence of oxygen).

Facultative: A pond (normally 1.5m to 2.5m deep) where both anaerobic and aerobic digestion of sewage takes place

Maturation: A pond (normally 0.9-1.5m deep) primarily responsible for pathogen removal by various mechanisms, including UV disinfection and daily high pH levels.

Figure 1 shows a schematic of a standard WSP process, which is in common use around the world. Waste stabilization ponds (WSP) with a series of anaerobic, facultative, and maturation ponds were selected as the best option, providing good levels of treatment with the lowest possible operating cost for Battambang.

Figure 1 - Schematic of Waste Stabilization Ponds



(source: Waste Stabilization Ponds, design manual, 19 Dec. 2011, by J.Ashworth; M.Skinner)

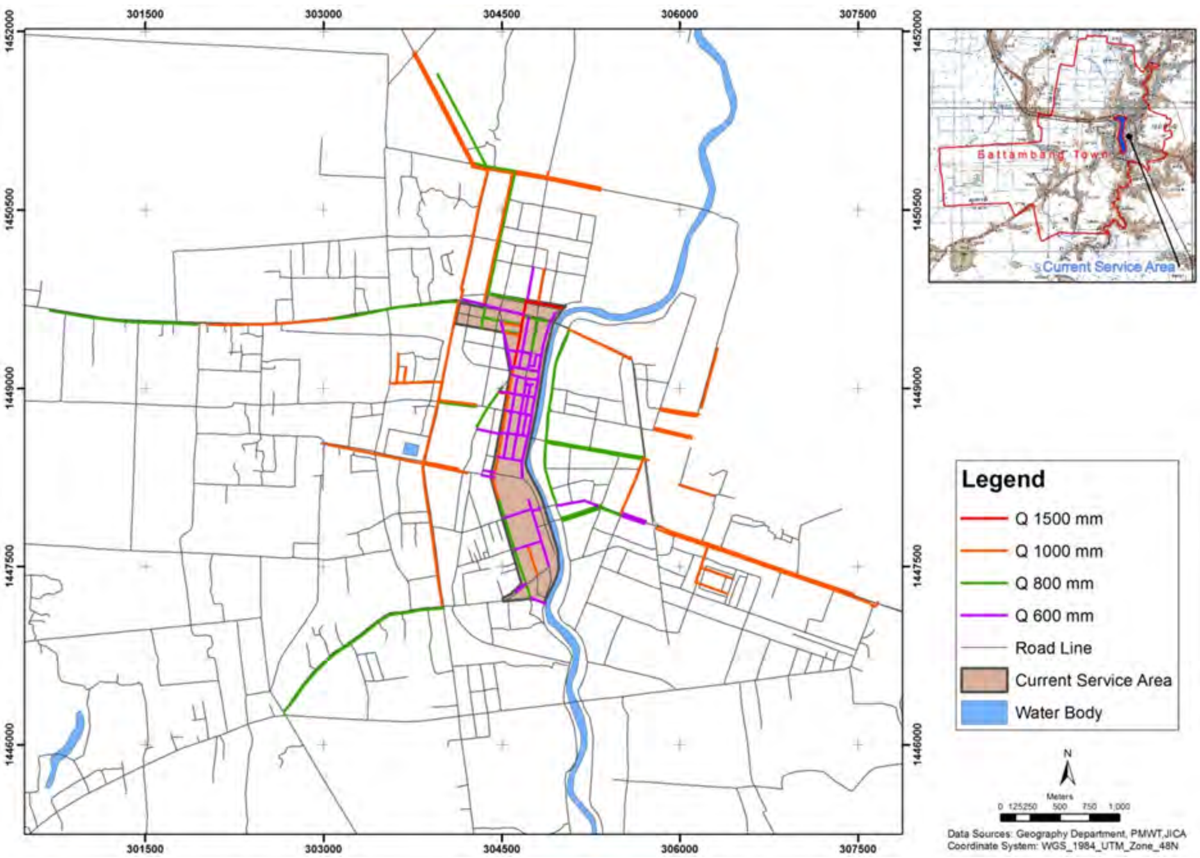
The WWTP is still in operation and has a full time operator on-site, but the pumps and rising mains that deliver WW from the anaerobic ponds to the aerobic ponds have been repaired many times and are very inefficient, reducing the WWTP capacity to 450m^3/day. The anaerobic ponds are manually de-sludged but not in a routine manner and are in need of excavation and rehabilitation of the bunds.

The operators building is in a poor state and is no longer fit for purpose.

The area still has a low population density, and there have been no reported smell complaints. During three site visits April – June 2016 no smells were noticed.

Figure 2 - Existing 1994 WWTP

Figure 3 - Coverage area for 1994 SAWA WW project



2.2 Population

Population has gradually increased with average annual growth rate of 1.31% from 2009 to 2014. The annual growth rate west of the river was approximately 1.31% and east of the river approximately 1.36%. The female:male ratio was 51% in 2014. The commercial facilities increased up until the last few years, but has remained almost constant since. The population statistics and commercial facilities are shown in Table 1 and Table 2, respectively.

Table 1 - Population in Battambang city

Sangkat	2009		2010		2011		2012		2013		2014	
	Family	Pop.	Family	Pop.	Family	Pop.	Family	Pop.	Family	Pop.	Family	Pop.
Westerm River	**20,730**	**116,856**	**20,879**	**118,378**	**21,312**	**118,160**	**21,473**	**118,829**	**21,735**	**121,675**	**22,425**	**124,673**
Chamkar Samraong	3,213	16,356	3,218	17,179	3,249	17,722	3,288	17,799	3,307	18,254	3,430	18,512
Kdol Doun Teav	1,809	9,072	1,839	9,359	1,874	9,403	1,880	9,357	1,945	9,716	2,026	9,876
Ou Char	2,776	17,049	2,801	16,993	2,890	17,836	2,917	17,971	2,872	17,840	2,878	17,865
Ou Mal	2,228	10,523	2,286	10,720	2,270	10,877	2,364	10,971	2,460	11,140	2,474	11,430
Sla Kaet	1,392	6,897	1,353	6,474	1,373	7,003	1,386	7,476	1,379	7,781	1,401	8,152
Svay Pao	3,103	23,521	3,102	23,532	3,262	21,064	3,121	21,033	3,135	21,338	3,283	21,588
Tuol Ta Aek	3,417	17,343	3,499	18,276	3,612	18,676	3,712	18,530	3,793	19,144	3,992	19,173
Voat Kor	2,792	16,095	2,781	15,845	2,782	15,579	2,805	15,692	2,844	16,462	2,941	18,077
Eastern River	**5,150**	**27,197**	**5,184**	**27,846**	**5,328**	**29,322**	**5,304**	**28,833**	**5,336**	**29,036**	**5,342**	**29,054**
Preaek Preah Sdach	2,463	12,549	2,466	13,277	2,527	13,987	2,503	13,543	2,495	13,842	2,500	13,836
Rotanak	2,687	14,648	2,718	14,569	2,801	15,335	2,801	15,290	2,841	15,194	2,842	15,218
Total	**25,880**	**144,053**	**26,063**	**146,224**	**26,640**	**147,482**	**26,777**	**147,662**	**27,071**	**150,711**	**27,767**	**153,727**

Source: CDB database, 2009-2014

Table 2 - Commercial facilities in Battambang city

Sangkhat Name	2009					2010				
	Market	Clinic	Hotel	Gesthou	Restaur	Market	Clinic	Hotel	Gesthou	Restaur
Westerm River	11	7	12	16	23	11	8	13	20	22
Chamkar Samraong	2	0	0	3	6	2	0	0	3	4
Kdol Doun Teav	3	0	0	0	0	3	0	0	0	0
Ou Char	0	0	1	3	0	0	0	1	3	0
Ou Mal	1	0	0	0	0	1	0	0	0	0
Sla Kaet	1	0	1	0	0	1	0	1	1	0
Svay Pao	3	5	10	9	17	3	6	11	12	18
Tuol Ta Aek	1	2	0	0	0	1	2	0	0	0
Voat Kor	0	0	0	1	0	0	0	0	1	0
Eastern River	1	1	10	9	13	2	2	11	12	24
Preaek Preah Sdach	1	0	0	3	2	1	0	0	3	2
Rotanak	0	1	10	6	11	1	2	11	9	22
Total	12	8	22	25	36	13	10	24	32	46

Sangkhat Name	2011					2012					2013				
	Market	Clinic	Hotel	Gesthou	Restaur	Market	Clinic	Hotel	Gesthou	Restaur	Market	Clinic	Hotel	Gesthou	Restaura
Westerm River	18	8	15	39	33	17	10	15	48	26	18	14	18	46	27
Chamkar Samraong	2	0	0	2	2	2	1	0	0	1	2	2	0	1	1
Kdol Doun Teav	3	0	0	0	2	3	0	0	0	0	3	0	0	0	0
Ou Char	1	0	1	4	2	1	0	1	4	0	2	0	1	2	0
Ou Mal	1	0	0	0	1	1	0	0	0	1	1	0	0	0	0
Sla Kaet	1	0	1	2	7	1	0	1	2	6	1	0	1	4	8
Svay Pao	5	8	13	31	8	5	9	13	34	9	4	11	16	33	9
Tuol Ta Aek	2	0	0	0	2	2	0	0	5	3	3	0	0	4	2
Voat Kor	3	0	0	0	9	2	0	0	3	6	2	1	0	2	7
Eastern River	25	2	15	21	24	3	4	14	24	19	3	4	14	36	15
Preaek Preah Sdach	2	0	0	5	8	1	1	0	9	10	1	0	0	19	5
Rotanak	23	2	15	16	16	2	3	14	15	9	2	4	14	17	10
Total	43	10	30	60	57	20	14	29	72	45	21	18	32	82	42

Source: CDB database, 2009-2013

The populations of the selected service area and projections are provided under Section 4.

2.3 Industry

There is no significant large industry in the city centre area of Battambang. There are small scale water bottling plants and rice mills but these are considered commercial demand for the sake of calculating wastewater demand.

2.4 Effluent discharge standard

The Ministry of Environment, through its Pollution Control Department (PCD), sets effluent discharge standards in Cambodia. The PCD obtained approval for a comprehensive set of wastewater discharge standards in April 1999 under the Sub-decree on Water Pollution Control.
Table 3 summarizes the effluent standards for effluent discharge into a public water area.

Table 3 - Effluent standards in Cambodia

Test	Unit	Discharge to Protected public water area (Std1)	Discharge to public water area and sewer (Std2)
pH		6-9	5-9
BOD	Mg/l	<30	<80
COD	Mg/l	<50	<100
TSS	Mg/l	<60	<120
TDS	Mg/l	<1000	<2000
Grease & oil	Mg/l	<5	<15
Detergents	Mg/l	<5	<15
Nitrate	Mg/l	<5	<20
Phosphate	Mg/l	<3	<6
Ammonia	Mg/l	<5	<7

Source: Sub-Decree on Water Pollution Control (Council of Ministers No. ANRK.BK -06 April 1999), Annex No 2

3 Summary of Current Situation and Proposed Subproject Activities

Table 4 summarises the current situation, shortfalls in service, and the solutions to these shortfalls offered under the proposed subproject.

Table 4 - Current and Proposed situation for sanitation in West Battambang

Current (1994) project)	Shortfall	Proposed
Main colonial-era city centre area served	Very limited service area	Two phases of development: city centre area followed by wider urban area.
Combined system	Very large flows during wet season, with combined flows flooding town and entering environment untreated.	A separated system is proposed
Trunk sewers sized for city centre only, with restrictions in some areas.	Insufficient or wrongly configured trunk sewers	New main trunk sewers
Old pumps between anaerobic ponds and facultative ponds are limiting capacity of existing WWTP	Pumps operating at 450m3/d instead of designed 1,000m3/d	Decommissioning of existing 5.8ha WWTP lagoon site.
WWTP designed for 1,000m3/d which was intended to serve 15,000 people or 3,125 households. Now reduced to 450m3/d serving 1,400 households	11,645m3/d required for Phase 1; 20,694m3/d required for full city to 2040	Decommissioning of existing 5.8ha WWTP lagoon site. Development of conventional trickling filters for proposed new 6.6ha site, staged for 2 Phases. Development of aerated lagoons if a minimum of 15ha becomes available. Phase 1 will serve 40,311 people on commissioning in 2020 and 57,803 by 2040.
Current WWTP land was fully utilized by 1994 project	No further reserved land for expansion	A further 6.6ha site is approved by District Governor
Septic tank septage disposed of to agricultural land	No septage management	Each settlement pond to have septage disposal bay with concrete apron and service water for cleaning.
7 private septage vacuum trucks operating	No regulation of private septage trucks. No DPT vacuum truck.	Provide one 6m3 septage vacuum truck for DPWT. Develop regulation of private septage trucks under Capacity Building
No sludge removal or drying facility	No equipment or space on site	Include sludge drying beds plus supply of dewatering container and compact excavator.
Limited capacity in WW treatment & operation	Small number of expert staff	Capacity building for all DPWT staff
Low level of public awareness on wastewater	Septic tanks not maintained, some leaking, too small etc	Public awareness campaign through different media.

4 Proposed Coverage Area

4.1 Prioritized Subprojects

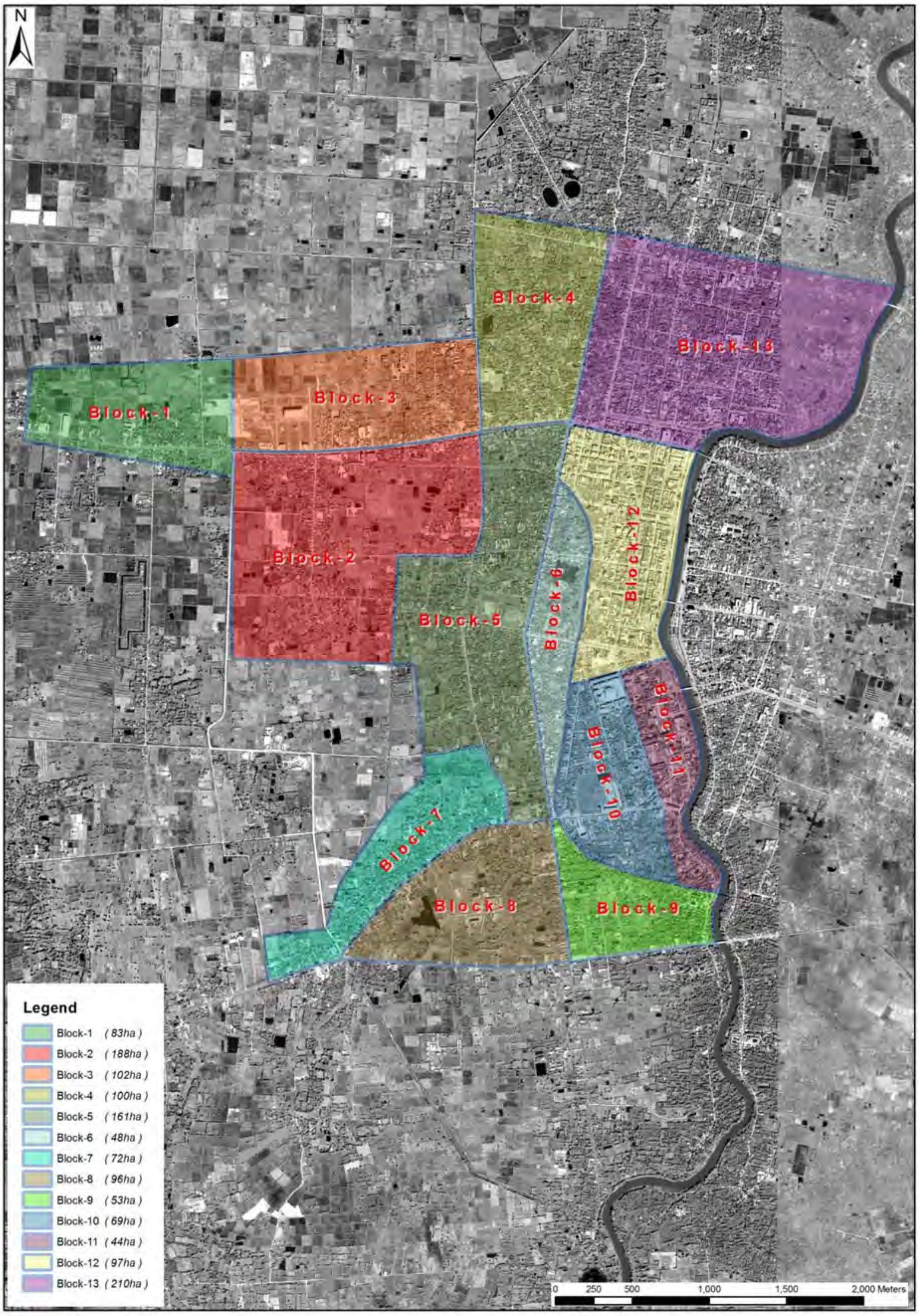
This subproject is targeted to improve the wastewater management on the west bank of the river. This area consists of 8 sangkats and 46 villages. Amongst the 8 sangkat, 3 sangkats are in the center of town and are densely populated, whereas the other 5 sangkats are sub-urban areas, reducing in population density further from the city centre. One of these 5 (Kdol Doun Taev) has no population inside the coverage area, and so is not considered under the design. The proposed wider service area then consists of 7 of the 8 sangkhats in Battambang.
The boundary of the wider coverage service area is shown in Figure 4, and the proportion of each sangkat in the service area is presented in Table 5.

Table 5 - Wider coverage service areas

No.	Sangkat Data		Covered by proposed project	
	Name	Area (ha)	Area (ha)	Cover (%)
1	Chamkar Samraong	925	229	24.7
2	Svay Pao	217	205	94.5
3	Tuol Ta Aek	276	258	93.5
4	Voat Kor	2,648	149	5.6
5	Ou Char	1,021	412	40.3
6	Ou Mal	3,942	19	0.5
7	Sla Kaet	714	45	6.3
8	Kdol Doun Teav	906	0	0.0
Total		10,649	1,316	12.4

Purely to facilitate the calculation of the wastewater generation, runoff and sizing of the sewer trunk mains, the proposed service areas have been divided into 13 “Blocks”, divided by main roads, as shown in Figure 4. How these blocks correspond to the sangkhats is shown in Table 7.

Figure 4 - Service areas divided into 13 "Blocks"for the purpose of calculation

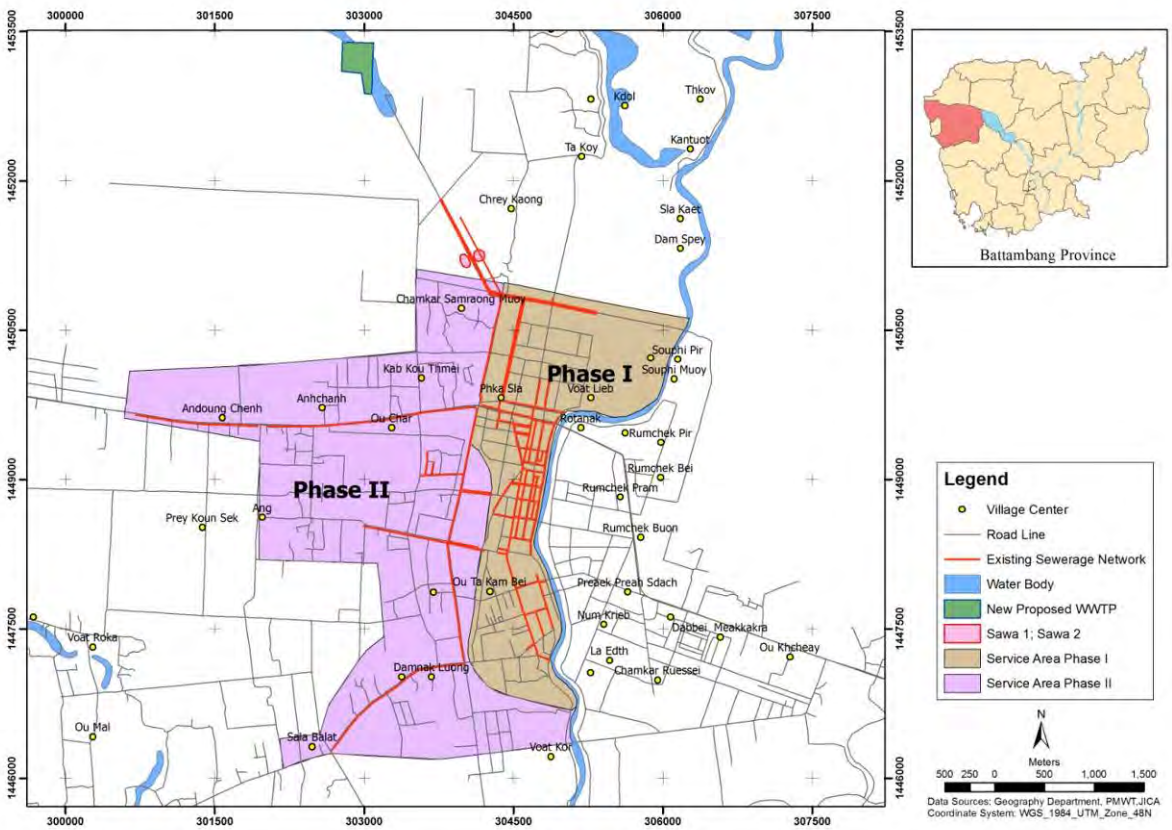


4.2 Phasing of Coverage

As Battambang is the second largest city of Cambodia, it is proposed to plan the wastewater system in 2 phases, the first being the main urban area between the railway line to the west and

the river to the east. The railway line is bunded and acts to create separate catchments either side of its route, and so provides a logical boundary for the proposed Phase 1. The area encompassed by Phase 1 includes all of the densely populated areas of the city centre and most of the intermediately populated areas just outside of the centre. On Figure 5, this area includes Blocks 10-13 inclusive. Blocks 1-9 inclusive constitute Phase 2 of the proposed subproject.
Figure 5 shows the Phase 1 and 2 areas of service in more detail;

Figure 5 - Phase 1 service area



4.3 Phasing of Coverage Wastewater Generation Calculations – full service area- Phases 1 and 2.

This section provides calculations for the full proposed service area (Phases 1 and 2) - all generated wastewater from all 13 “Blocks”, from both domestic and commercial facilities. The design capacity is 20 years from the project commissioning (approximately 2020) design year 2040.

4.2.1 Domestic population served and wastewater production

The population data for the study area (Phases 1 and 2) were collected from 2009 - 2014 and used for the estimation of population distribution for each sub-catchment and population projections. The annual growth rate is based on the average growth rate from 2009 to 2014. The coverage service areas and population in 2014 by sangkat is shown in Table 6 and by “Block” is shown in Table 7. The projected population from 2016 to 2040 is shown in Table 8.

Table 6 - Coverage area and population by sangkhat, full proposed service area

No.	Sub-catchment	Population data 2014(1)		Coverage area			Served Population 2014	
		Total Family	Total People	Total area (ha)	Coverage area (ha)	(%)	Family	Person
1	Chamkar Samraong	3,430	18,512	925	229	25	2,497	13,734
2	Svay Pao	3,283	21,588	217	205	94	3,925	21,588
3	Tuol Ta Aek	3,992	19,173	276	258	93	2,936	16,148
4	Voat Kor	2,941	18,077	2,648	149	6	783	4,304
5	Ou Char	2,878	17,865	1,021	412	40	3,041	16,725
6	Ou Mal	2,474	11,430	3,942	19	0	108	596
7	Sla Kaet	1,401	8,152	714	45	6	550	3,023
8	Kdol Doun Teav	2,026	9,876	906	0	0	0	0
Total		22,425	124,673	10,649	1,316	12.4	13,840	76,118

Table 7 - Coverage area and population by Block, full proposed service area

No.	Block	Sangkat	Coverage area	Served Population 2014	
			ha	Family	Person
1	B-01	Ou Char	83	477	2,621
2	B-02	Ou Char	175	897	4,936
		Toul Ta Aek	12	76	417
3	B-03	Ou Char	101	480	2,642
4	B-04	Chamkar Songraong	53	497	2,731
		Sla ket	10	91	503
		Ou Char	36	805	4,430
5	B-05	Toul Ta Aek	144	1177	6,476
		Ou Char	16	381	2,095
6	B-06	Toul Ta Aek	49	1045	5,747
7	B-07	Ou Mal	19	109	597
		Toul Ta Aek	53	385	2,116
8	B-08	Voat Kor	96	515	2,834
9	B-09	Voat Kor	53	267	1,470
10	B-10	Svay Pao	69	917	5,041
11	B-11	Svay Pao	42	518	2,850
12	B-12	Svay Pao	94	2744	15,091
13	B-13	Chamkar Songraong	176	2001	11,003
		Sla ket	34	458	2,520

Table 8 - Projected populations by Block, full proposed service area (Phases 1 and 2).

Block	Growth rate (%)	Present served population (baseline)	Projected Population – Phases 1 and 2 together				
		2016	2020	2025	2030	2035	2040
B-01	0.96	2,672	2,776	2,911	3,054	3,203	3,360
B-02	0.96	5,456	5,669	5,946	6,237	6,542	6,862
B-03	0.96	2,693	2,798	2,935	3,078	3,229	3,387
B-04	1.74	7,933	8,500	9,265	10,100	11,010	12,002
B-05	1.51	8,832	9,377	10,107	10,894	11,741	12,655
B-06	2.05	5,985	6,491	7,184	7,952	8,801	9,740
B-07	1.86	2,815	3,030	3,323	3,643	3,995	4,381
B-08	2.44	2,974	3,275	3,695	4,168	4,702	5,304
B-09	2.44	1,543	1,699	1,916	2,162	2,439	2,751
B-10	0.82	5,124	5,294	5,515	5,745	5,984	6,233
B-11	0.82	2,897	2,993	3,118	3,248	3,383	3,524
B-12	0.82	15,340	15,849	16,509	17,197	17,914	18,661
B-13	3.03	14,355	16,175	18,779	21,802	25,311	29,385
Total	-	78,617	83,926	91,204	99,280	108,255	118,246

Based on the current water supply data, the daily water consumption is about 140 litres/capita. The water consumption rate is assumed to gradually increase 1 liter/capita/year. A wastewater return rate of 80% of the total daily water consumption is used for the calculation of domestic wastewater generation. The commissioning of the sanitation subproject component is expected at the end of year 2020, and the design life is 20 years (2040). The WW generation in each sub-catchment by target year is shown in Table 9.

Table 9 - Domestic WW generation 2016-2040, full proposed service area

Service Areas/Block	2016	2020	2025	2030	2035	2040
B-01						
Served population	2,672	2,776	2,911	3,054	3,203	3,360
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**299**	**311**	**338**	**366**	**397**	**430**
B-02						
Served population	5,456	5,669	5,946	6,237	6,542	6,862
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**611**	**635**	**690**	**748**	**811**	**878**
B-03						
Served population	2,693	2,798	2,935	3,078	3,229	3,387
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**302**	**313**	**340**	**369**	**400**	**434**
B-04						
Served population	7,933	8,500	9,265	10,100	11,010	12,002
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**888**	**952**	**1,075**	**1,212**	**1,365**	**1,536**
B-05						
Served population	8,832	9,377	10,107	10,894	11,741	12,655
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**989**	**1,050**	**1,172**	**1,307**	**1,456**	**1,620**
B-06						
Served population	5,985	6,491	7,184	7,952	8,801	9,740
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**670**	**727**	**833**	**954**	**1,091**	**1,247**
B-07						
Served population	2,815	3,030	3,323	3,643	3,995	4,381
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**315**	**339**	**385**	**437**	**495**	**561**
B-08						
Served population	2,974	3,275	3,695	4,168	4,702	5,304
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**333**	**367**	**429**	**500**	**583**	**679**
B-09						

Service Areas/Block	2016	2020	2025	2030	2035	2040
Served population	1,543	1,699	1,916	2,162	2,439	2,751
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**173**	**190**	**222**	**259**	**302**	**352**
B-10						
Served population	5,124	5,294	5,515	5,745	5,984	6,233
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**574**	**593**	**640**	**689**	**742**	**798**
B-11						
Served population	2,897	2,993	3,118	3,248	3,383	3,524
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**324**	**335**	**362**	**390**	**420**	**451**
B-12						
Served population	15,340	15,849	16,509	17,197	17,914	18,661
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**1,718**	**1,775**	**1,915**	**2,064**	**2,221**	**2,389**
B-13						
Served population	14,355	16,175	18,779	21,802	25,311	29,385
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	1,608	1,812	2,178	2,616	3,139	3,761
Grand Total (m3/day)	**8,805**	**9,400**	**10,580**	**11,914**	**13,424**	**15,136**

4.2.2 Served commercial/tourism facilities and demands

The new service area covers both the central town and less populated peri-urban areas. There are many commercial facilities located in these areas, divided into five categories for the purposes of the calculations; market, restaurant, hotel, guesthouse, and health clinic.

The commercial facilities data is available from 2009 to 2013 and has been used to estimate future growth. The annual growth rate is based on the average growth rate from 2009 to 2013. However, the growth rate of markets (large and medium) has been assumed to have no increase, as the total number of markets has remained constant over the last five years. The growth rate of hotels, restaurants, guesthouses and health clinics were found to increase significantly over the past 5 years. However, this growth rate is not expected to maintain this upward trend to 2040, and has slowed in recent years. The 2013 data has been used as a baseline and projected to 2040. The number of facilities in each category is presented in Table 10, and the growth rate and projected numbers of facilities are shown in Table 11 and Table 12, respectively.

Table 10 - Commercial facilities in each Block in 2016

Block	Market-large	Market-medium	Market-small	Health clinic	Hotel	Guesthouse	Restaurant	Total
B-01	0	0	0	0	0	0	0	0
B-02	1	0	1	0	1	1	0	4
B-03	0	0	0	0	0	1	0	1
B-04	0	0	0	1	0	1	2	5
B-05	0	1	1	0	0	2	1	5
B-06	0	0	1	0	0	1	1	3
B-07	0	0	0	0	0	1	0	1
B-08	0	0	1	1	0	1	4	8
B-09	0	0	1	0	0	1	3	6
B-10	0	1	0	2	5	9	2	20
B-11	0	0	0	1	1	2	1	6
B-12	1	2	0	9	14	27	7	59
B-13	0	1	2	1	1	3	4	13
Total	2	5	7	16	23	52	27	132

Table 11 - Annual growth rate of each category based on data 2009-2013

Categories	Growth rate (%)
Market-Large	0.0
Market-Medium	0.0
Market-Small	1.8
Health Clinic	5.0
Hotel	7.9
Guesthouse	5.0
Restaurant	3.8

Table 12 - Projected commercial facilities in each Block by 2040

Block	2016	2020	2025	2030	2035	2040
B-01	0	0	0	0	0	0
B-02	4	5	7	8	11	14
B-03	1	1	2	2	3	4
B-04	5	5	7	8	10	13
B-05	5	6	7	9	11	13
B-06	3	4	5	6	7	8
B-07	1	1	2	2	3	4
B-08	8	9	11	13	16	20
B-09	6	6	8	9	11	14
B-10	20	24	32	42	56	75
B-11	6	7	9	12	16	22
B-12	59	73	95	125	166	221
B-13	13	16	19	24	30	38
Total	132	160	204	263	340	445

The water consumption unit and rate of each commercial facility are estimated based on literature values and data from direct interview from local authorities. The unit of water consumption and the number of units in each category is presented in Table 13. The wastewater return rate of 80% of the total water consumption is used for the estimation of wastewater generation from commercial facilities. The results of the commercial wastewater generation calculations are shown in Table 14.

Table 13 - Water consumption per unit by commercial users

Categories	Unit	Number/unit	Water use (L/unit/day)
Market-Large	person	1000	44
Market-Medium	stall	300	57
Market-Small	stall	150	57
Health Clinic	bed	10	833
Hotel	room	50	416
Guesthouse	room	30	144
Restaurant	seat	50	76

Table 14 - Wastewater generation by commercial users

Service Areas/Block	2016	2020	2025	2030	2035	2040
B-01						
No. of commercial facilities	0	0	0	0	0	0
Water consumption amount (m3/day)	0	0	0	0	0	0
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**0**	**0**	**0**	**0**	**0**	**0**
B-02						
No. of commercial facilities	4	5	7	8	11	14
Water consumption amount (m3/day)	84	95	114	141	180	236

Service Areas/Block	2016	2020	2025	2030	2035	2040
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**67**	**76**	**91**	**113**	**144**	**188**
B-03						
No. of commercial facilities	1	1	2	2	3	4
Water consumption amount (m3/day)	50	6	8	10	13	16
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**40**	**5**	**6**	**8**	**10**	**13**
B-04						
No. of commercial facilities	5	5	7	8	10	13
Water consumption amount (m3/day)	23	28	35	43	54	68
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**18**	**22**	**28**	**35**	**43**	**54**
B-05						
No. of commercial facilities	5	6	7	9	11	13
Water consumption amount (m3/day)	40	44	49	55	63	73
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**32**	**35**	**39**	**44**	**51**	**59**
B-06						
No. of commercial facilities	3	4	5	6	7	8
Water consumption amount (m3/day)	18	21	24	29	34	40
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**15**	**17**	**19**	**23**	**27**	**32**
B-07						
No. of commercial facilities	1	1	2	2	3	4
Water consumption amount (m3/day)	5	6	8	10	13	16
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**4**	**5**	**6**	**8**	**10**	**13**
B-08						
No. of commercial facilities	8	9	11	13	16	20
Water consumption amount (m3/day)	41	47	57	69	84	102
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**32**	**38**	**46**	**55**	**67**	**82**
B-09						
No. of commercial facilities	6	6	8	9	11	14
Water consumption amount (m3/day)	27	30	36	43	51	61
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**21**	**24**	**29**	**34**	**41**	**49**
B-10						
No. of commercial facilities	20	24	32	42	56	75
Water consumption amount (m3/day)	189	241	328	452	627	878
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**151**	**193**	**263**	**362**	**502**	**702**
B-11						

Service Areas/Block	2016	2020	2025	2030	2035	2040
No. of commercial facilities	6	7	9	12	16	22
Water consumption amount (m3/day)	50	64	88	122	169	236
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**40**	**51**	**71**	**97**	**135**	**189**
B-12						
No. of commercial facilities	59	73	95	125	166	221
Water consumption amount (m3/day)	583	730	981	1,335	1,834	2,543
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m^3/day)	466	584	785	1,068	1,467	2,034
B-13						
No. of commercial facilities	13	16	19	24	30	38
Water consumption amount (m3/day)	103	121	152	193	250	328
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	82	97	121	155	200	262
Grand Total (m3/day)	**970**	**1,146**	**1,504**	**2,001**	**2,698**	**3,677**

4.2.3 Served Industry

There is no significant industry in Battambang urban area other than small scale water bottling plants and rice milling, which do not produce significant quantities of wastewater. They have therefore not been considered as a separate category of wastewater production.

4.2.4 Summary of wastewater generation: Full service area - Phases 1 & 2

The total WW generation is from both domestic and commercial, plus infiltration into the sewerage system due to groundwater and storm water. The infiltration rate is assumed to be 10% of the total WW generation. The summary of WW generation is shown in Table 15.

Table 15 - Total WW generation 2016-2040

Service Areas/Block	2016	2020	2025	2030	2035	2040
B-01						
Domestic WW	299	311	338	366	397	430
Commercial WW	-	-	-	-	-	-
Sub-Total (m3/day)	**299**	**311**	**338**	**366**	**397**	**430**
B-02						
Domestic WW	611	635	690	748	811	878
Commercial WW	67	76	91	113	144	188
Sub-Total (m3/day)	**678**	**711**	**781**	**861**	**955**	**1,067**
B-03						
Domestic WW	302	313	340	369	400	434
Commercial WW	40	5	6	8	10	13
Sub-Total (m3/day)	**342**	**318**	**347**	**377**	**410**	**446**
B-04						
Domestic WW	888	952	1,075	1,212	1,365	1,536
Commercial WW	18	22	28	35	43	54
Sub-Total (m3/day)	**907**	**974**	**1,102**	**1,247**	**1,409**	**1,591**

Service Areas/Block	2016	2020	2025	2030	2035	2040
B-05						
Domestic WW	989	1,050	1,172	1,307	1,456	1,620
Commercial WW	32	35	39	44	51	59
Sub-Total (m3/day)	**1,021**	**1,085**	**1,212**	**1,352**	**1,507**	**1,679**
B-06						
Domestic WW	670	727	833	954	1,091	1,247
Commercial WW	15	17	19	23	27	32
Sub-Total (m3/day)	**685**	**744**	**853**	**977**	**1,118**	**1,279**
B-07						
Domestic WW	315	339	385	437	495	561
Commercial WW	4	5	6	8	10	13
Sub-Total (m3/day)	**319**	**344**	**392**	**445**	**505**	**574**
B-08						
Domestic WW	333	367	429	500	583	679
Commercial WW	32	38	46	55	67	82
Sub-Total (m3/day)	**366**	**404**	**474**	**555**	**650**	**761**
B-09						
Domestic WW	173	190	222	259	302	352
Commercial WW	21	24	29	34	41	49
Sub-Total (m3/day)	**194**	**215**	**251**	**294**	**343**	**401**
B-10						
Domestic WW	574	593	640	689	742	798
Commercial WW	151	193	263	362	502	702
Sub-Total (m3/day)	**725**	**785**	**902**	**1,051**	**1,244**	**1,500**
B-11						
Domestic WW	324	335	362	390	420	451
Commercial WW	40	51	71	97	135	189
Sub-Total (m3/day)	**364**	**387**	**432**	**487**	**555**	**640**
B-12						
Domestic WW	1,718	1,775	1,915	2,064	2,221	2,389
Commercial WW	466	584	785	1,068	1,467	2,034
Sub-Total (m3/day)	**2,184**	**2,359**	**2,700**	**3,131**	**3,689**	**4,423**
B-13						
Domestic WW	1,608	1,812	2,178	2,616	3,139	3,761
Commercial WW	82	97	121	155	200	262
Sub-Total (m3/day)	1,690	1,909	2,300	2,771	3,338	4,023
Total (m3/day)	**9,775**	**10,546**	**12,083**	**13,915**	**16,121**	**18,813**
Infiltration Rate-10% (m3/day)	977	1,055	1,208	1,391	1,612	1,881
Grand Total (m3/day)	**10,752**	**11,601**	**13,292**	**15,306**	**17,733**	**20,694**

4.3 Wastewater Generation Calculations – Phase 1 area only

Phase 1 encompasses the main urban centre as shown in Figure 5 above, and is proposed as the area to serve initially, in the 2020-2040 timeframe. The calculations above in Section 4.3 are for the city as a whole, ie. Phases 1 and 2, and the following calculations and tables under Section 4.4 are for Phase 1 area only.

The following tables follow the same format as Section 4.3 for the entire proposed service area (Phases 1 and 2) and the descriptions provided under Section 4.3 apply equally here.

4.3.1 Domestic population served and wastewater production: Phase 1

Table 16 shows the proportion of each sangkhat covered by the Phase 1 blocks 10-13.

Table 16 - Coverage area and population by block: Phase 1

No.	Block	Sangkat	Coverage area	Served Population 2014	
			ha	Family	Person
10	B-10	Svay Pao	69	917	5,041
11	B-11	Svay Pao	42	518	2,850
12	B-12	Svay Pao	94	2744	15,091
13	B-13	Chamkar Songraong	176	2001	11,003
		Sla ket	34	458	2,520

Table 17 - Projected populations by block: Phase 1

Block	Growth rate (%)	Present served population (baseline)	Projected Population: Phase 1 only				
		2016	2020	2025	2030	2035	2040
B-10	0.82	5,124	5,294	5,515	5,745	5,984	6,233
B-11	0.82	2,897	2,993	3,118	3,248	3,383	3,524
B-12	0.82	15,340	15,849	16,509	17,197	17,914	18,661
B-13	3.03	14,355	16,175	18,779	21,802	25,311	29,385
Total	-	37,716	40,311	43,921	47,992	59,592	57,803

Table 18 - Domestic wastewater generation: Phase 1

Service Areas/Block	2016	2020	2025	2030	2035	2040
B-10						
Served population	5,124	5,294	5,515	5,745	5,984	6,233
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**574**	**593**	**640**	**689**	**742**	**798**
B-11						
Served population	2,897	2,993	3,118	3,248	3,383	3,524
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**324**	**335**	**362**	**390**	**420**	**451**
B-12						
Served population	15,340	15,849	16,509	17,197	17,914	18,661
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**1,718**	**1,775**	**1,915**	**2,064**	**2,221**	**2,389**
B-13						
Served population	14,355	16,175	18,779	21,802	25,311	29,385
Water consumption rate lpc	140	140	145	150	155	160
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**1,608**	**1,812**	**2,178**	**2,616**	**3,138**	**3,761**
Grand Total (m3/day)	**4,224**	**4,515**	**5,095**	**5,759**	**6,521**	**7,399**

Number of connections required for 2020 can be determined by dividing total 2020 population given here by 4.8

4.3.2 Served commercial/tourism facilities and demands: Phase 1

Table 19 - Commercial facilities in each block for Phase 1

Block	Market-large	Market-medium	Market-small	Health clinic	Hotel	Guesthouse	Restaurant
B-10	0	1	0	2	5	9	2
B-11	0	0	0	1	1	2	1
B-12	1	2	0	9	14	27	7
B-13	0	1	2	1	1	3	4
Total	1	4	2	13	21	41	14

Table 20 - Annual growth rate of each category based on data 2009-13

Categories	Growth rate (%)
Market-Large	0.0
Market-Medium	0.0
Market-Small	1.8
Health Clinic	5.0
Hotel	7.9
Guesthouse	5.0
Restaurant	3.8

Table 21 - Projected commercial facilities by block to 2040 for Phase 1

Block	2016	2020	2025	2030	2035	2040
B-10	20	24	32	42	56	75
B-11	6	7	9	12	16	22
B-12	59	73	95	125	166	221
B-13	13	16	19	24	30	38
Total	98	120	155	203	268	356

Table 22 - Water use per unit by commercial users

Categories	Unit	Number/unit	Water use (L/unit/day)
Market-Large	person	1000	44
Market-Medium	stall	300	57
Market-Small	stall	150	57
Health Clinic	bed	10	833
Hotel	room	50	416
Guesthouse	room	30	144
Restaurant	seat	50	76

Table 23 - Wastewater generation by commercial users for Phase 1

Service Areas/Block	2016	2020	2025	2030	2035	2040
B-10						
No. of commercial facilities	20	24	32	42	56	75
Water consumption amount (m3/day)	189	241	328	452	627	878
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**151**	**193**	**263**	**362**	**502**	**702**
B-11						
No. of commercial facilities	6	7	9	12	16	22
Water consumption amount (m3/day)	50	64	88	122	169	236
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**40**	**51**	**71**	**97**	**135**	**189**
B-12						

Service Areas/Block	2016	2020	2025	2030	2035	2040
No. of commercial facilities	59	73	95	125	166	221
Water consumption amount (m3/day)	583	730	981	1,335	1,834	2,543
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**466**	**584**	**785**	**1,068**	**1,467**	**2,034**
B-13						
No. of commercial facilities	13	16	19	24	30	38
Water consumption amount (m3/day)	103	121	152	193	250	328
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	**82**	**97**	**121**	**155**	**200**	**262**
Grand Total (m3/day)	**739**	**925**	**1,240**	**1,682**	**2,304**	**3,187**

4.3.3Summary of wastewater generation: Phase 1

Table 24 - Summary of wastewater generation for Phase 1 – central city

Service Areas/Block	2016	2020	2025	2030	2035	2040
B-10						
Domestic WW	574	593	640	689	742	798
Commercial WW	151	193	263	362	502	702
Sub-Total (m3/day)	**725**	**786**	**903**	**1,051**	**1,244**	**1,500**
B-11						
Domestic WW	324	335	362	390	420	451
Commercial WW	40	51	71	97	135	189
Sub-Total (m3/day)	**364**	**386**	**433**	**487**	**555**	**640**
B-12						
Domestic WW	1,718	1,775	1,915	2,064	2,221	2,389
Commercial WW	466	584	785	1,068	1,467	2,034
Sub-Total (m3/day)	**2,184**	**2,359**	**2,700**	**3,132**	**3,688**	**4,423**
B-13						
Domestic WW	1,608	1,812	2,178	2,616	3,138	3,761
Commercial WW	82	97	121	155	200	262
Sub-Total (m3/day)	**1,690**	**1,909**	**2,299**	**2,771**	**3,338**	**4,023**
Total (m3/day)	4,963	5,440	6,335	7,441	8,825	10,586
Infiltration Rate-10% (m3/day)	496	544	634	744	882	1059
Grand Total (m3/day)	**5,459**	**5,984**	**6,968**	**8,185**	**9,707**	**11,645**

The wastewater treatment plant for a separated system, for Phase 1 to year 2040 therefore needs to have capacity 11,645 m^3/day.

5 WW Treatment Plant Options

5.1 Introduction

Currently there are two areas of land available for use, the 5.8ha plot of land currently housing the existing 1994 SAWA WWTP, and 6.6ha to the north west of this area available for development. The MPWT have also requested that options should include the possibility of a total of 10ha or more of new land be available, to be purchased with Government budget.

There are several technologically appropriate options to consider in identifying the most appropriate solution for treating Battambang's wastewater to year 2040. The main options presented revolve around different configurations of aerated lagoon systems, and trickling filters preceded by settlement. Technologies that are widely seen as inappropriate for countries at the beginning of their wastewater treatment capacity growth have not been considered, and case studies of technologies that have succeeded or failed across the region[1], have been reviewed in identifying these technologies. Lagoon based systems or Wastewater Stabilisation Ponds (WSP's) are widely acknowledged as the most appropriate technology for countries with a limited financial or trained operational staff resource – provided that sufficient land is available for use. If the large amounts of land required for WSP's is not available, then trickling filters preceded by a sedimentation pond are a good alternative which require far less land than WSP's, although they do require a greater level of operational skill and cost more to run.

A **separated system** is proposed, as per the technology choice guidelines of the Urban Sanitation Policy in the "National Guidelines on Water Supply and Sanitation (RGC, 2003)" [2]

This enables all wastewater to be treated, retains a higher BOD enabling the WWTP to operate more efficiently, and provides for cleaner stormwater to enter the river during the wet season (not being mixed with overflowed wastewater). Whist all roads in the Phase 1 area will require excavating for pipelaying for a separated system, pipe diameters are small (100-150mm) and disruption minimal, whereas if a combined system was recommended, current drain (800mm-1500mm) upgrades would require far bigger excavations and disturbance to residents and traffic.

As agreed with MPWT, options described below are to serve the Phase 1 area of the city only. The boundary of Phase 1 is shown in Figure 5, and has flows summarised in Table 25 below.

[1] Vietnam Urban Wastewater Review 2013, World Bank & AusAID. Da Lat Trickling filter WWTP Danida 2008 Quy Ngon Trickling filter WWTP

[2] Item 4 states "The use of separate sewerage and drainage systems should be promoted and encouraged particularly in new installation areas."

Table 25 - Summary of Phase 1 WW flows

WW Flow (m3/d)	2016	2020	2030	2040
Domestic	4,224	4,515	5,759	7,399
Commercial	739	925	1,682	3,187
Infiltration	496	544	744	1,059
Total	5,459	5,984	8,185	11,645

5.2 Aerated lagoon options

With the ambient temperatures experienced in Cambodia, aeration by mixing the upper layers of the lagoons has the potential to increase the capacity of the WWTP by 2-2.5 times. This represents the most technically simple option for ease of operation and maintenance from a staff base of limited capacity and experience.

There are three main options for use of aerated lagoons to treat the projected 2040 Phase 1 wastewater flows, differing in whether and how the existing current 5.8ha WWTP site is used in conjunction with the proposed new 6.6ha WWTP site, and furthermore whether a total 10ha site will become available for development during the course of this project preparation.
Appropriate mixers should be of the floating surface variety as opposed to submersible so that the depth of mixing can be better controlled in the lagoons. In the facultative lagoons the objective would only to be to aerate the upper layers whilst leaving the deeper anaerobic layers undisturbed. Whilst many brands of mixer are widely available in China, Thailand and Vietnam, they are generally simple and durable, requiring little maintenance, and so source-country is not as important as more complex or high maintenance equipment.

Solar powered mixers have been requested by the DPWT. These have the obvious advantage of reducing power costs, taking advantage of the long hours of sunlight in Battambang. A brand of solar mixers sourced from the US has been investigated and they provide several models, including a very shallow surface water mixer designed for odour reduction in the anaerobic lagoons, which provides the added advantage of removing surface scum and aiding methane release improving improves digestion of wastewater. These mixers are specifically designed to stop the escape of aerosols. A second mixer type designed for the facultative and maturation lagoons mixes water to a deeper level and this depth can be controlled and set. See Appendix E for further information on these solar mixers.

The mixers are proposed to also be connected to mains power for those times of year when it may be overcast, or in case of any of the solar panels being out of service.

5.2.1 AL1: Aerated lagoons for existing and new sites

SUMMARY AL1: This option proposes to rehabilitate and utilize the existing 5.8ha site, plus develop the proposed new 6.6ha site into an aerated lagoon system, such that both operate **in parallel**. The existing site, with aeration, desludging of anaerobic lagoons and new delivery pumps, can have capacity increased from the current 450m^3/day to 2,000-2,500 m^3/day.

The new 6.6ha of land can be developed into a lagoon system capable of treating at least 6,500-8,125 m^3/day. Combined, and in series, these two plants could treat the Phase 1 flows projected for year 2037-38.

With the ambient temperatures experienced in Cambodia, aeration by mixing the upper layers of the lagoons has the potential to increase the capacity of the WWTP by 2-2.5 times, or up to 8,500 – 10,625m^3/day for both sites. If a 2.5 factor increase in capacity is achieved by mixing, then the capacity falls just 2 years short of the 2040 target, based on all assumptions involved in the projected flow calculations.

The advantages of retaining the existing site is that the land is already owned, has a sparse population around it, and would require a minimal investment to rehabilitate it and to get it operating with a capacity over 4 times greater than at present. On its own, it should be able to treat around 60% of the 2016 wastewater flow from the defined Phase 1 area. Being in a separate physical location to the proposed new 6.6ha WWTP is not a significant disadvantage – the distance between them is short (around 1.5km) and a full time operator is not required for both plants.
The manufacturer has proposed the following configuration:

Table 26 - Mixer configuration – existing SAWA WWTP

Pond type	Pond Qty	Mixer type	Mixer Qty (per pond)	Mixer Qty (total)
Anaerobic	2	Odour capping	1	2
Facultative	2	Aeration	2	4
Maturation	0	Aeration	0	0

Table 27 - Mixer configuration – proposed new 6.6ha WWTP

Pond type	Pond Qty	Mixer type	Mixer Qty (per pond)	Mixer Qty (total)
Anaerobic	2	Odour capping	1	2
Facultative	2	Aeration	2	4
Maturation	2	Aeration	1	2

Mixer data sheets are contained in Appendix C.

5.2.2 AL2: Aerated lagoons for existing and new sites with redevelopment of existing "SAWA" site

SUMMARY AL2: This option proposes to completely redevelop the existing 5.8ha SAWA site, plus develop the proposed new 6.6ha site into an aerated lagoon system, such that both operate as one WWTP with lagoons **in series**. The existing site will be redeveloped into larger anaerobic lagoons with septage disposal bays, sludge drying facilities and a downstream pump station.
The new 6.6ha of land can then be developed into the second part of the lagoon system, consisting of aerated facultative and maturation lagoons. The combined area would be capable of treating approximately 10,000 m^3/day. Combined, and in series, these two plants could treat the Phase 1 flows projected up to around year 2037.

This option proposed to bulldoze the existing WWTP site and redevelop it as the first two (anaerobic) lagoons in a proposed larger split-site WWTP. The proposed two anaerobic ponds could be constructed on the east side of the road, and a sludge drying facility on the land reclaimed from the existing "Pond 2" on the west side of the road. Shallow-draw "odour capping" mixers are recommended for these two anaerobic lagoons both to reduce odour from the upper layers, and break the scum layer which withholds methane and reduces treatment efficiency. These mixers are specifically designed to stop the escape of aerosols (data sheets in Appendix A).

The new 6.6ha site would be developed as the downstream part of the split-site WWTP, housing the bunded facultative lagoons and maturation lagoons, with aeration, site roads and an operators building/store. The mixer configuration is suggested as per Table 27 above.

5.2.3 AL3: Aerated lagoons for an extended (15ha) new site only

SUMMARY AL3: This option is dependent on the DPWT acquiring more land at the current new 6.6ha site. This option proposes to decommission the existing 5.8ha site, retaining it for future public works use. This proposed new 15ha site would be developed into a single-site aerated lagoon system.

This total 15ha of land can be developed into a lagoon system capable of treating at least 12,000 m^3/day, sufficient capacity to serve the Phase 1 area beyond 2040.

5.3 Trickling Filter Options

Trickling filters are designed primarily for BOD removal. They are basically a form of biological filter as opposed to a physical filter in that solids are not removed. Biofilm forms on media inside the filter, which convert pollutants into harmless compounds in the presence of air. Wastewater is sprinkled onto the surface of the media from a rotating arm after which it passes vertically through the filter media. Air enters from both underneath and the top and creates a mostly aerobic environment, except for the centre of the filter which can become anaerobic.

An underdrain system collects treated wastewater, which then usually requires further treatment and settlement in an oxidation pond prior to release into receiving waters. Wastewater needs to be settled in advance of feeding it into the trickling filter, which itself has a very low TSS removal. Filters will clog if solids are not removed upstream of the inlet. Trickling filters are a secondary treatment. The hydraulic and *nutrient* loading rate (i.e., capacity) is determined based on the characteristics of the *wastewater,* the type of filter media, the ambient *temperature*, and the discharge requirements.

The filter is usually above ground, and is usually a cylinder 1-3m deep filled with large surface area media, to which micro-organisms attach. In the past, stones were usually used for media. In more recent times high surface area plastic media have been developed which are much lighter than the traditional stone media and allow the filters to be higher and better ventilated, increasing capacity further. Using these specially designed plastic media can double the capacity when compared to using stone media in the same sized trickling filter.

Sloughing of the micro-organisms from the media into the effluent can occur which requires further settlement downstream of the filter. Sloughing can be managed by a skilled operator finding a balance between bio-film growth and amount of bio-film falling off the media into the effluent, by controlling the organic and hydraulic loads into the filter.

Pumps are required to feed the wastewater to the top of the filter unit, so a constant power supply is required. Whilst trickling filters require more power than a lagoon based system, they require significantly less power than recirculated sludge based plants and can be considered a relatively low-energy green technology. A standard low-rate trickling filter can remove 80-85% of BOD when operated correctly.

Table 28 - Advantages and disadvantages of trickling filters

Advantages	Disadvantages
Can be operated at a range of loading rates	High capital costs
Resistant to shock loadings	Requires expert design and construction
Efficient nitrification	Requires full time O&M staff for efficient operation
High BOD removal	Requires a constant flow of wastewater and source of power
Small land required compared to lagoon systems	Flies and odours are often a problem
	Pre-treatment and sludge treatment are required
	Risk of clogging
	Not all parts & materials available locally

Modern more complex "**high rate**" trickling filters can have greatly increased capacities compared to the standard configurations that have been used for many decades.
A constant flow is desired, and this can be achieved during times of varying wastewater inflow by recirculation of effluent to the sedimentation pond upstream of the filter or the filter itself. This also increases dissolved oxygen levels, ensure wetting of the media and helps control sloughing. Trickling filters with one or two stages of recirculation are sometimes known as "high rate" trickling filters. The dosing rate onto the media is also important and can be controlled by the sprinkler arm rotation speed and amount of dilution by recirculation. A good operator can maximise treatment capacity by monitoring incoming wastewater BOD and adjusting the trickling filter operation appropriately. A high rate trickling filter can typically remove 60-80% of BOD.

The advantages of recirculation are:

- It allows constant dosage regardless of fluctuation in sewage flow and thus keeps the bed working.
- It dilutes the influent with better quality water and, making it fresh and reducing odour.
- It maintains a uniform rate of organic and hydraulic loading.
- It provides longer contact of the applied sewage with the bacterial film on the contact media and accelerates the biological oxidation process.
- It increases the efficiency by reducing the BOD load generally.

A high rate trickling filter can have a capacity up to 10x that of a standard trickling filter and therefore require less land.
Figure 6 shows the basic features of a trickling filter.

Figure 6 - Trickling filter example

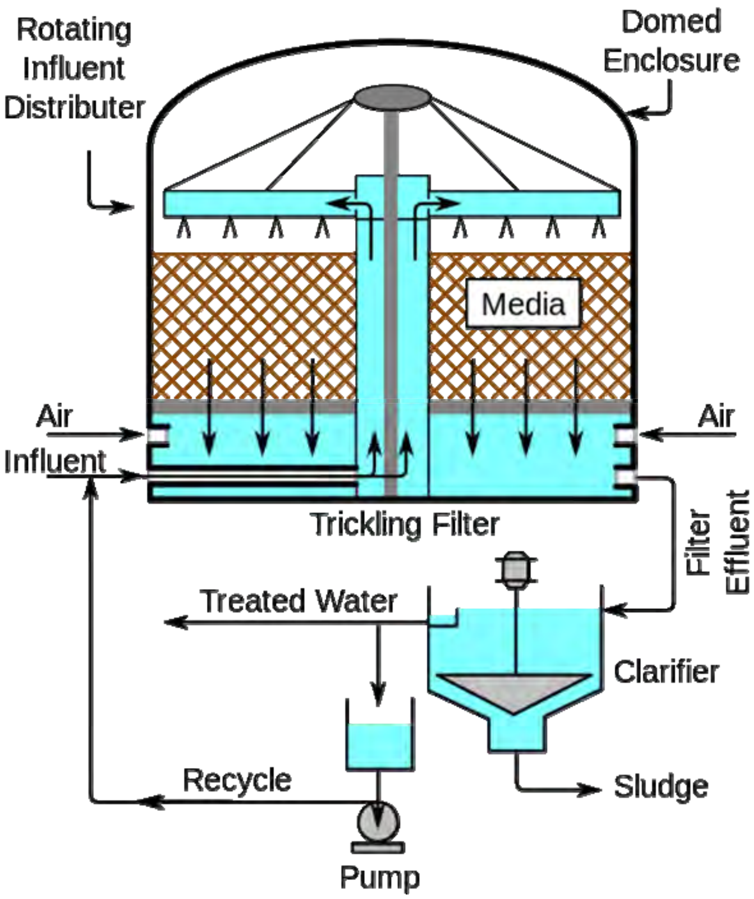


Table 29 below illustrates some expected capacities from both "Conventional" and "High rate" trickling filter types and configurations for a filter of 2m media depth. Literature[3] has a conventional trickling filter capable of treating 1-4m^3/m^3 media/day and a high rate filter with 10-40m^3/m^3 media/day. For the purpose of this example calculation a median flow of 2.5m^3/m^3/d and 25m^3/m^3/d respectively have been used. The 2040 design flow of 11,645m^3/d has been used to calculate diameter.

Table 29 - Trickling filter configurations and capacities to design year 2040

Filter configuration	Expected flow range (m3 flow/m3 media/day)	Flow rate used in calculation (m3 flow/m3 media/day)	2040 Flow/filter (m3/d)	Filter diameter (m)
1 x conventional TF	1-4	2.5	11,645	54
2 x conventional TF	1-4	2.5	5,822	38
3 x conventional TF	1-4	2.5	3,882	31
4 x conventional TF	1-4	2.5	2,911	27
1 x high rate TF	10-40	25	11,645	17
2 x high rate TF	10-40	25	5,822	12
3 x high rate TF	10-40	25	3,882	10

[3] *www.idc-online.com/technical_references/pdfs/civil.../**Trickling_Filter**.pdf*

Trickling filters are generally not larger than 60m diameter as the rotating arm distributing influent must be supported by cables and the larger the diameter, the more challenging this becomes.
The construction of the trickling filters may also be staged, with immediate construction allowing for wastewater flows to 2030. Options for this scenario are shown below in Table 30.

Table 30 - Trickling filter configurations and capacities to design year 2030

Filter configuration	Expected flow range (m3 flow/m3 media/day)	Flow rate used in calculation	2030 Flow/filter (m3/d)	Filter diameter (m)
1 x conventional TF	1-4	2.5	8,185	46
2 x conventional TF	1-4	2.5	4,092	32
3 x conventional TF	1-4	2.5	2,728	26
4 x conventional TF	1-4	2.5	2,046	23
1 x high rate TF	10-40	25	8,185	14
2 x high rate TF	10-40	25	4,092	10
3 x high rate TF	10-40	25	2,728	8

5.3.1TF1: Conventional trickling filters on new 6.6ha site with sedimentation ponds on existing "SAWA" Site

SUMMARY TF1: This option proposes redeveloping the existing "SAWA" WWTP site as upstream sedimentation ponds from which settled wastewater will be pumped to one or more trickling filters on the new 6.6ha site.

The existing "SAWA" site would be redeveloped with four settlement ponds in parallel, septage bays for co-treatment, and a sludge drying facility on the west side of the road. The four ponds sized for the 2040 Phase 1 flows (11,645m^3/d) would be 50m x 30m each. Four ponds have been selected over two to allow better operational flexibility prior to the full 2040 flow being met, in particular easier desludging. The outlet from the ponds would be pumped to the new 6.6ha WWTP site.
The nearby new 6.6ha site would be developed with standard trickling filters. From Table 29 above, four standard trickling filters of 27m diameter could be employed to treat 2040 flows, which would require approximately 100m x 100m or 1ha of land. Alternatively, filters to treat up to 2030 flows only could be installed initially, with further filters to be added prior to 2030 to handle additional flows expected between 2030 -2040. There would be sufficient space on the allotted land for future expansion to serve Phase 2 of the proposal.

5.3.2 TF2: Conventional trickling filters with sedimentation ponds all on new 6.6ha site

TF2: SUMMARY: This option proposes the same as the previous Option but with both sedimentation ponds and trickling filters on the new 6.6ha site. The existing “SAWA” WWTP site would be decommissioned.

Due to there being sufficient land for both sedimentation ponds, trickling filters and sludge drying beds on the 6.6ha of available land, this option is preferred over Option 5.2.1. All separated wastewater from the Phase 1 area of the city would be pumped to the WWTP site where it would enter the 4 settlement (anaerobic) ponds of 50m x 30m each. From here wastewater would be further pumped to each of 4 conventional trickling filters of diameter 27m each (from Table 29).
As for the previous option, trickling filters to treat up to 2030 flows only could be installed initially, with further filters to be added later to handle additional flows expected between 2030-2040.

5.3.3 TF3: High rate trickling filters with sedimentation ponds all on new 6.6ha site

SUMMARY TF3: This option proposes the same as Option 5.3.2 but with the use of high rate trickling filters on the new 6.6ha site. The existing “SAWA” WWTP site would be decommissioned.

From Table 30 it can be seen that one option could be to construct one high rate trickling filter with recirculation and high surface area plastic media of 14m diameter (area approx. 400m^2) on part of the 6.6ha. The upstream settlement lagoons would require a further 9,600m^2. This could provide secondary treatment to design year 2030, after which a second trickling filter could be constructed, should the technology prove itself to be appropriate for Battambang staff capacity levels. Should the more advanced high rate trickling filters be selected, periodical ongoing external operational support to DPWT staff is recommended over several years.
However, as there is sufficient land immediately available to utilize simpler, lower energy conventional trickling filters, there are no driving factors to support the use of high rate filters at this stage.

5.4 Further expansion to serve Phase 2 areas

Consideration has been given to the long term wastewater treatment coverage requirements for the entire urban area. It can be seen from the above (Options under 5.2) that Phase 1 year 2040 wastewater flows of 11,645m^3/d can be accommodated in a lagoon-based WWTP if 15ha becomes available, or by conventional trickling filters on the currently available 6.6ha. However, if both Phase 1 and 2 are to be served with a year 2040 flow of 20,694m^3/d, then the lagoon option will not be viable with 15ha. A total of around 25ha land will be required, to also include site roads and services.

For both Phase 1 and Phase 2 areas to be served on the land currently available then one of the trickling filter options described under Section 5.3 will be required.

5.5 Consideration of other technologies

Whilst not deemed appropriate for Battambang, several other technologies have been used in the region, mostly Vietnam, and are briefly described here.

5.5.1 Oxidation ditch technology (Activated sludge)

An oxidation ditch is used for the low strength WW usually associated with municipal use. The wastewater is circulated around an oval "ditch" by horizontal aerators, and activated sludge is reintroduced into the process. Oxidation ditches require more land than other activated sludge processes, and more land than trickling filters but do not require settlement lagoons upstream as trickling filters do. They also have several stages of sludge pumps, blowers and other moving machinery which require maintenance and correct operation.

An oxidation ditch WWTP in Bac Giang, Vietnam, of capacity 10,000m^3/day utilises a site of 145m x 75m or 1.1ha. The proposed Battambang WWTP site is 300m x 300m or 6.6ha so could easily accommodate two or more trains of oxidation ditch plants to accommodate future flows for the whole urban area. However, there has been little success in Vietnam with this Activated sludge technology due to operational skill and budget requirements and it is not recommended as a first WWTP for Battambang or any Cambodian city.

5.5.2 UASB technology

The Upflow Anaerobic Sludge Blanket reactor is a single stage process where the wastewater enters the plant from below and flows upwards through a (normally) cylindrical body, passing through a suspended sludge blanket consisting of granules and formed gas bubbles, which acts as both a mechanical filter and provides biological treatment. Biogas is produced, which can either be collected or released. The sludge blanket takes several months to form following start-up. Figure 7 below shows the basic components of a UASB reactor.

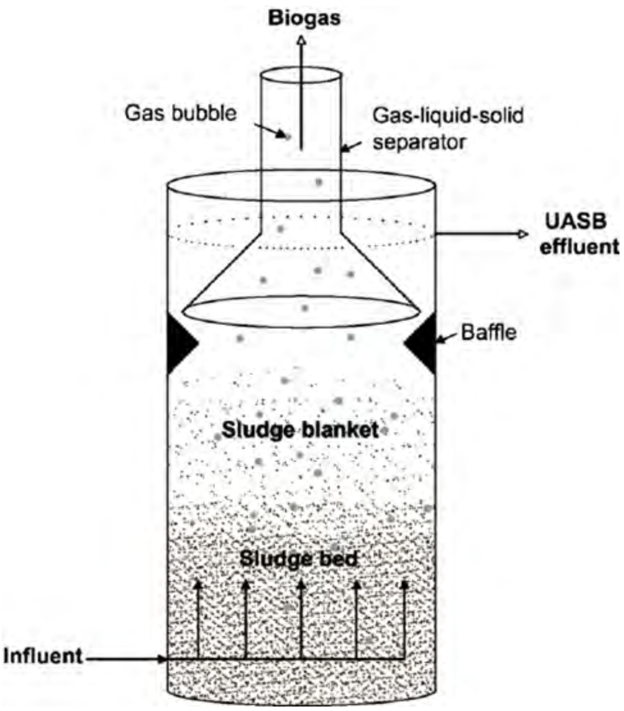
The effluent is nutrient-rich and should ideally be disposed of to agricultural lands rather than natural watercourses.

The UASB plant requires a high level of professional operator skill and understanding. Areas of operation particular to UASB plants are:

- They are more successful with high COD content influent – a minimum of 250ml/l and over 400mg/l for optimum performance.
- The influent must be consistently distributed across the base of the structure to enable good formation of the blanket and even contact with it. This requires a good influent distribution system that must be monitored.
- The hydraulic load must be set to match the organic load. The upflow velocity must be set to balance the floating sludge blanket to keep it suspended without washing suspended solids out the top of the filter. The control of organic loads is by the operator using a feed pump to match the hydraulic loads to the fluctuating strength of the wastewater coming into the plant.
- A continuous and stable WW flow is required. The UASB has little capacity to handle shock loads.
- The process has a low pathogen reduction

Whilst the UASB technology requires less land than many options, the level of operator skill required, the high influent strength required for efficient anaerobic treatment, and the high levels of nutrients and pathogens in the effluent makes it unsuitable for this application.

Figure 7 - UASB reactor basic layout



5.6 Use of higher technology, energy saving equipment and processes.

There is a Japanese grant fund available (JFCJM) to offset any additional costs associated with purchasing higher technology or more advanced equipment that will have an energy saving or carbon footprint-reducing benefit over a more standard or basic technology selection.

Where possible, preliminary designs selected and recommended under this subproject should take advantage of this potential funding source.

For the above WWTP option of aerated lagoons, the most basic option to increase treatment capacity through mixing/aeration would be banks of submersible or beam-mounted mixers powered from the mains. These are widely available in SE Asia for a relatively low cost, and many are manufactured in China. Utilising higher technology, standalone solar powered floating mixers can be used, which whilst more expensive have no mains energy requirements, and are tailored to provide far more directional mixing specific to the requirements.

For the option of using conventional trickling filters, there are opportunities to increase efficiency and treatment capacity with use of higher technology. Proprietary high surface area media, media support and underdrains can all improve treatment efficiency.

Under all of the described WWTP technology options, there is a requirement for 3 pump stations to deliver wastewater to the WTWP from the city. Two key improvements can be made in energy saving at these pump stations.

Firstly, the correct selection of a high quality submersible wastewater pump taking into account the true duty point range from superimposing the duty and operating curves. Whilst a standard method of specifying pumps for many decades, it is often not done, with some engineers and contractors instead relying on a manufacturer's duty data only. Purchasing a more expensive but well known trusted international brand also has long term advantages in power efficiency over cheaper regional brands.

Secondly, the use of variable speed drives (also called variable frequency drives, adjustable speed drive or frequency inverters) can adjust the speed of the pump to suit a variable flow or head requirement, saving power over the more common on/off configuration associated with a fixed speed pump.

6 Selected options for WWPT

Two options have been selected for recommendation, dependent on final land availability, as follows.

6.1 Selected lagoon option

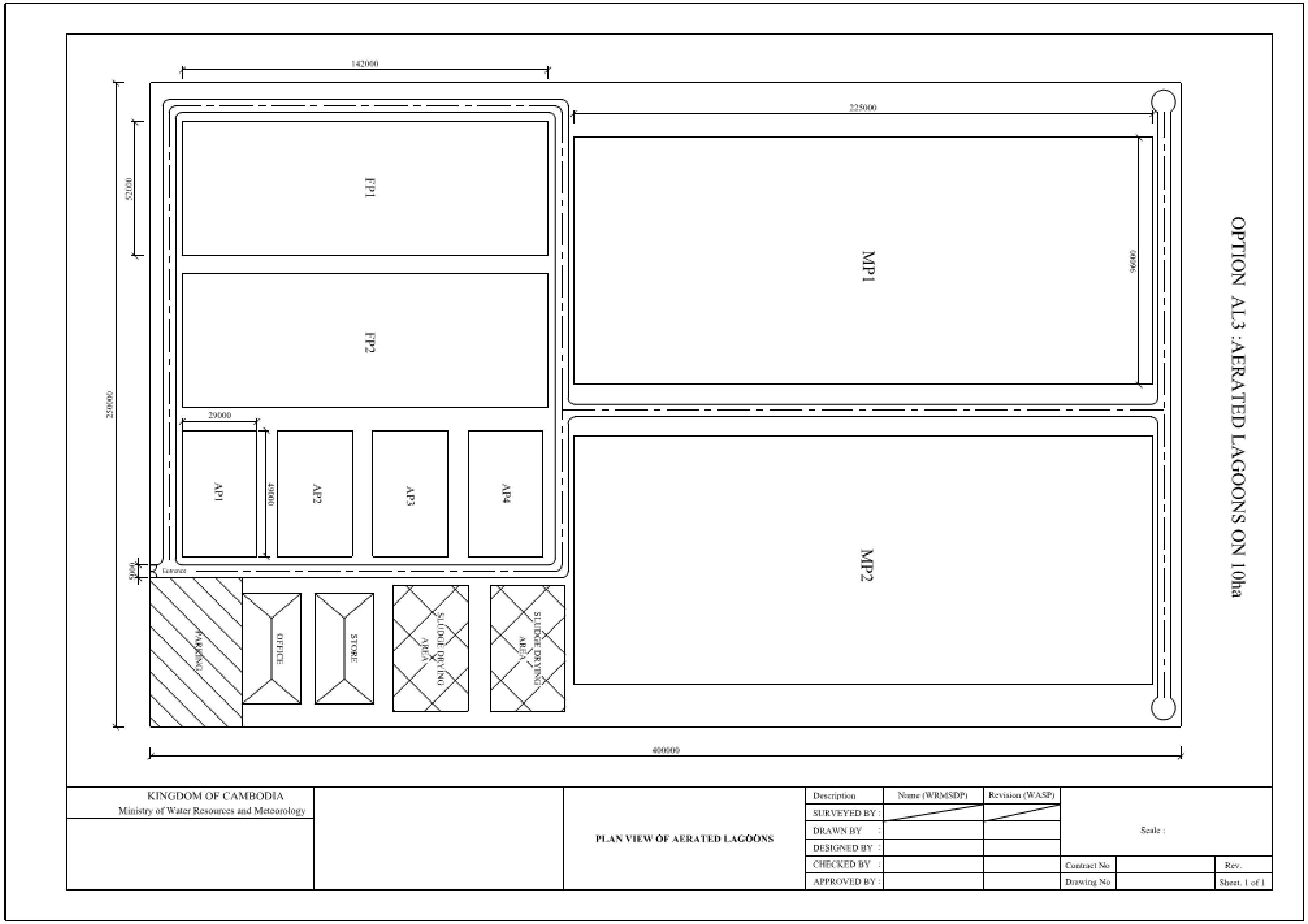
Option **AL3** described in Section 5.2.3 is the preferred option – use of aerated lagoons on a single site comprised of the current 6.6ha extended to a minimum of 15ha by purchase of more land with government budget, as desired by MPWT. The current "SAWA" WWTP on 5.8ha would be decommissioned. With a 15ha site all separated wastewater to year 2040 could be treated using aerated lagoons. The WWTP would consist of two separable trains of lagoons, each train being two anaerobic lagoons, one facultative lagoon and one maturation lagoon. The extension to land already acquired would need to be of an appropriate shape to accommodate these lagoon trains. The anaerobic lagoons would have solar aerators specifically designed for these lagoons installed, which disturb only the upper 20-30cm of water and reduce odours and break the scum layer without producing aerosols (see Appendix C for manufacturers data)
The aerated lagoon sizes required are shown in Table 31.

Table 31 - Sizes of selected aerated lagoons

Process	No. ponds	Length	Width	Depth	Area (ha)
Anaerobic	4	40	25	4	0.193
Facultative	2	157	57	1.75	1.499
Maturation	2	348	120	1.5	8.352
					10.044

The remaining 5.6ha of the required 15ha site will be taken up with site roads, sludge drying beds, office, store, car parking and bunds.
The effluent discharge standard for coliforms when discharged to a river is under 5000MPN/100ml, and for discharge to lakes and reservoirs under 1000MPN/100ml. The preliminary design has used the higher standard for rivers and lakes, as discharge will not be directly to the river but through a series of public irrigation waterways. An example layout is provided in figure 8. Calculations for the required lagoon dimensions to meet these effluent standards are contained in Appendix B.

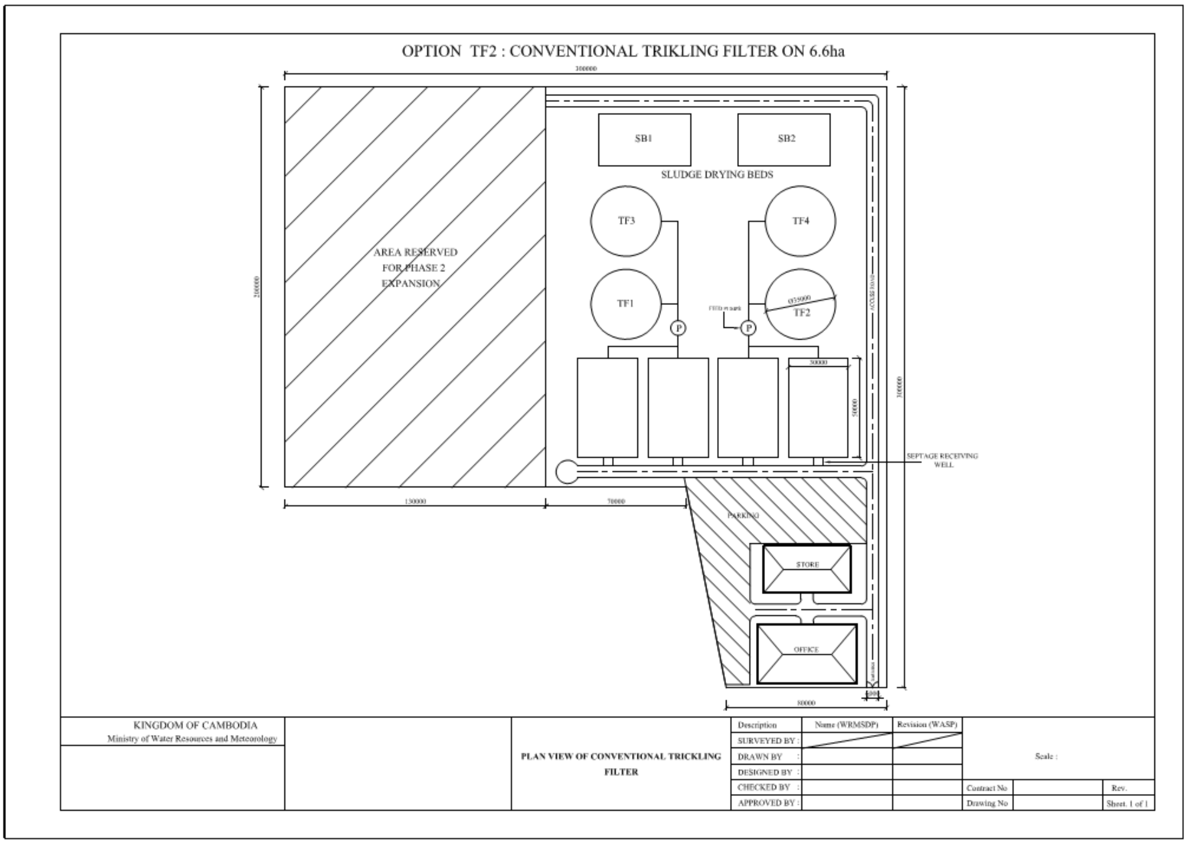
Figure 8 - Potential lagoon layout



6.2 Selected trickling filter option

Should an additional 8.4ha immediately adjacent to the ready-acquired 6.6ha not become available in the near future, then the recommendation is for option **TF2** described in section 5.3.2. This is to utilise sedimentation ponds and conventional trickling filters on the same 6.6ha site. A potential layout is shown in Figure 9.

Figure 9 - Potential trickling filter layout



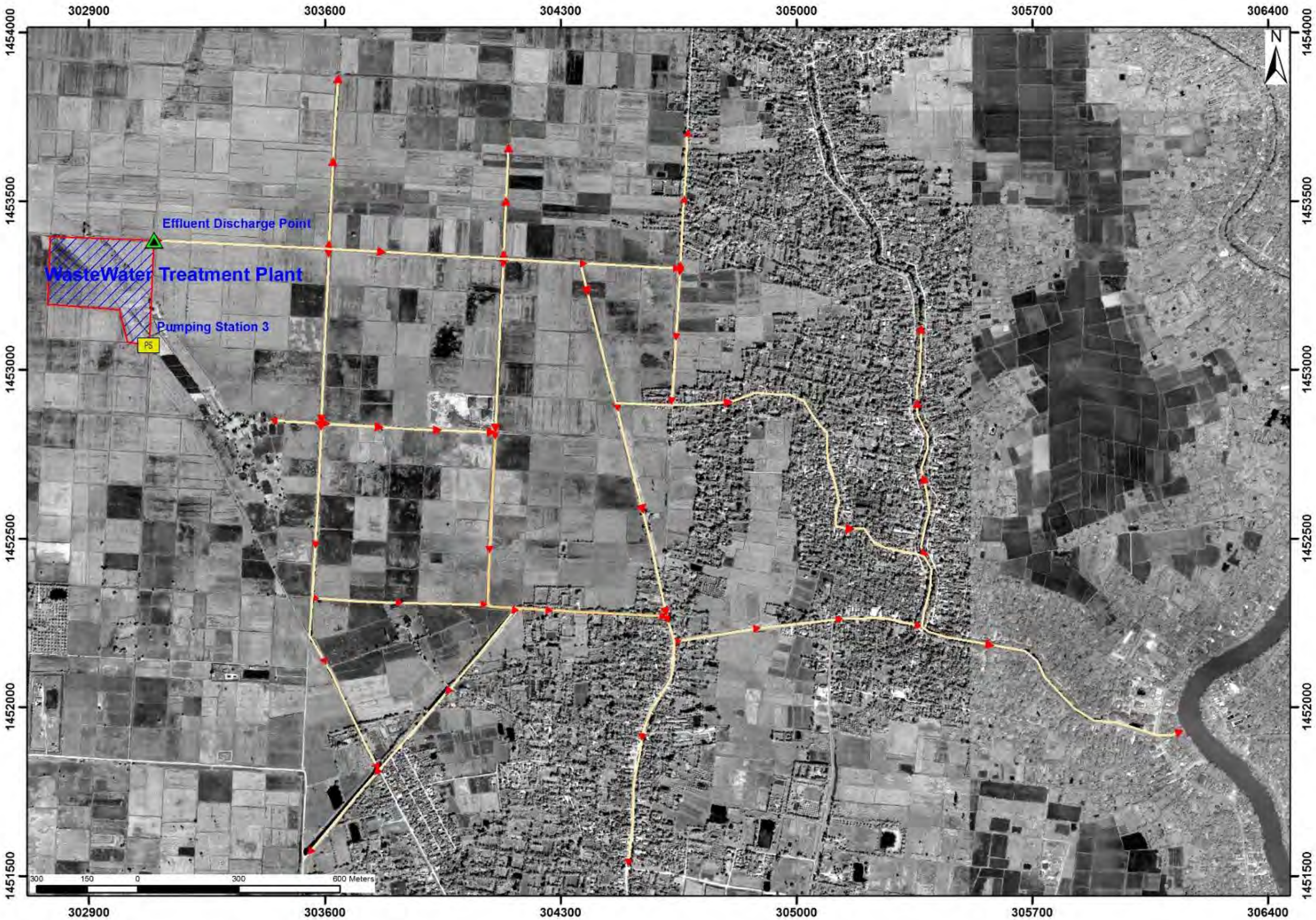
6.3 Effluent Disposal

Effluent from the proposed WWTP will be released into an existing irrigation canal, which forms one part of a large network of canals surrounding Battambang city. The canals drain to the north east, where they enter the river approximately 2.5km downstream of the city centre near Phum Kdol.

The treated effluent will be to national standards and will be suitable for both irrigation and river discharge.

The figure below shows the effluent discharge path.

Figure 10 - Effluent discharge path to river



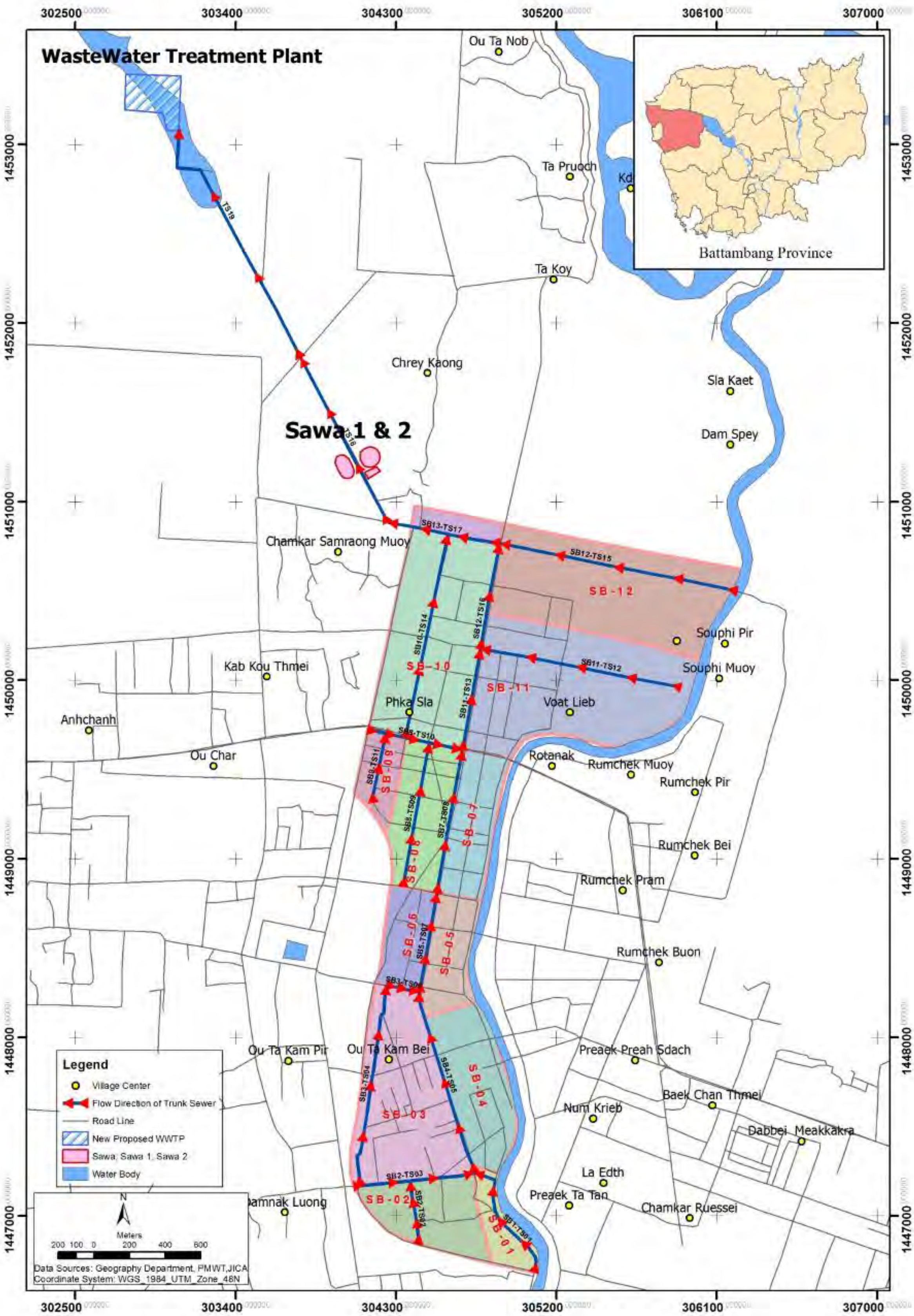
7 Trunk Sewers

7.1 Existing trunk sewer locations and flow direction

Battambang town is a relatively flat zone. At the town boundary, the slope is from the line of the riverbank westwards to the town center and to the north of the town. In the central Phase I service area, the elevation along the river bank is higher than that of town center. The main waterway flows from south to north through the center of the town.

There are existing trunk sewers which have been installed and are in generally good condition in the existing downtown coverage service area. These trunk sewers are proposed to be retained for stormwater only, with wastewater separated. The design flow direction of the existing trunk sewers is shown in Figure below.

Figure 11 - Main trunk sewer flow direction



7.2 Proposed separated trunk sewers

Based on the identified flow direction, the main trunk sewer size has been estimated using the Manning formula. In this project, uPVC or GRP pipe are proposed depending on the size of the trunk. For sewers with diameter less than 400mm uPVC is proposed, and GRP for diameters greater than 400mm.

Manning formula is used in this calculation with the assumption that the depth of water flow inside the pipe is 90% of the pipe diameter and the slope is uniform with 0.15%. The manning formula is expressed as below:

$$Q = \frac{S}{n} R^{2/3} i^{1/2}$$

where:

Q	: Flow (m3/s)
S	: Sectorial area of flow (m2)
n	: friction coefficient (-)
R	: Hydraulic radius (m)
i	: slope (m/m)

The total discharge in 2040 is used for the pipe size calculation. The trunk sewer diameter was found to range from 100mm to 450mm with a total length of 15,387m. The detail of the pipe size, flow velocity, and length of the trunk sewer for each design block is shown in Table 32 and the total length of pipe with diameter is shown in Table 33.

Table 32 - Proposed trunk sewer data for each block

Service Areas/Trunk	Flow (m3/day)	Length (m)	Slope (%)	Velocity (m/s)	Diameter (mm)
SB1-TS01	201	738	0.3	0.527	100
SB2-TS02	550	358	0.3	0.611	125
SB4-TS05	1254	1061	0.25	0.763	200
SB3-TS06	1385	190	0.2	0.682	200
SB2-TS03	550	682	0.3	0.611	125
SB3-TS04	1361	1150	0.2	0.682	200
SB5-TS07	4269	563	0.15	0.774	300
SB7-TS08	5356	812	0.15	0.858	350
SB8-TS09	1346	820	0.2	0.682	200
SB8-TS10	1863	545	0.2	0.792	250
SB9-TS11	518	395	0.3	0.611	125
SB11-TS12	1591	1147	0.2	0.682	200
SB11-TS13	7219	575	0.15	0.938	400
SB10-TS14	1286	1165	0.25	0.763	200
SB12-TS15	1423	1359	0.25	0.763	200
SB12-TS16	8800	600	0.15	0.938	400
SB13-TS17	11645	640	0.15	1.015	450
TS18	11645	1046	0.15	1.015	450
TS19	11645	1541	0.15	1.015	450

Table 33 - Total length of trunk pipe with different diameter

No.	Trunk Diameter	Length (m)
1	uPVC 100mm	738
2	uPVC 125mm	1435
3	uPVC 200mm	6,892
4	uPVC 250mm	545
5	uPVC 300mm	563
6	uPVC 350mm	812
7	uPVC 400mm	1,175
8	GRP 450mm	3,227
Total		15,387

7.3 Pipe material selection

Both uPVC and GRP pipe types have good resistance against the corrosive nature of wastewater, and good hydraulic flow characteristics. A summary of advantages and disadvantages of both pipe types is provided in Table 34.

Table 34 - Pipe material comparison for use as sewer pipe

Material	Advantage	Disadvantage
uPVC	Superior all round performance	Easily cracked (impact damage)
	Lightweight	UV degradation for lengths above ground
	Resistant to chemical attack	Difficult to locate underground
	Flexible – resistant to fracture & adaptable to earth movements	Leak location difficult
	Watertight gasket joints	Not suitable for large diameters
	Smooth wall surfaces – better flow characteristics and less slime build-up	
	Predominant material for sewer pipes	
	Relatively low installed cost	
	Sufficient longitudinal rigidity to allow easy laying to grade without ponding	
	Long service life expectation (>100 years with no deterioration in bulk properties)	
	Good structural performance of buried sewer pipes over time	
GRP	Long service life >50 years	Low mechanical strength
	High corrosion resistance	Difficult to locate underground
	Low weight	Leak location difficult
	Simple push-on couplings	
	High stiffness available	
	High abrasion resistance	
	High tensile strength	
	Resistant to scaling	
	Flexible	
	Available in longer lengths (less joints)	
	Better hydraulic performance than DI & RC	
	Large diameters available	

The main reason for selecting GRP for larger diameters in addition to the reason given above, is that rigid uPVC is not generally available for larger diameters (most suppliers provide up to 24” or 600mm, and above 300mm is not common) and at these sizes it is a special order and expensive (eg. $360/m for schedule 40 and $540/m for schedule 80). GRP pipe can be supplied fit for purpose to much larger diameters.

7.4 Collection pipelines

The main trunk sewers are described above. In addition, pipelines along every road in the Phase 1 area will be required, both sides of the road, in order to make domestic connections.

7.5 Household Connections

It is proposed to install free domestic and commercial connections to all existing properties in the proposed new Phase 1 service area (8,400 in 2020 from Table 18). Commercial connections are additional to these figures and are shown in Tables 19-23.

All connections should be made free under the project, to enable the WWTP to get up to operating flows, to enable DPWT to start collecting revenue as soon as possible, and to carry out disturbance due to construction all at one time as opposed to excavations for connections spread out ad-hoc over the following years. The ADB project review of the original project recommended that all wastewater projects include free connections as the primary lesson learned.

8 Pump Stations

8.1 Pump Stations

For the Phase 1 service area, two pump stations in town plus one at the WWTP are required. The topography is flat, but the distance of trunk sewer to WTP is up to 7.5km.

As shown on Figure 12, pump stations 1 and 2 will lift wastewater from a collector sump back into a shallower gravity pipeline so that wastewater can continue on to the WWTP. There will also be auxiliary pumps associated with distribution of wastewater around the trickling filter site, particularly recirculation if utilised.

Table 35 - Pump station data

Criteria/Block	Condition	PS1	PS2	PS3
Elevation (m)	Ground level	+12.09	+8.97	+7.45
	Inlet (invert)	+6.09	+3.17	+2.55
	Outlet (invert)	+10.59	+7.47	+5.95
	Highest point	+12.09	+8.97	+7.45
Flow (m3/day)	Year 2030	2,203	5,712	5,712
	Year 2040	3,265	8,505	8,505
Pumping main diameter (mm)	Year 2030	150	250	250
	Year 2040	200	300	300
Pumping main distance (m)	-	20	20	20

8.2 Variable speed drives

Most pump applications do not require full pump performance 24 hours a day. Therefore, it is an advantage to be able to adjust the pump's performance in the system automatically. The best possible way of adapting the performance of a centrifugal pump is by means of speed control of the pumps. Speed control of pumps is normally made by a frequency converter unit.

Variable Speed drives could be utilized in the wastewater lifting station especially with pump motor rating above 5.5kW. These pumps with big pump motor rating as mentioned above does not require full pump performance 24-hours a day, thus when pumping water level or when the water level at the sump pit doesn't change abruptly or remained constant at a longer period, a level sensor which gives information and signal to the motor control with variable speed control should operate at a reduced speed or frequency thus resulting a lesser current consumption and contributes to a lower power consumption.

Speed controlled pumps with frequency inverter provides and gives the benefits of a **Reduced Energy Consumption,** Speed controlled pumps only use the amount of energy needed to solve and operates only on a specific pump job. Compared to other control methods, frequency controlled speed control offers the highest efficiency and thus the most efficient utilization of the energy. It gives a **Low Life Cycle Cost,** The energy consumption of a pump is a very important factor considering a pump's life cycle costs. Therefore, it is important to keep the operating costs of a pumping system at the lowest possible level. The efficient operation leads to a lower energy consumption and thus to lower operating costs. Compared to fixed speed pumps it is possible to reduce the energy consumption by up to 50% with a speed controlled pump. Speed controlled pumps **Protects the environment.** Energy efficient pumps pollutes less and thus do not harm the environment. Pumps with variable speed controller is easy and more comfortable to use. Speed control in different pumping systems provides increased comfort: In water supply systems, automatic pressure control and soft start of the pumps reduce water hammer and noise generated by too high pressure in the system, especially to pressure pipe system. Speed controlled pumps ensure that the different pressure is kept at a level so that the noise in the system is minimized. **Reduced system costs,** Speed-controlled pumps can reduce the need for commissioning and controlling of valves in the system. Therefore, the total system costs can be reduced.

8.3 Pump selection – submersible

The submersible pump with auto-coupling is a kind of pump which is easy to install and pulled-out if required during its maintenance. The auto-coupling system submersible pumps enable automatic connection or disconnection from its connection with the discharge elbow under the water to the outside of the sump pit. When the pump is installed on an auto-coupling system where the base is fixed to the bottom of the pump pit, the pump is lowered into the pit on a dual guide rail system. The pump automatically connects to the base unit in a tilted position in order to evacuate possible air in the pump housing and to prevent clogging or jamming. Submerged installation on hookup auto-coupling When the pump is installed on a hookup auto-coupling system, the base is fixed on a crossbar above the liquid level in the pit. The pump is lowered into the pit with the discharge pipe and the counter part of the coupling. The pump will be fixed in a tilted position when it is connected to the base. With both auto-coupling systems, the weight of the pump in combination with the sealing system will prevent leakage when the pump is operating.

The major advantage of a submersible pump is that, it never needs to be primed, because it is already submerged in the fluid. Submersible pumps are also very efficient because they don't really have to spend a lot of energy moving water into the pump. Water pressure pushes the water into a submersible pump, thus "saving" a lot of the pump's energy. Furthermore, submersible pumps can easily handle solids, while some are better for liquids only. Submersible pumps are quiet, because they are under water, and cavitation is never an issue, because there is no "spike" in pressure as the water flows through the pump.

Pump selection calculations are contained in Appendix D, and are summarised in Table 36.

Table 36 - Pump selection details

Pump Location	Pump Type	Quantity		Pump Rating		Total kW
		Duty	Stand BY	Hp	Kw	Rating,
B-1_2030, QTotal=25.5LPS, H=6M	Submersible sump pump	1	1	5	3.7	3.7
B-1_2040, QTotal=38LPS, H=8M	Submersible sump pump	1	1	7.5	5.5	5.5
					Total, kW	**9.2**
B-2_2030, QTotal=66LPS, H=16M	Submersible sump pump	1	1	10	7.5	7.5
B-2_2040, QTotal=98.4LPS, H=8M	Submersible sump pump	1	1	20	15	15
					Total, kW	**22.5**
B-3_2030, QTotal=66LPS, H=6M	Submersible sump pump	1	1	10	7.5	7.5
B-3_2040, QTotal=98.4LPS, H=7M	Submersible sump pump	1	1	15	11	11
						18.5

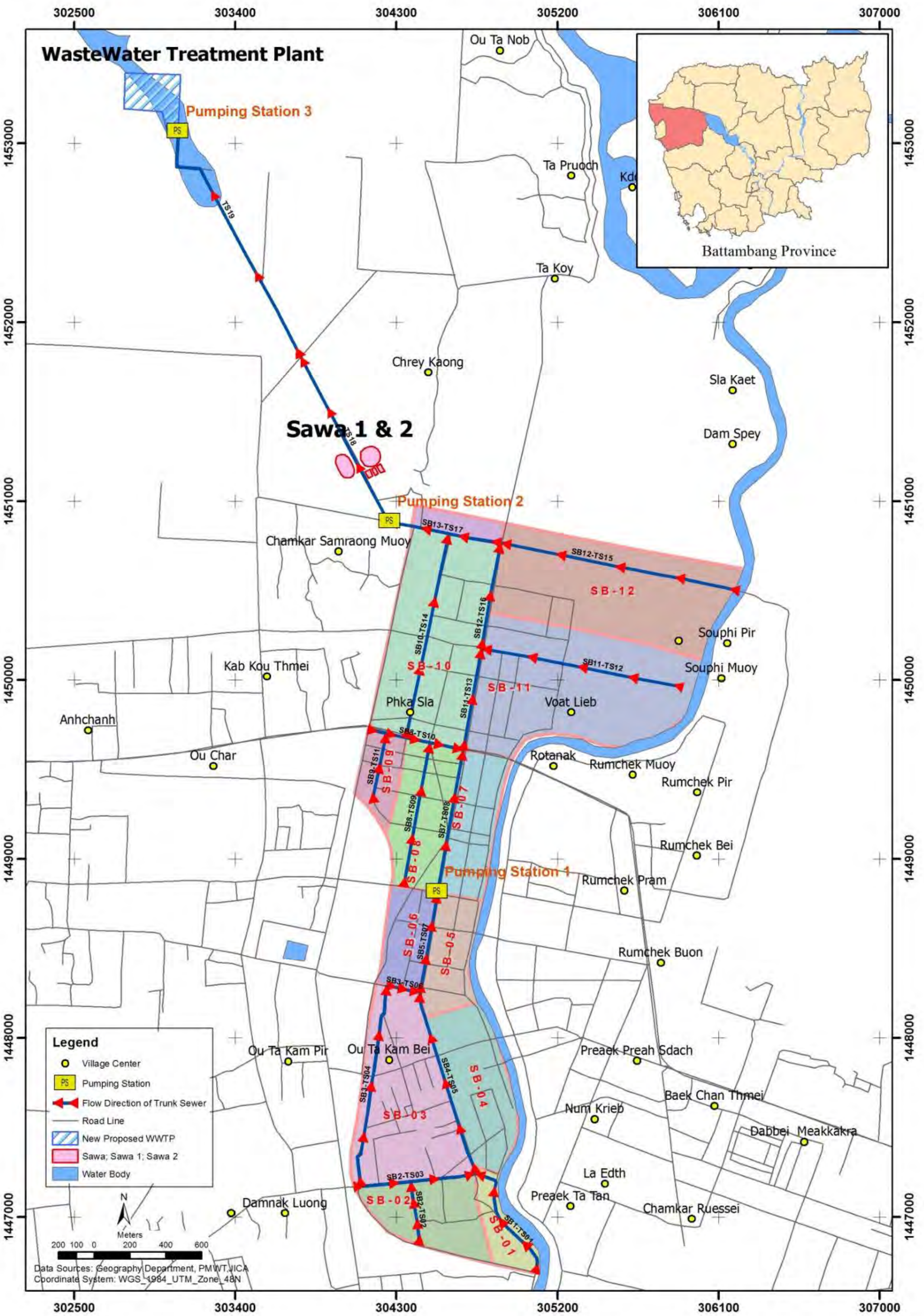
8.4 Duty-Standby

The pump system will occur during operation, wherein, one pump will operate when pumping is required while the other pump is on a Stand-by status. Upon reaching the low water level, the duty pump or pump in operation will stop and upon reaching High water level where pumping is required again, previously stand-by pump will now operate as duty pump. In this arrangement of operation, no pump is non-operational or stagnant for a long period of time as the two (2)- pumps, 1-Duty/1-Stand by pumps are operating alternatively. On the other hand, when the water level reached the High-High water level, the stand-by pump will operate to assist the duty pump in pumping water until it reaches the Low water level which will send a signal to the controller that both pumps will stop.
Above mode of operation will be utilized on the 2030 design period while a third pump will be added on the 2040 design period. Thus, a configuration of two (2)-Duty/ 1-Stand-by pump mode of operation will then be utilized. Again, said three (3) pumps will operate alternatively thus making no pumps non-operational or stagnant for a long period of time.

8.5 O&M

Pump/Lifting stations should be regularly monitored and visited on a regular interval to see if it operates normally and no irregularities and abnormalities in its operation is happening. Removal of debris and sediments that cannot be pumped should be removed manually to avoid accumulation of the debris in the sump. Clogging of the pump system and its piping could also occur thus will result to down time in its operation. A by-pass pumping system utilizing an external pump should be integrated in the design of the discharge piping to enable emergency pumping especially during brown outs or non-availability of power thus also preventing overflow of the sump.

Figure 12 - Location of pumping station and route of pumping main



9 Septage Treatment

9.1 Septage collection

With only 17%[4] of the central city area (Phase 1) currently connected to the wastewater treatment plant, the majority of urban properties are reliant on either a septic tank or concrete holding/soakaway tank (pour flush pit latrine).

Septage in Battambang is currently collected by 7 private vacuum trucks. Demand from householders to have their septic tanks de-sludged is based on when they block or cause a smell nuisance rather than by preventative maintenance or periodic checking of sludge depth.
There is no record of numbers, volumes or condition of septic tanks in the city. A septic tank survey and public awareness campaign are recommended and are described in detail in the "Sector Analysis, Development Roadmap and 20 Year Investment Roadmap for Urban Sanitation" part of the Draft Final Report.

Private septage collectors will require regulating once the new WWTP is opened, both to ensure that septage is co-treated in the WWTP and that private operators pay for this service. It is suggested that all septage collection operators require a license to operate in the urban area, and that a condition of this licence is that all septage is disposed of to the WWTP. An annual refresher course could be tied to the renewing of this licence to ensure that operators are aware of how to use the receiving bays of the anaerobic ponds correctly.

9.2 Septage treatment

Septage in Battambang, where collected, is currently disposed of to agricultural land on the edges of the town. If the new WWTP proposal option of provision of trickling filters are taken, co-treatment of septage is recommended in the upstream anaerobic ponds.

Receiving bays on each of the four proposed anaerobic lagoons are to be constructed, each with washdown facilities for the vacuum trucks and surrounding concrete skirt. Bar screens will be installed across the lagoon septage receiving bays to collect solids such as sanitary towels. Design measures should be taken to minimise both mess and odours during the emptying of the vacuum trucks, such as the end of the lagoon inlet pipe being under the lagoon surface.

9.3 Sludge removal from anaerobic lagoons

When the anaerobic lagoons approach being half-full with sludge, they need to be dewatered, allowed to dry to a level where sludge can be handled, and emptied.
It is unlikely that operating staff will remove treated sludge manually. This is usually due to lack of staff, funds to hire temporary labour, equipment and priorities.
Portable submersible sludge pumps are recommended to aid in dewatering, but as each of the four lagoons are 50m x 30m it is unlikely that pumps alone would enable consistent desludging across the footprint of each lagoon without a barge or floating pontoon on which to mount the pumps

It is instead recommended that concrete access tracks are incorporated into the lagoon design to allow a compact excavator ("bobcat") to access each lagoon.

9.4 Drying options

The degree of "drying" for sludge can be defined as:

- Ordinary sludge – from unsettled lagoon - 2% solids
- Thickened sludge - from settled lagoon – 5% solids
- Dewatered sludge – from drying beds – 20% solids
- Dewatered sludge – from sludge press/dewatering container – 80% solids
- Dried sludge granules – from final drier - 95% or greater solids

There are three main feasible options for sludge drying:

9.4.1 Mechanised sludge press

A mechanized sludge press could be purchased and used on the limited land available on the current lagoon site. This has the advantage of having a small footprint, but the obvious disadvantage of requiring maintenance and requiring power. A sludge drier can further dry the dewatered sludge from the press into dry granules but this is not necessary for landfill disposal.

9.4.2 Sludge dewatering container

This resembles a shipping container but is watertight and contains a filter, often a geotextile with fine holes. Polymer is added to bond small biosolids and stop them from forming a film across the filter that eventually blinds it. Whilst mixers and vacuum pumps can be added to improve results, a more basic model that would be appropriate for Battambang requires no power or moving parts other than the small mixer in the poly storage tank. The container is by an excavator from a truck, and clear filtrate water is piped back into the facultative lagoon. The dewatered sludge can then be loaded back onto a truck for delivery to landfill or land application. As de-sludging only occurs for each lagoon in rotation every year or longer, a truck can be rented or taken from PWD rather than purchased. This is an appropriate option if the lagoons are fully retained and land is limited.

9.4.3 Incorporation of drying beds into new 6.6ha site

Should the option of trickling filters be selected, there will be some remaining space which could be used for drying beds as shown in Figure 9 above. However, as space is limited, the preference would be use of a sludge dewatering container with treated dewatered sludge being disposed of to landfill.

9.5 Composting

Composting of final dried, treated sludge is worthwhile if there is a local market for the final product with no cultural aversions against using fertilizer of human origin on food crops. Composting methods are discussed in the "Sector Analysis, Development Roadmap and 20 Year Investment Roadmap for Urban Sanitation" part of the Draft Final Report.

Composting requires a fair amount of attentive manual labour in the form of turning the compost, and ensuring it is covered from rain. There is no existing market for it in Battambang or Cambodia, in particular as there are large food growing areas with naturally fertile soils from the Tonle Sap lake and many river flood plains. Composting of drier treated sludge is therefore not recommended.

9.6 Disposal to landfill

Dewatered sludge can then be used as cover for the proposed landfill rehabilitation project, which is due to be constructed in 2017. Once landfill material has been compacted in layers every 1-2 weeks, it is capped, and treated dewatered sludge is ideal for this purpose.

10 Recommended Option

10.1 Selected options

Of the 8 options described under Section 5, two were put forward under Section 6 as the most viable options. To recap, these were:

- Aerated lagoons on full 15ha (minimum) site

This option is dependent on the DPWT acquiring more land at the current new 6.6ha site. This option proposes to decommission the existing 5.8ha site, and reserve this land for future urban infrastructure needs. This proposed new (minimum) 15ha site would be developed into a single-site aerated lagoon system. This total 15ha of land can be developed into a lagoon system capable of treating the required 11,645 m^3/day, sufficient capacity to serve the Phase 1 area beyond 2040.

- Conventional trickling filters on current 6.6ha site

This option proposes both sedimentation ponds and conventional trickling filters on the new 6.6ha site. The existing “SAWA” WWTP site would be decommissioned.

10.2 Recommended option

Whilst aerated lagoons would be the preferred option, a 15ha site is required for these, and currently the site acquired is limited to 6.6ha. The aerated lagoons would be preferable due to them being simpler technology, lower cost O&M, lower capital cost, and advantages of DPWT being able to share experience with Sihanoukville and Siem Reap who have similar plants. However, the minimum 15ha land required is not currently available, and so the option put forward for further development is the conventional trickling filter option described under Sections 5.3.2 and 6.2. Should a full 15ha site become available prior to final design then this aerated lagoon option should be revisited.

With the above caveat, the option of 4 sedimentation (anaerobic) ponds with 4 conventional trickling filters, each of 2,991m^3/day capacity (to meet total year 2040 flow of 11,645m^3/d) is recommended. If the filters are 2m deep, with a flowrate of 2.5m^3/m/day, the diameter of each will be approximately 27m (see Table 29).

A concrete receiving bay with bar screens and a service water point will be added to each anaerobic lagoon to facilitate offloading of septage form vacuum trucks into the lagoons for co-treatment.
Concrete access ramps constructed to provide access for a compact excavator. A 6m^3 septage vacuum truck and sludge dewatering container will be supplied. Portable wastewater quality testing equipment will be provided.

10.3 Operation and Maintenance requirements

The operation of the WWTP will primarily be the monitoring of the flows through the conventional trickling filters, which utilise feed pumps to keep the flow constant. The main operational requirement is that flow needs to be constant, as if flow is paused the filter drains, which can cause bacteria on the media to dry out and die. This constant hydraulic loading may be difficult during low flow periods, such

as night time or during power outages. There are various design methods often utilised to maintain a constant flow, such as dosing siphons or recirculation, but only the lowest technology option is recommended for this application.
It is therefore most important for the Operator to monitor flows and be prepared to repair or replace feed pumps at short notice.

The rotating arm that distributes WW over the media surface may also have mechanical problems from time to time and require rapid maintenance to prevent loss of the filter live bacteria.

The Operator needs to monitor odour. Odour problems can occur when the centre of the filter becomes anaerobic due to not having enough air getting through, or due to excessive or unexpected loads.

The biofilm, or layer of bacteria on the media, must be kept thin to prevent the filter clogging. The filter will need cleaning with a backwash periodically to avoid this.

The main costs of O&M are from paying the salary of one or more full time operators, power costs for the pumps and motor to power the rotating arm, and occasional repairs and replacements of mechanical parts. Under normal circumstances, the power costs are the most significant of these. Table 39 summarises expected O&M costs.

Figure 13 - Table 39: Typical annual O&M costs

Item	Estimated annual cost ($)
Trickling filter pumps - spares	5,000
Trickling filter pumps- power[5]	50,000
Trickling filter arms - power	15,000
Fuel for excavator	2,000
Fuel for vacuum truck	2,000
Mechanical maintenance for vehicles	5,000
Permanent operator salary x2	25,000
Labour for desludging	10,000
Total	**114,000**

[5] 22kW running full time, $0.25/kWhr

10.4 Sustainability

The proposal must be seen to be sustainable for the investment to be attractive to all parties with a vested interest. The preliminary design of the Battambang WW collection and treatment system has been carried out with this in mind, and factors illustrating the likelihood of ongoing financial sustainability are;

- Battambang is one of the largest cities in Cambodia, and therefore has one of the largest customer bases.
- Free household connections are proposed for all properties under the project. This amounts to 7,330 households by 2020 which will be obliged to pay a WW fee.
- Street collection pipelines along both sides of every street are proposed under the preliminary design. This will enable easy connection of both current and future properties
- Battambang has current experience in collecting a drainage and WW fee. It is collected as part of the water bill at present, and managed by the Municipality. The billing process is already in place.

11 Equipment and Vehicle Purchases

The following equipment is recommended for purchase under the contract.

Table 37 - Equipment purchase

Description	Quantity	Purpose
6m3 vacuum septage truck	1	Septage collection DPWT
High flow portable sludge pumps (min 10l/sec)	2	Lagoon desludging
Compact excavator	1	Lagoon desludging
Portable influent/effluent quality testing equipment	1	Monitoring
Sludge dewatering container	1	Sludge dewatering

12 Preliminary Cost Estimate

13.1 Preliminary cost estimates for recommended option

The table below shows the preliminary cost estimate.

Table 38 - Preliminary cost estimate

Item	Unit	Unit rate	Qty	Total ($)
Preparatory earthworks	LS	100,000	1	100,000
Excavate for anaerobic ponds (2/3 below GL)	m3	10	8259	82,587
Compacted bunds around lagoons 2m high x 2m thick at base	m3	30	624	18,720
Compacted bunds around site 2m high x 2m thick at base	m3	30	6400	192,000
Liners	m2	5	6600	33,000
DCI Pipework, valves & meters inside WWTP	LS	200,000	1	200,000
Conventional trickling filter diameter 27m, 2,911m3/day	ea	1,000,000	4	4,000,000
Site access road from main road 6m wide	m	100	1000	100,000
Site roads 4m wide (gravel)	m	50	1850	92,500
Site office	LS	100,000	1	100,000
Site storeroom	LS	60,000	1	60,000
Site carparking	LS	20,000	1	20,000
Sludge drying areas (bunded)	ea	50,000	2	100,000
Solar Aerators with hoses and freight	ea	75,000	8	600,000
Concrete pad and effluent pipe for dewatering container	LS	5,000	1	5,000
Receiving bays for anaerobic ponds	ea	20,000	4	80,000
Pump stations/pump wells for WW collection	ea	150,000	3	450,000
Pumps with VSD's	ea	50,000	6	300,000
Trunk sewers	LS		1	1,032,000
Pumping mains	LS		1	500,000
WW mains (for separated connections)	LS		1	1,250,000
Household connections (year 2020)	ea	100	8,400	840,000
Supply compact excavator	ea	50,000	1	50,000
Supply 6m3 vacuum truck	ea	80,000	1	80,000
Supply sludge dewatering container	ea	100,000	1	100,000
Portable sludge pumps	ea	20,000	2	40,000
Supply WQ testing equipment	kit	25,000	1	25,000
WQ testing training	LS	10,000	1	10,000
Public awareness campaign	LS	100,000	1	100,000
Training/capacity building	LS	150,000	1	150,000
Subtotal				10,710,807
10% contingency				1071080
TOTAL				**11,781,888**

Appendix A: Topo survey

Appendix B: Aerated lagoon design and size calculations

Preliminary Engineering Design of Waste Stabilization Ponds	Project name:

Basis of Design

Data taken from World Bank Technical Paper No. 7 Notes on the Design and Operation of WSP in Warm Climates of

Type of Pond System	BOD_5 Removal Rate (%)			Coliform Removal Rate (%)		
	12^0C	20^0C	25^0C	12^0C	20^0C	25^0C
Anaerobic Pond	45	62	70	60	86	93
Anaerobic and facultative	80	88	90	96	99.5	99.2
Anaerobic and facultative and maturation	86	92	94	99.00	99.98	99.95
Anaerobic and facultative and 3 x maturation	94	95	95+	99.95	99.9996	99.99999
Facultative Pond	75	80	84	91	97	98
Facultative and maturation	86	90	93	98.20	99.94	99.98
Facultative and 3 x maturation	93	95	95+	99.90	99.998	99.99993
Aerated Lagoon	70	80	82	7	93	96
Aerated lagoon and maturation (10 day)	84	92	93	95	99.5	99.9
Aerated lagoon and 3 x maturation	93	95	95+	99.90	99.996	99.999

Retention time in anaerobic ponds	up to 2 days
Retention time in facultative ponds	7 to 15 days depending on water
Retention time in maturation ponds	5 days but 10 days if after aerated lagoons
Retention time in aerated lagoons	4 days

Flows				
Population	84,016		Equivalent @126 l/person.d	
Per capita flow	126	l/person.day		
Total average flow	10,586	m^3/d		
Infiltration	1,059	m^3/d	10.00	%
Total average flow	11,645	m^3/d		
Organic load				
Per capita BOD load	45.88	g/person.day		
Domestic load	3,855	kg/day		
Industrial load	0	kg/day		
Total BOD load	3,855	kg/day		
Lowest monthly mean ambient temperature	24	^0C		

Anaerobic Ponds

Volumetric loading rate	0.50	kg BOD/day/m^3	(between 0.1 to 0.4 kg BOD/day/m^3 depending on lowest average temperature. 0.1 when temp.< 14^0C and 0.4 when
Volume of ponds	7,709	m^3	
Detention time	0.66	days	
Adopt depth of	4.00	m	
Area of ponds at mid depth	1,927	m^2	
Number of ponds	4		
Area of each ponds at mid depth	482	m^2	
Length to width aspect ratio	2 to 1		
Pond side slope	1 to 2		
Pond width at mid depth	15.52	m	
Pond length at mid depth	31.04	m	
Pond width at top water level	24.52	m	
Pond length at top water level	40.04	m	
Pond width at top water level (rounded)	25.0	m	
Pond length at top water level (rounded)	40.0	m	
Adopt BOD removal from Table above	65	%	
BOD load in effluent	1,349	kg/day	
Cross pond liquid area (m2)	4,000		
Cross pond liquid area (ha)	0.40		

Facultative Ponds

Lowest monthly mean ambient temperature	24	^0C	
BOD load in influent to facultative ponds	1,349	kg/day	
Areal loading rate y_s	20T - 60	kg BOD/ha.day (where T = minimum mean monthly	
	900	kg BOD/ha.day	
Facultative pond area at mid depth	BOD load/y_s		
	1.499	ha	
	14,990	m^2	
Adopt depth of	1.75	m	
Facultative pond volume	26,233	m^3	
Detention time	2.25	days	
Length to width aspect ratio	3 to 1		
Pond side slope	1 to 2		
Number of ponds	2		
Area of each ponds at mid depth	7,495	m^2	
Pond width at mid depth	49.98	m	
Pond length at mid depth	149.95	m	
Pond width at top water level	56.98	m	
Pond length at top water level	156.95	m	
Pond width at top water level (rounded)	57.0	m	
Pond length at top water level (rounded)	157.0	m	
Adopt BOD removal from Table above	90	%	
BOD load in effluent	134.9	kg/day	
BOD load in effluent	0.012	mg/l	
Cross pond liquid area (m2)	17,898.00		
Cross pond liquid area (ha)	1.790		

Maturation Ponds

Influent bacterial concentration (B_i)	5	x 10^7	FC/100 ml
Lowest monthly mean ambient temperature	24	^0C	
Factor for temperature correction (T-20)	4		
Temp. corrected FC removal rate (K_B(T))	5.21		
Adopt number of ponds	2		
Adopt hydraulic retention time for each	5	days	
Pond volume (each)	58,223	m^3	
Adopt pond depth of	1.50	m	
Each pond area at mid depth	38,815	m^2	
Length to width aspect ratio	3 to 1		
Pond side slope	1 to 2		
Pond width at mid depth	113.75	m	
Pond length at mid depth	341.24	m	
Pond width at top water level	119.75	m	
Pond length at top water level	347.24	m	
Pond width at top water level (rounded)	120	m	
Pond length at top water level (rounded)	348	m	
Gross pond liquid area	83520	m^2	
Gross pond liquid area	8.35	ha	

Effluent bacterial concentration (Be) = Bi / (((1+(K_B(T))xtan)) x ((1+(K_B(T))xtfac)) x

tan	0.66	detention time in anaerobic pond
tfac	2.25	detention time in facultative pond
tmat	5.00	detention time in maturation pond
n	2.00	no. of maturation ponds in series
Be	1,202.55	FC/100 ml

Adopt BOD removal from Table above	95	%
BOD load in effluent	771	kg/day
BOD load in effluent	0.07	mg/l
Total pond liquid area Requirement (ha)	10.54	
Road access and free space- 35% (ha)	3.69	
Total	14.23	

Appendix C: Solarbee technical brochures

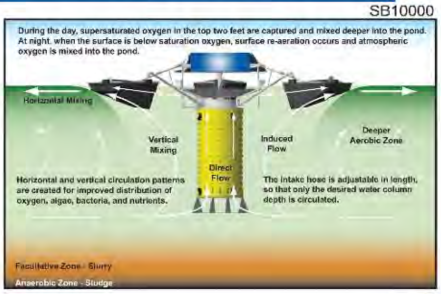
GridBee®
SolarBee®

Wastewater Benefits

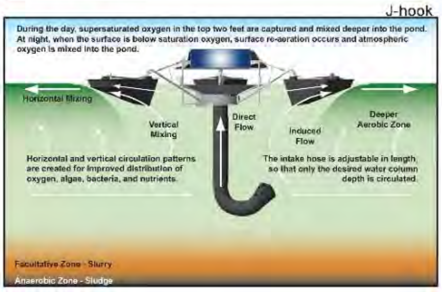
SolarBee and GridBee mixers are designed to solve a variety of municipal and industrial wastewater quality problems including high energy costs, EPA discharge permit violations and odor control. SolarBee and GridBee mixers can operate 24/7 to improve the treatment system, supplying most of the mixing energy required in any treatment pond and reducing the hours that the aeration system must operate.

Flow rates vary by mixer size and range from 1,250 to 10,000 gallons per minute. For each application, a custom proposal is provided at no charge, which shows the expected results.

SB10000



J-hook



Wastewater Benefits:

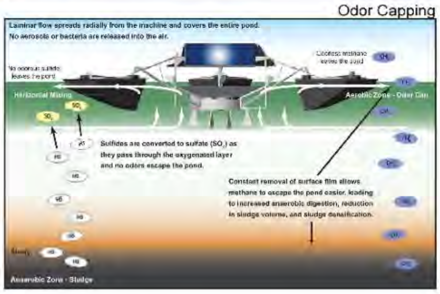
- Reduces energy consumption by reducing run time of existing aeration/mixing equipment
- Provides improved mixing in all systems
- Improves BOD, TSS and ammonia reduction
- Controls odors (odor capping systems available)
- Improves sludge digestion, reduces the need for dredging
- Reduces short-circuiting and fecal coliform counts

Dual Mix

Features:

- 316 stainless steel and non-corrosion polymer construction
- Intake designs specific to the application
- 2-year machine warranty, 10-year motor warranty, 25-year service life
- Factory installation, water testing and sludge depth testing available
- Solar units: Day and night operation on solar power
- Solar units: Digital control system programmable with anti-jam and auto-reverse; SCADA outputs available
- Solar units: High-efficiency, high-torque brushless motor with no gearbox for low-speed operation

Odor Capping



Applications:

- All aerobic wastewater ponds
- Activated sludge reactor basins
- Water reuse / effluent storage ponds
- Odor capping of anaerobic ponds

SolarBee® and GridBee® are brands of Medora Corporation

Locally Represented By:

Medora Corporation
3225 Hwy 22 • Dickinson, ND 58601
Ph +1 866 437 8076 • www.medoraco.com

SolarBee®

Circulating the World's Water

SB10000 v18 FEATURES

Technology Description:	Floating, solar powered, circulation equipment for wastewater treatment and freshwater applications. Day/night operation on solar only by utilizing a battery to store excess daytime energy for nighttime operation.
Flow Rates:	*Flow rates at full speed at 10 feet (3 meter) diameter.*
Direct Flow Rate - Day	3,000 gallons per minute, (12,000 Liters per minute).
Induced Flow Rate - Day	7,000 gallons per minute. (26,000 Liters per minute).
Combined Flow Rate - Day	10,000 gallons per minute. (38,000 Liters per minute).
Combined Flow Rate - Night	10,000 gallons per minute (38,000 Liters per minute).
Machine Size/Weight:	Assembled machine is 16 feet (5m) in diameter and weighs 850 pounds (385kg).
Shipping Size/Weight:	Machine can be crated and shipped in a 87 inch (2.2 m) wide X 87 inch (2.2 m) long X 65 inch (1.7 m) high crate, estimated shipping weight is 1500 pounds (680 kg). Hose lengths greater than 40 ft (6 m) may require additional crating.
Materials of Construction:	316 stainless steel construction. Foam-filled high-density polyethylene (HDPE) floats. Thermoplastic rubber intake hose. HDPE strainer. Concrete mooring blocks are encapsulated in HDPE.
Drive System:	High torque, direct drive (no gearbox), low voltage brushless D.C. motor.
Power Supply/Control System:	*PV solar panels are protected from bird fouling with bird deterrent kit.*
PV Solar Panels	3 X 80-watt photovoltaic solar panels orientated in triangular pattern. On-board battery storage for day/night operation.
Electronic Controller	Digital solid-state controller, mounted in weather-tight (NEMA 4X) enclosure with externally fused disconnect. SCADA output through RS-232 serial communication (Modbus RTU), DB9 male connection point inside enclosure. Wireless options available, not included.
Wiring	Corrosion-resistant industrial cord with molded watertight connectors that are indexed to prevent improper wiring.
Rotating Assembly:	Removable assembly with easy access to motor and digital controller. Impeller handles 4-inch (10 cm) spherical solids. Oil-filled (food grade) teflon freeze sleeve with o-rings, shaft. Rotational indicator on shaft.
Flotation System:	Three floats in triangular pattern each with an adjustable float arm for proper vertical positioning, total float buoyancy of 1,350 pounds (612kg).
Fluid Intake Assembly - Option 1:	*Hose system bolted to bottom of structural assembly.*
Hose System	10 to 100 feet (3 to 30.5 meters) available in 36 inch (91 cm) diameter X 10, 15, or 20 feet (3, 4.5, 6 meter) sections.
Intake Type	Horizontal plate with 12-inch (31 cm) openings.
Intake Depth Adjustment	15 feet (4.5 m) of field adjustment with three SS chains connected to hose coupling.
Fluid Intake Assembly - Option 2:	Fixed horizontal plate bolted to bottom of structural assembly.
Intake Type	Fixed horizontal plate with 12-inch (31 cm) openings.
Intake Depth Adjustment	No adjustment necessary. Horizontal inflow from 25 in (64 cm) below distribution dish.
Fluid Intake Assembly - Option 3:	*See SB10000DM v18 Dual Mix Features Sheet.*
Anchoring:	(1) Two mooring blocks tethered together with SS chain and attached to structural member on unit or (2) Tethered to shore with SS cable.
Ice Protection:	Freeze sleeve and positive pumping under distribution dish to maintain circulation.
Minimum Operating Depth:	31 inches (0.8 m) with fixed horizontal plate. No damage to machine or bottom of reservoir when run dry in shallow water.
Accessories Available:	(1) Supplemental Shore Power Kit, (2) Chemical Injection Kit, (3) Marker Light Kit
Life/Maintenance/Warranty:	Expect 25-year life, minimal maintenance. Limited 2-year parts and conditional labor warranty. Limited 25 year photovoltaic module manufacturer performance warranty and 10 year motor warranty.

Patent Pending

Subject to change without notice.

Medora Corporation • 3225 Hwy 22 • Dickinson, ND 58601
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10000v18-FEATURE_20150714

Appendix D: Pump selection

	Contract No.:			Job No.:	-
				Sheet No.:	1 of 1
	Project:	Battambang_SANITATION		Revision No.:	
	Designed by:	Date:	Checked	Date:	
	F.S.ABRIGO	19-Oct-16	-	-	

SHEET TITLE: ***B-1 PS_Battambang, SUMP/PUMP SIZING AT 2,203 cu.M/day (2033)***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 2,203.0 m^3/day = 25.50 lps
1.530 m^3/min = 91.8 m^3/hr
= 404.2 gpm
No. of Pumps = 1 + 1 **Stand-By**
Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.
Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)
Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	6.15 ft.	K Factor
Piping	0.1126	6.00
Check Valves	1.044	3.00
Ball Valves	0.052	0.15
Elbows	0.063	0.18
Tee	0.209	0.60
Total H_L	**1.48** ft.	
	0.45 m	

Headloss for Valves & Fittings $= k(v^2/2g)$
where: $(v^2/2g)$ = 0.11
Headloss for Straight Pipe = k*(L/100)
Length of Force main, m = 20.0
Diameter of Force main, mm = 0.5
K Factor = 0.01
Total Headloss at Force Main, m: **0.0066**

C. DESIGN CALCULATION

Q per pump = 25.50 lps
= 91.79 m^3/hr
= 404.19 gpm

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min.
q =pump capacity, m^3/min

Volume Required: For Maximum 10 cyc/hour = 2.29 m^3
For Minimum 6 cyc/hour = 3.82 m^3
Volume Required: *6.05* ***m^3***
Wet Well Volume Required: *USE:* **2.20** ***m^3***

Water in Sump Vol.$_{Total}$ = **2.2**
Water in Pipe Vol.$_{Total}$ = **3.85**
(incoming)

Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55** **m**
Riser Pipe Diameter Required: = 0.096 m say **0.15** **m**

T D H$_{Required}$ = PWL + Discharge Head + Headloss$_{Total}$
where: PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}\text{-HWL})+(DP\ el.\text{-}PS_{GL})+HL_{FM}$ = 5
Total Headloss= = 0.5
Total Dynamic Head$_{Required}$ = 5.76 m say = **6** m
Pump Efficiency = 70.0%
Power Required = 3.45 hp say **5** **hp**
= 3.73 KW say **3.7** **kW**

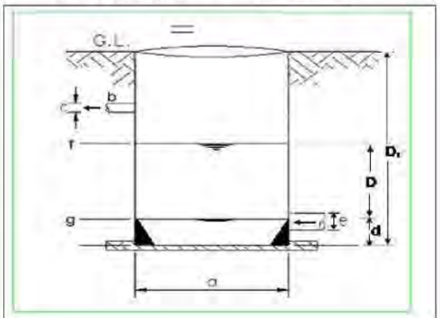
Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%
*			

Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **0.772** **m**

To Check for **Submergence = S=D*(1 + 2.3F)**
Where: **F= v/(gD)$^{1/2}$** = **3.278**
V= vel. @ suction bell, m/s = 3.246
D= O.D of Suction bell, m = 0.1
Submergence= 0.854 **m**

FOR PUMP's DISCHARGE VELOCITY
Q $_{Required}$ = 0.0255 m^3/s
Pipe Area = 0.0177 m^2
Velocity = **1.44** m/s < 3.5 m/s **'PASSED'**
FOR SUCTION VELOCITY:
v= vel.@ suction bell, m/s = **3.25**



SUMP DETAILS

ITEM	DESCRIPTIONS	DIMENSION, m	ELEVATION , m
PS_{GL}	Ground Level	n/a	12.090
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	10.890
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	5.50	
e	Incoming Pipe Diameter	0.7	6.790
	I.L of Incoming Pipe		6.090
f	Pump Start, **(HWL)**		7.340
g	Pump Stop Level **(LWL)**	0.55	6.790
	Pump Min. Water Level Reqt. **(d)**	0.85	
h	Wet well Depth	1.40	
i	Dis. From Incoming Pipe to Bottom	1.40	5.636
BL	Bottom Level	n/a	5.636
DP_{GL}	Discharge Point Ground level		12.09

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS
(For Structural Design Reference)

7.84	**HHWL,** m
7.34	**HWL,** m
6.05	**Volume** $_w$, cu.m
6.79	**LWL,** m
6.49	**LLWL,** m
0.85	**d**, m
1.40	**D + d**, m
6.45	**Depth** $_{TOTAL}$

Contract No.:			Job No.:	-
			Sheet No.:	1 of 1
Project:	Battambang_SANITATION		Revision No.:	
Designed by:	Date:	Checked / Date:		
F.S.ABRIGO	20-Oct-16	- / -		

SHEET TITLE: ***B-1 PS_Battambang-2040, SUMP SIZING AT 3,265 cu.M/day***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 3,265 m^3/day = 37.79 lps
2.267 m^3/min = 136.0 m^3/hr
= 599.0 gpm
No. of Pumps = 1 + 1 **Stand-By**
Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.
Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)
Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	7.50 ft.	K Factor
Piping	0.1371	6.00
Check Valve:	2.293	3.00
Ball Valves	0.115	0.15
Elbows	0.138	0.18
Tee	0.459	0.60
Total H_L	**3.14** ft.	
	0.96 m	

Headloss for Valves & Fittings = $k(v^2/2g)$
where: $(v^2/2g)$= 0.233
Headloss for Straight Pipe = k*(L/100)
Length of Force main, m = 1,528.0
Diameter of Force main, mm = 0.7
K Factor = 0.01
Total Headloss at Force Main: **0.5012**

C. DESIGN CALCULATION

Q per pump = 37.79 lps
= 136.04 m^3/hr
= 599.04 gpm
Volume Required: For Maximum 10 cyc/hour = 3.40 m^3
For Minimum 6 cyc/hour = 5.67 m^3
Volume Required: *6.05* ***m^3***
Wet Well Volume Required: *USE:* **2.20** ***m^3***
Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55 m**
Riser Pipe Diameter Required: = 0.117 m say **0.15** m

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min
q =pump capacity, m^3/min

Water in Sump Vol.$_{Total}$= **2.2**
Water in Pipe Vol.$_{Total}$= **3.85**
(incoming)

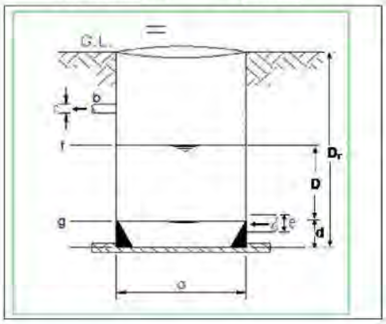
T D $H_{Required}$ = PWL + Discharge Head + $Headloss_{Total}$
where:
PWL= HWL-LWL = 0.55 *
Discharge Head = $(PS_{GL}$-HWL)+(DP el.-PS_{GL})+HL_{FM} = 5
Total Headloss= = 1.5
Total Dynamic $Head_{Required}$ = **7.26** m say = **8** m
Pump Efficiency = 68.0%
Power Required = 6.62 hp say **7.5** hp
= 5.60 KW say **5.5** kW

Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%

Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **0.772 m**
To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **$F= v/(gD)^{1/2}$** = **14.261**
V= vel. @ suction bell, m/s = 11.388
D= O.D of Suction bell, m = 0.065
Submergence= 2.197 **m**

FOR PUMP's DISCHARGE VELOCITY
Q $_{Required}$ = 0.0378 m^3/s
Pipe Area = 0.0177 m^2
Velocity = **2.14** m/s < 3.5 m/s **'PASSED'**
FOR SUCTION VELOCITY:
v= vel.@ suction bell, = **11.39**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION, m**
PS_{GL}	Ground Level	n/a	12.090
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	10.890
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	5.50	
e	Incoming Pipe Diameter	0.7	6.790
	I.L of Incoming Pipe		6.090
f	Pump Start, **(HWL)**		7.340
g	Pump Stop Level **(LWL)**	0.55	6.790
	Pump Min. Water Level Reqt. **(d)**	2.20	
h	Wet well Depth	2.75	
i	Dis. From Incoming Pipe to Bottom	2.75	4.293
BL	Bottom Level	n/a	4.293
DP_{GL}	Discharge Point Ground level		12.09

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS (For Structural Design Reference)	
7.84	**HHWL**, m
7.34	**HWL**, m
6.05	**Volume** $_W$, cu.m
6.79	**LWL**, m
6.49	**LLWL**, m
2.20	**d**, m
2.75	**D + d**, m
7.80	**Depth** $_{TOTAL}$

Contract No.:				Job No.:	-
				Sheet No.:	1 of 1
Project:	Battambang_SANITATION			Revision No.:	
Designed by:	Date:	Checked	Date:		
F.S.ABRIGO	20-Oct-16	-	-		

SHEET TITLE: ***B-2.PS_Battambang, SUMP SIZING AT 5,712 cu.M/day***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow	= 5,712.0 m³/day	=	66.11	lps	
	3,967 m³/min	=	238.0	m³/hr	
		=	1,048.0	gpm	
No. of Pumps		=	1	+ 1 **Stand-By**	
Recommended Pump Cycle	maximum	=	10	cycle/hour	
		=	6	mins.	
	minimum	=	6	cycle/hour	
		=	10	mins.	
Design Velocity					
at Suction:	For Pump Entrance of larger than	=	3.6	m/s	200 mm dia.,
	For Pump Entrance of smaller than	=	5.5	m/s	200 mm dia.,
at Discharge:	Should be Less Than	=	3.5	m/s	(As per recommendation)
Length of Rectangular Sump		=	2	m	
Width of Rectangular Sump		=	2	m	
Diameter of Circular Sump		=	2	m	

Total Friction Loss in Pipes & Fittings

(Pipe length)	6.47 ft.	K Factor
Piping	0.1183	6.00
Check Valves	2.221	3.00
Ball Valves	0.111	0.15
Elbows	0.133	0.18
Tee	0.444	0.60
Total H_L	**3.03** ft.	
	0.92 m	

Headloss for Valves & Fittings = $k(v^2/2g)$
where: $(v^2/2g)$= 0.23
Headloss for Straight Pipe = $k*(L/100)$
Length of Force main, m = 20.0
Diameter of Force main, mm = 0.3
K Factor = 0.01
Total Headloss at Force Main: **0.0066**

C. DESIGN CALCULATION

Q per pump		=	66.11	lps
		=	238.00	m³/hr
		=	1,047.99	gpm
Volume Required:	For Maximum 10 cyc/hour	=	5.95	m³
	For Minimum 6 cyc/hour	=	9.92	m³
Volume Required:			*6.05*	***m³***
Wet Well Volume Required:		***USE:***	**2.20**	***m³***
Height of The Sump:	Rectangular	=	0.55	m
	Circular	=	*0.70*	m
USE:	**Rectangular**	=	**0.55**	**m**
Riser Pipe Diameter Required:		=	0.155	m say **0.2 m**

Volume, $_{Req}$ **=(Φ*q)/4** M³
where:
Φ =minimum time in minutes, min.
q =pump capacity, m³/min

Water in Sump Vol.$_{Total}$ = **2.2**
Water in Pipe Vol.$_{Total}$ = **3.85**
(Incoming)

T D H$_{Required}$ = PWL + Discharge Head + Headloss$_{Total}$

where:

PWL= HWL-LWL	=	0.55
Discharge Head = $(PS_{GL}$-HWL)+(DP el.-$PS_{GL)}$+HL_{FM}	=	5
Total Headloss=	=	0.9
Total Dynamic Head$_{Required}$ = **6.48** m	say =	**7** m
Pump Efficiency	=	71.0%
Power Required	=	9.91 hp say **10 hp**
	=	7.46 KW say **7.5 kW**

Pump Brand/Model	Q, m³/min	TDH, m	Eff.,%
*			

Choose: *

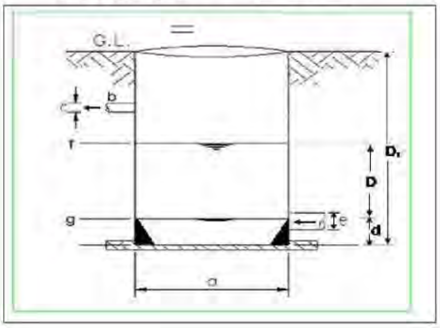
Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **0.772 m**

To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **F= $v/(gD)^{1/2}$** = **3.084**
V= vel. @ suction bell, m/s = 3.741
D= O.D of Suction bell, m = 0.15
Submergence= 1.214 **m**

FOR PUMP's DISCHARGE VELOCITY

Q $_{Required}$	=	0.0661	m³/s
Pipe Area	=	0.0314	m²
Velocity	=	**2.10**	m/s < 3.5 m/s **'PASSED'**

FOR SUCTION VELOCITY:
V= vel.@ suction bell, m/s = **3.74**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION, m**
PS_{GL}	Ground Level	n/a	8.970
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	7.770
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	5.45	
e	Incoming Pipe Diameter	0.7	3.717
	I.L of Incoming Pipe		3.017
f	Pump Start, **(HWL)**		4.267
g	Pump Stop Level **(LWL)**	0.55	3.717
	Pump Min. Water Level Reqt. **(d)**	1.21	
h	Wet well Depth	1.76	
i	Dis. From Incoming Pipe to Bottom	1.76	2.203
BL	Bottom Level	n/a	2.203
DP_{GL}	Discharge Point Ground level		8.97

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS (For Structural Design Reference)	
4.77	**HHWL,** m
4.27	**HWL,** m
6.05	**Volume** $_W$, cu.m
3.72	**LWL,** m
3.42	**LLWL,** m
1.21	**d**, m
1.76	**D + d**, m
6.77	**Depth** $_{TOTAL}$

	Contract No.:			Job No.:	-
				Sheet No.:	1 of 1
	Project:	Battambang_SANITATION		Revision No.:	
	Designed by: F.S.ABRIGO	Date: 5-Oct-16	Checked: -	Date: -	
SHEET TITLE:	***B-2.PS_Battambang, SUMP SIZING AT 8,505 cu.M/day***				

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 8,505.0 m^3/day = 98.44 lps
5.906 m^3/min = 354.4 m^3/hr
= 1,560.4 gpm
No. of Pumps = 1 + 1 **Stand-By**
Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.

Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)

Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	6.33 ft.	K Factor
Piping	0.1158	6.00
Check Valves	4.924	3.00
Ball Valves	0.246	0.15
Elbows	0.295	0.18
Tee	0.985	0.60
Total H_L	**6.57** ft.	
	2.00 m	

Headloss for Valves & Fittings $= k(v^2/2g)$
where: $(v^2/2g)$ = 0.5
Headloss for Straight Pipe $= k*(L/100)$
Length of Force main, m = 20.0
Diameter of Force main, mm = 0.4
K Factor = 0.01
Total Headloss at Force Main: 0.0066

C. DESIGN CALCULATION

Q per pump = 98.44 lps
= 354.38 m^3/hr
= 1,560.43 gpm
Volume Required: For Maximum 10 cyc/hour = 8.86 m^3
For Minimum 6 cyc/hour = 14.77 m^3
Volume Required: *6.05 m^3*
Wet Well Volume Required: *USE:* **2.20** m^3
Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55 m**
Riser Pipe Diameter Required: = 0.189 m say **0.2** m

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min
q =pump capacity, m^3/min

Water in Sump $Vol._{Total}$ = **2.2**
Water in Pipe $Vol._{Total}$ = **3.85** (incoming)

T D $H_{Required}$ = PWL + Discharge Head + $Headloss_{Total}$
where: PWL= HWL-LWL = 0.55 *
Discharge Head = $(PS_{GL}$-HWL)+(DP el.-PS_{GL})+HL_{FM} = 5
Total Headloss= = 2.0
Total Dynamic $Head_{Required}$ = 7.50 m say = **8** m
Pump Efficiency = 76.0%
Power Required = 15.95 hp say **20.0 hp**
= 14.92 KW say **15 kW**

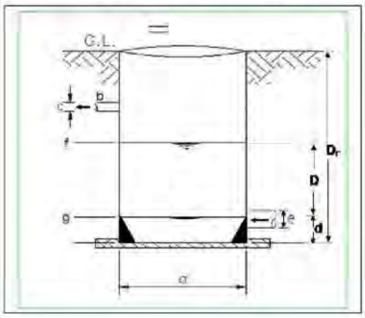
Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%

Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **0.772 m**

To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **$F = v/(gD)^{1/2}$** = **2.237**
V= vel. @ suction bell, m/s = 3.133
D= O.D of Suction bell, m = 0.2
Submergence= 1.229 **m**

FOR PUMP's DISCHARGE VELOCITY
$Q_{Required}$ = 0.0984 m^3/s
Pipe Area = 0.0314 m^2
Velocity = **3.13** m/s < 3.5 m/s **'PASSED'**
FOR SUCTION VELOCITY:
V= vel.@ suction bell, m/s = **3.13**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION, m**
PS_{GL}	Ground Level	n/a	8.970
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	7.770
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	5.30	
e	Incoming Pipe Diameter	0.7	3.870
	I.L of Incoming Pipe		3.170
f	Pump Start, **(HWL)**		4.420
g	Pump Stop Level **(LWL)**	0.55	3.870
	Pump Min. Water Level Reql. **(d)**	1.23	
h	Wet well Depth	1.78	
i	Dis. From Incoming Pipe to Bottom	1.78	2.341
BL	Bottom Level	n/a	2.341
DP_{GL}	Discharge Point Ground level		8.97

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS (For Structural Design Reference)	
4.92	**HHWL**, m
4.42	**HWL**, m
6.05	**Volume** $_W$, cu.m
3.87	**LWL**, m
3.57	**LLWL**, m
1.23	**d**, m
1.78	**D + d**, m
6.63	**Depth** $_{TOTAL}$

	Contract No.:			Job No.:	-
				Sheet No.:	1 of 1
	Project:	Battambang_SANITATION		Revision No.:	
	Designed by: F.S.ABRIGO	Date: 20-Oct-16	Checked: -	Date: -	
SHEET TITLE:	***B-3.PS_Battambang, SUMP/Pump SIZING At 5,712 cu.M/day (2030)***				

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 5,712.0 m^3/day = 66.11 lps
3,967 m^3/min = 238.0 m^3/hr
= 1,048.0 gpm

No. of Pumps = 1 + 1 **Stand-By**

Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.

Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)

Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	5.09 ft.	K Factor
Piping	0.0931	6.00
Check Valves	2.221	3.00
Ball Valves	0.111	0.15
Elbows	0.133	0.18
Tee	0.444	0.60
Total H_L	**3.00** ft.	
	0.92 m	

Headloss for Valves & Fittings $= k(v^2/2g)$
where: $(v^2/2g)$= 0.23
Headloss for Straight Pipe $= k*(L/100)$
Length of Force main, m = 20.0
Diameter of Force main, mm = 0.4
K Factor = 0.01
Total Headloss at Force Main: **0.0066**

C. DESIGN CALCULATION

Q per pump = 66.11 lps
= 238.00 m^3/hr
= 1,047.99 gpm

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min.
q =pump capacity, m^3/min

Volume Required: For Maximum 10 cyc/hour = 5.95 m^3
For Minimum 6 cyc/hour = 9.92 m^3
Volume Required: *6.05* ***m^3***
Wet Well Volume Required: *USE:* **2.20** ***m^3***

Water in Sump Vol.$_{Total}$ = **2.2**
Water in Pipe Vol.$_{Total}$ = **3.85**
(Incoming)

Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55** **m**
Riser Pipe Diameter Required: = 0.155 m say **0.2** **m**

T D $H_{Required}$ = PWL + Discharge Head + $Headloss_{Total}$
where: PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}$-HWL)+(DP el.-PS_{GL})+HL_{FM} = 4
Total Headloss= = 0.9
Total Dynamic Head$_{Required}$ = **5.52** m say = **6** m
Pump Efficiency = 69.0%
Power Required = 8.68 hp say **10** **hp**
= 7.46 KW say **7.5** **kW**

Pump Brand/Model	Q, m^3/min	TDH, m	Eff..%
*			

Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **1,190.0** **m**

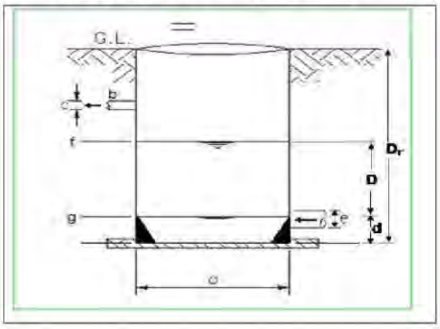
To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **F= v/(gD)$^{1/2}$** = **1.502**
V= vel. @ suction bell, m/s = 2.104
D= O.D of Suction bell, m = 0.2
Submergence= 0.891 **m**

FOR PUMP's DISCHARGE VELOCITY

Q $_{Required}$ = 0.0661 m^3/s
Pipe Area = 0.0314 m^2
Velocity = **2.10** m/s < 3.5 m/s **'PASSED'**

FOR SUCTION VELOCITY:

V= vel.@ suction bell, m/s = **2.10**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION , m**
PS_{GL}	Ground Level	n/a	7.450
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	6.250
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	4.40	
e	Incoming Pipe Diameter	0.7	3.250
	I.L of Incoming Pipe		2.550
f	Pump Start, **(HWL)**		3.800
g	Pump Stop Level **(LWL)**	0.55	3.250
	Pump Min. Water Level Reqt. **(d)**	0.89	
h	Wet well Depth	1.44	
i	Dis. From Incoming Pipe to Bottom	1.44	2.059
BL	Bottom Level	n/a	2.059
DP_{GL}	Discharge Point Ground level		7.45

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS	
(For Structural Design Reference)	
4.30	**HHWL,** m
3.80	**HWL,** m
6.05	**Volume** $_{w,}$ cu.m
3.25	**LWL,** m
2.95	**LLWL,** m
0.89	**d**, m
1.44	**D + d**, m
5.39	**Depth** $_{TOTAL}$

Appendix 4
Preliminary Design Report – Sihanoukville Subproject

Appendix H4 Preliminary Design Report –Sihanoukville Subproject

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1. Introduction

The CDIA TA subproject for Sihanoukville wastewater aims to address the priority areas selected by MPWT and DPWT by designing and constructing a separated sewer network, installing free domestic connections, and upgrading the existing WWTP to increase capacity to enable the additional WW load to be treated to national effluent quality standards. The upgrade of the current WWTP is limited to the current land footprint – no more land is available – and several options for increasing wastewater treatment capacity are discussed.

2. Current Situation

2.1 Existing system

The existing lagoon wastewater treatment plant was contracted and commissioned in 2008 under the ADB “Provincial Towns Improvement Project”. The wastewater treatment plant (WWTP) was designed with the total capacity of 6,900 m^3/day, of which 5,700 m^3/day was for households (3,368 households) and 1,200 m^3/day for industry (Cambrew only was considered during design). This design capacity came from projections to year 2010 only[1]. The sewerage system currently serves the central part of the town with total area 321ha in Sangkhat Pir (sangkhat 2), Sankhat Buon (sangkhat 4) and Sangkhat Mouy (sangkhat 1). The proportion of coverage in each area is; Sangkhat 2 (38.7%), Sangkhat 4 (8.6%) and Sangkhat 1 (0.4%). The current total population inside this service area is estimated to be about 21,341 people. Approximately 52% of this population is currently connected. The service area from the PTI project is shown in Figure 1 overleaf.

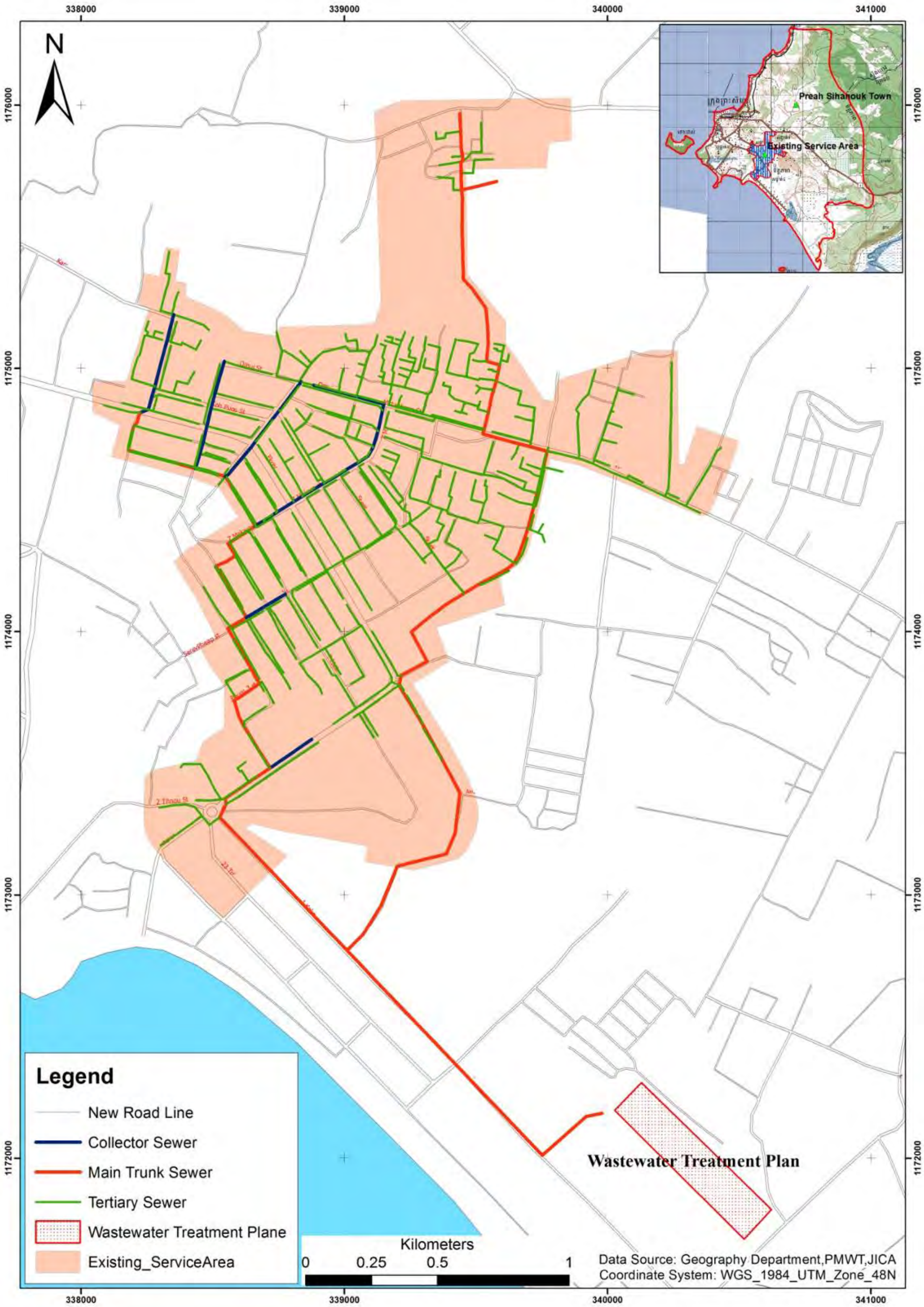
The system is separated, and 7km of collector pipe and 51km of reticulation pipe was laid. Only around 665 connections were originally made due to people not wanting to pay for their connection. This has since risen to 1,767 in September 2016. The system is now operating well, and revenue being collected, but is reaching capacity as the industrial inflow is far higher than designed for. The brewery giant Cambrew Ltd. discharges to the WWTP, utilizing around half of the capacity of the WWTP. Until recently a garment factory, RCI, also discharged into the WWTP, but they have recently gone out of business. CAMBREW has current plans to extend its soft drink production and its primary treatment from 2,500m^3/d to 5,000m^3/d.

The areas not yet included in the wastewater facility discharge their wastewater to the urban drainage system which feeds to the sea either directly or indirectly through previously natural watercourses. Approximately 751m^3/d of wastewater is currently (2016) generated by the Occheuteal, Serendipity and Otres beach areas, and 7,266m^3/d total from the wider urban area.

The WWTP is in good condition and working relatively well although does not always treat the wastewater to meet the national effluent discharge standards. The WWTP site is 800m x 140m. One of the four anaerobic ponds is full of sludge and out of service, whilst the other three are half-filled with sludge, due to lack of an appropriate mobile sludge pump. The facultative and maturation ponds are in good condition and working relatively well. The natural creek (Occheuteal Stream) nearby the treatment plant that WWTP effluent is discharged to is seriously contaminated due to the combined discharge from upstream residents who are not yet connected to the sewer.

[1] Table 4, Provincial Towns Improvement Project, Sewerage System in Sihanoukville, Design Report, ADB Feb 2002

Figure 1 - Coverage of WW collection under the 2007/08 Provincial Towns Improvement project.



2.2 Population

Population has been gradually increased with the average annual growth rate of 2.13% since 2009. The commercial facilities has seen to increase until last few years and remained almost constant afterward. The population statistics and commercial facilities are shown in Table 1 and Table 2, respectively.

Table 1 - Population in Preah Sihanouk city

Sangkhat Name	2009		2010		2011		2012		2013		2014	
	Family	Pop.	Family	Pop.	Family	Pop.	Family	Pop.	Family	Pop.	Family	Pop.
Bei	4134	20324	4226	20466	4250	20598	4400	21123	4377	20978	4403	21199
Buon	4919	22727	5236	24017	5252	23933	5312	24204	5353	24280	5427	24238
Kaoh Rung*	469	2089	493	2016	495	2127	530	2211	503	2180	560	2456
Muoy	3317	15479	3358	15667	3558	17394	3732	17327	4411	19796	4405	19843
Pir	1985	9702	2015	9883	2195	10262	2250	10361	2359	10423	2361	10514
Total	**14824**	**70321**	**15328**	**72049**	**15750**	**74314**	**16224**	**75226**	**17003**	**77657**	**17156**	**78250**

Source: CDB database, 2009-2014
*Note: * Settled on the island with about 22km from dry land*

Table 2 - Commercial facilities in Preah Sihanouk city

Sangkhat Name	2009					2010					2011					2012					2013				
	Market	Clinic	Hotel	Gesthou	Restaur	Market	Clinic	Hotel	Gesthou	Restaur	Market	Clinic	Hotel	Gesthou	Restaur	Market	Clinic	Hotel	Gesthou	Restaur	Market	Clinic	Hotel	Gesthou	Restaura
Bei	3	0	7	24	5	3	0	7	24	5	0	3	10	44	8	3	3	10	44	8	3	3	10	45	8
Buon	2	1	10	35	28	6	1	12	39	33	10	6	13	110	36	17	11	14	112	49	19	7	15	119	14
Kaoh Rung*	0	0	0	0	0	0	0	0	0	0	0	0	1	12	11	0	0	0	27	8	0	0	0	51	32
Muoy	3	0	0	0	2	3	0	0	0	2	4	2	0	0	1	4	1	0	8	0	3	1	0	8	0
Pir	1	10	5	15	4	1	8	4	16	2	7	5	8	25	19	6	5	9	25	12	3	8	5	25	14
Total	**9**	**11**	**22**	**74**	**39**	**13**	**9**	**23**	**79**	**42**	**21**	**16**	**32**	**191**	**75**	**30**	**20**	**33**	**216**	**77**	**28**	**19**	**30**	**248**	**68**

Source: CDB database, 2009-2013
*Note: * Settled on the island with about 22km from dry land*

The populations of the selected service area and projections are provided under Sections 4 and 5.

2.3 Industry

There are currently 50 industries and 223 handicraft businesses in Sihanoukville province, of which 11 industries are located in Sihanoukville town. Eight out of the eleven industries are located in Sangkhat 1 around National Road 4 and other three industries are located in Sangkhat 3 and Sangkhat 4. Currently, only one industry is discharging wastewater to the WWTP, Cambrew which discharges pre-treated wastewater. The list of industries along NR4 is shown in Table 3.

Table 3 - Industries along NR4 in Sihanoukville

No.	Names	Country Origin	Product	Location
1	New Star Shoes Co.,Ltd	Taiwan	Shoes	Village1, Sangkat1,
2	Leader's Industrial (former You Xin)	Taiwan	Garment/ knitting /printing	Village1, Sangkat1,
3	Cambrew (Angkor Beer)	Malaysia	Beer and soft drink	Village3, Sangkat1,
4	Royal Crowntex International	American	Cloth/ paper tissue	Village3, Sangkat1,
5	Lin's Textiles Co,Ltd	Taiwan	Garment	Village3, Sangkat1,
6	Sun Wah Fishing Co.,Ltd	England	Frozen	Village3, Sangkat1,
7	W.E.G.C Manufacturing Ltd/ Keep Top Sporting Gold	France	Sport stuff	Sangkat1,
8	CROWN Beverage Cans Sihanoukville Ltd	Belgium	Beverage can production	Sangkat1,
9	Springdale International Textile	Philippine	Garment	Village3, Sangkat3,
10	K-Som Textile	Cambodia	Garment	Sangkat 4,
11	P.Y International	American	Garment	Sangkat4,

Source: Provincial Department of Industry and Handicraft, 2015.

2.4 Effluent discharge standard

The Ministry of Environment, through its Pollution Control Department (PCD), sets effluent discharge standards in Cambodia. The PCD obtained approval for a comprehensive set of wastewater discharge standards in April 1999 under the SubDecree on Water Pollution Control.

Table 4 summarizes the effluent standards for effluent discharge into a public water area.

Table 4 - Effluent standards in Cambodia

Test	Unit	Discharge to Protected public water area (Std1)	Discharge to public water area and sewer (Std2)
pH		6-9	5-9
BOD	Mg/l	<30	<80
COD	Mg/l	<50	<100
TSS	Mg/l	<60	<120
TDS	Mg/l	<1000	<2000
Grease & oil	Mg/l	<5	<15
Detergents	Mg/l	<5	<15
Nitrate	Mg/l	<5	<20
Phosphate	Mg/l	<3	<6
Ammonia	Mg/l	<5	<7

Source: Sub-Decree on Water Pollution Control (Council of Ministers No. ANRK.BK -06 April 1999), Annex No 2

2.5 Existing WWTP

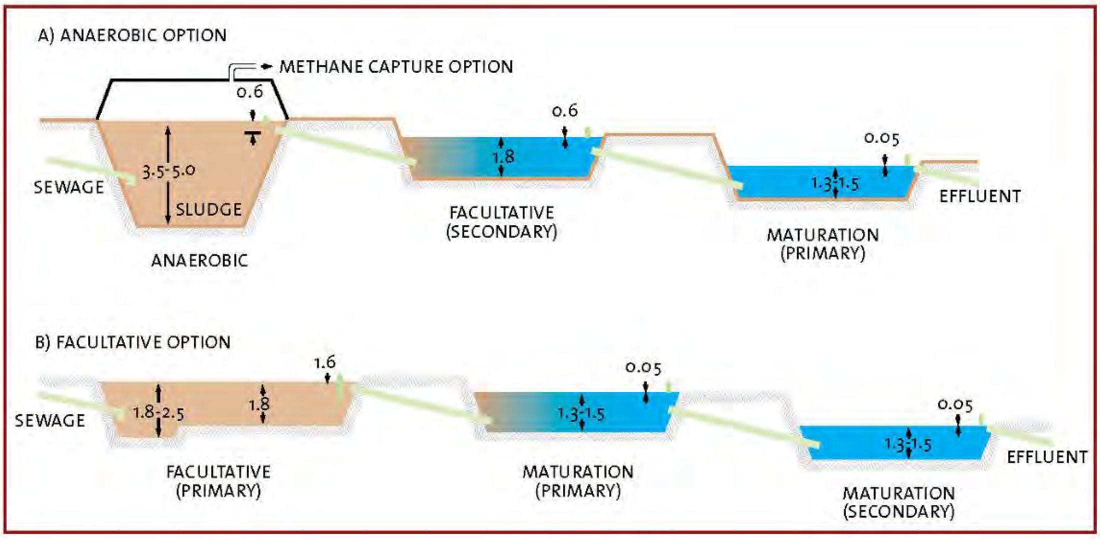
Waste stabilisation ponds (WSP) or lagoons are currently used in Sihanoukville.

WSP are shallow man-made basins into which wastewater flows and from which, after a retention time of several days (rather than several hours in conventional treatment process), a well-treated effluent is discharged. WSP systems comprise a series of ponds including anaerobic, facultative, and several maturation ponds. The advantage of WSP systems are simplicity, low cost, and high efficiency. If a suitable amount of cheap land is available, it is generally acknowledged that they should always be the first choice of technology in developing countries.

Anaerobic:	A pond (normally at least 3-5m deep) where sewage is digested anaerobically (in the absence of oxygen).
Facultative:	A pond (normally 1.5m to 2.5m deep) where both anaerobic and aerobic digestion of sewage takes place
Maturation:	A pond (normally 0.9-1.5m deep) primarily responsible for pathogen removal by various mechanisms, including UV disinfection and daily high pH levels.

Figure 2 shows a schematic of the WSP process, which is in common use around the world. Waste stabilization ponds (WSP) with a series of anaerobic, facultative, and maturation ponds were selected as the best option, providing good levels of treatment with the lowest possible operating cost for Sihanoukville.

Figure 2 - Schematic of Waste Stabilization Ponds



(source: Waste Stabilization Ponds, design manual, 19 Dec. 2011, by J.Ashworth; M.Skinner)

The existing lagoon dimensions are shown in Table 5.

Table 5 - Current lagoon dimensions:

Lagoon type	Qty	L(m)	W(m)	H(m)
Anaerobic	4	61.5	32.5	5.5
Facultative	2	165.75	62.75	2.75
Maturation	2	445.5	62.5	2.5

3 Summary of Current Situation and Proposed Subproject Activities

Table 6 summarises the current situation, shortfalls in service, and the solutions to these shortfalls offered under the proposed subproject.

Table 6 - Current and Proposed situation for sanitation in Sihanoukville

Current (1998 project)	Shortfall	Proposed
Main city centre area served	Beach tourism areas and rest of city not served. Areas that could be connected without pumping only were considered.	4 main beach areas plus main urban centre to north west of existing service area to be connected
Only 1,767 of 3,368 (52%) properties proposed under first project have been connected to date	Free connections were not included under first project	Free connections for remainder of 3,368 properties under first project service area, plus growth to 2020 for this service area (1,707 properties), plus 5 new service areas. Total new connections 8,295.
Trunk sewers sized for city centre only	Existing trunk sewers cannot be used for proposed extended service area	New trunk sewers proposed in parallel with existing ones to maintain service during construction
No pump stations. The part of the city centre forming the original service area was defined by the areas that could be served by gravity flow.	Large tracts of the city not connected due to pumping requirements.	Three new pump stations in the city and one each on Occheuteal and Otres beaches.
WWTP limited to 6,900m3/d which was intended for 5,900m3/d to serve 3,368 households plus a 1,000m3/d allowance for effluent from Cambrew.	In reality approx. 3,000m3/d used by 1,767 households and the remaining 3,900m3/d (or more) from Cambrew.	Upgrade of WWTP by mixing/aeration or other methods can increase capacity. Aeration alone can increase capacity up to 250% to meet 2040 demands for proposed new service areas. Trickling filters, oxidation ditches and UASB have also been considered.
Current WWTP land was intended for expansion by 2020 under original design	Land no longer available	WWTP expansion is limited to current site.
Beaches were intended to each have their own localised WWTP under original design	Land no longer available	WWTP expansion is limited to current site.
1 anaerobic pond full of sludge and other 3 half full	No portable sludge pump, no willingness to desludge manually, limited access into ponds.	Purchase of suitable portable sludge pumps with long intakes. Desludging under proposed project with improved access and equipment supply (bobcat)
No designed sludge drying bays. Treated sludge currently dumped at side of lagoons as very limited space.	No proper sludge drying. No sludge disposal or re-use.	Supply of simple containerised sludge dewatering. Disposal to landfill.
Septic tank septage disposed of to anaerobic ponds in adhoc manner	Operation not controlled and not clean	Each anaerobic pond to have septage disposal bay with concrete apron and service water for cleaning.
1 DPWT 6m3 septage truck plus one 4m3 private truck	Limited number of trucks for relatively large population and number of hotels	Provide one 6m3 further septage vacuum truck for DPWT
Limited capacity in WW treatment & operation	Small number of expert staff	Capacity building for all DPWT staff
Low level of public awareness on wastewater	Septic tanks not maintained, some leaking, too small etc	Public awareness campaign through various media.

4 Proposed Coverage Area

4.1 Priority Catchments

The Proposed new service areas ("Blocks"), confirmed at the Inception Workshop, are:

- Ocheuteal beach (B1)
- Otres beach (B2)
- Victory Beach and Hill (B3)
- Kanpenh/Kouch Asia/Borey Kampor St/part of Ekkareach Rd/north of Ekkareach Rd (B4)
- Independence beach (B5)

These 5 new service areas, along with the original service area from the project commissioned 2008, make up all of the main developed area of Sihanoukville. Whilst there are a few pockets of developed land outside of these areas they are small, and some new housing developments remain empty several years after completion. These few pockets of investor development aside, population is sparse outside of the total 6 service areas to be served.

Hawaii beach was originally included but was subsequently dropped due to the very low population there. As shown on Figure 3 next page, all five of these new service areas will be pumped to the WWTP, increasing the flow from 6,900m3/d to 20,500m3/d by 2040 which includes 4,000m3/d for industry. Any industrial expansion beyond this wastewater allowance will need to be treated to national standards privately. The current 1,757 connections will rise to 9,589 by 2040. The wastewater collection network is to be fully separated, and free domestic connections are to be provided both for the new service areas and the remainder of the original service area from the 2008 project. This will enable the DPWT to quickly generate revenue and will enable a tangible improvement in the environmental conditions in the beach areas and Kouch Asia areas to be seen. It was initially suggested by DPWT to construct a series of smaller WWTP's at each proposed new service area, however there is no land available, and regardless this approach would involve multiple effluent outfalls in popular beach areas.

Up to 4,000m3/d of pre-treated wastewater from CAMBREW may also be treated in the WWTP depending on the results achieved in using mixers. An increase in capacity of 2.5 times will enable this volume to be treated up to 2040. If a capacity of less than 20,500m3/d is achieved with mixers, and all service areas are fully connected, then peak WWTP capacity will be reached between 2035 - 2040 unless CAMBREW provides treatment to National effluent standards by that time.

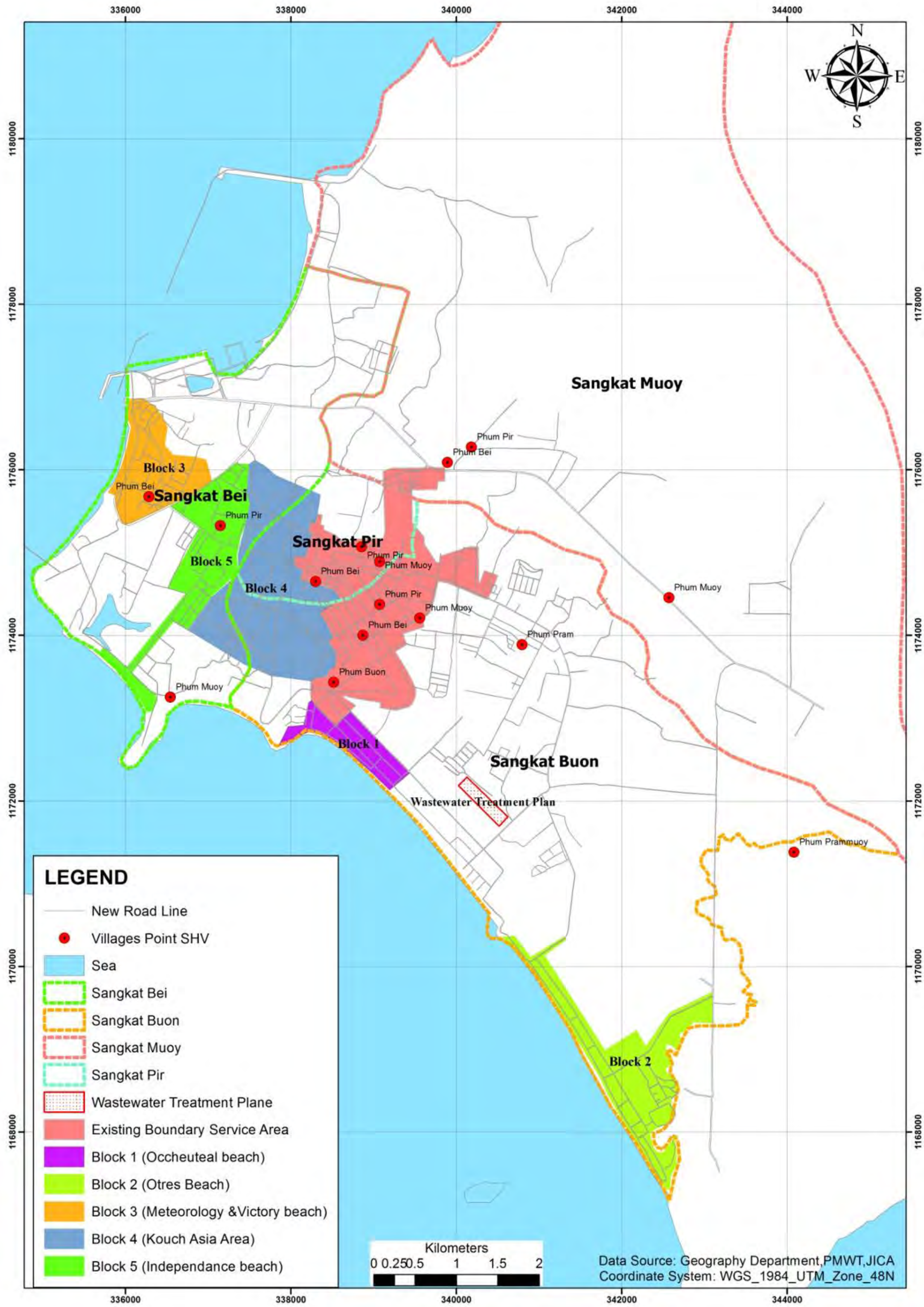
After the field visit and discussion with relevant stakeholders, block 6 (Hawaii beach) was dropped from this project as due to the fact that only few houses resided along the beach and one private apartment/houses development. Commune data and location for proposed new service areas are shown in Table 7 and Figure 3 below.

Table 7 - Commune data for each new service areas

Block	Commune data		Land area coverage by proposed extensions		
	Name	Area (ha)	Area (ha)	Cover (%)	Total coverage area by each block (ha)
B-1 Occheuteal	Sangkat Buon	2,445	56.8	2.32	56.8
B-2 Otres	Sangkat Buon	2,445	164.8	6.74	164.8
B-3 Victory Hill	Sangkat Bei	1,108	99.1	8.94	99.1
B-4 Kuch Asia	Sangkat Pir	237	81.7	34.47	262.0
	Sangkat Bei	1,108	74.9	6.76	
	Sangkat Buon	2,445	105.4	4.31	
B-5 Independence	Sangkat Bei	1,108	129.4	11.68	129.4

There is no further land available to expand the current WWTP and, consequently, new technology must be introduced if the capacity is to be increased. Whilst there are several viable methods to achieve an increase in capacity, it is important to utilize appropriate technology that will not become a financial or O&M burden on the DPWT in future. Technology options are presented in Section 6. However, generally speaking, the lesser the deviation from the current simple, familiar lagoon based system to achieve the increased capacity required, the more appropriate the solution will be.

Figure 3 - Proposed new service areas



4.2 Wastewater Generation Calculations

The design of the sewerage system is intended to collect all generated wastewater from the five blocks plus the original existing service area, from both domestic and commercial facilities. The design flow capacity is calculated for 20 years from project completion.

4.2.1 Domestic population served and wastewater production

The population data for the study area by sangkhat was collected for 2016, and used to estimate population distribution for each block and for population projections. The annual population growth rate used is based on the average growth rate from 2009 to 2013.

Table 8 shows current population by sangkhat and calculated current population by proposed new service block. The average population per household used throughout this study is 4.47.

Table 8 - Population in each proposed block

Block	Proposed new service Area	Sangkat	Population data 2016(1)		Coverage area (ha)			Population	
			Total Family	Total People	Total area	Coverage area	(%)	Family	Person
B-1	Occheteul	Buon	5,455	25,067	2445	56.8	2.32	254	1,137
B-2	Otres	Buon	5,455	25,067	2445	164.8	6.74	186	833
B-3	Victory	Bei	4,272	18,922	1108	99.1	8.94	991	4,430
B-4	Kuch Asia	Pir	2,474	10,467	237	81.7	34.47	718	3,208.7
		Bei	4,272	18,922	1108	74.9	6.76	1,196	5,347.7
		Buon	5,455	25,067	2445	105.4	4.31	516	2,306.0
B-5	Independence	Bei	4,272	18,922	1108	129.4	11.68	911	4,071
Total:			**31,655**	**142,434**	**10,896**	**712**	**6.54**	**4,773**	**21,333**

(1) Obtained from Sangkhat administration, June 2016

Table 9 - Population projections by proposed block

Block	Annual growth rate (%)	Present population	Projected Population				
		2016	2020	2025	2030	2035	2040
B-1	1.32	1,137	1,199	1,280	1,367	1,459	1,558
B-2	1.32	833	877	937	1,000	1,068	1,140
B-3	0.85	4,430	4,583	4,781	4,987	5,203	5,428
B-4	1.27	10,862	11,423	12,165	12,956	13,797	14,693
B-5	0.85	4,071	4,211	4,393	4,583	4,781	4,988
Total	**-**	**21,333**	**22,293**	**23,556**	**24,893**	**26,308**	**27,807**

As the subproject proposes to increase the capacity of the existing WWTP to treat the flows from the expanded service area, the total served population from both proposed new service areas and existing service areas must be taken into account. The existing service area covers three part-sangkhats with a total area of 321ha. The distribution of population and service area over the three part-sangkhats are presented in Table 10, and the projected population is also shown in Table 11.

Table 10 - Served population in the existing service area

Sangkat	Population data 2016(1)		Coverage area			Served Population	
	Total Family	Total People	Total area (ha)	Coverage area (ha)	(%)	Family	Person
Sangkat Mouy	4,411	19,855	4554	18.8	0.41	133	593
Sangkat Pir	2,474	10,467	237	91.5	38.59	1,624	7,258
Sangkat Buon	5,455	25,067	2445	210.6	8.62	3,018	13,490
Total	**12,340**	**55,389**	**7,236**	**321.0**	**4.44**	**4,774**	**21,341**

It can be seen that, based on applied growth rates, the population of the original existing service area under the 2008 project (design 2002) has increased from 3,368 housesholds to 4,774 households.

Table 11 - Projected population served in the existing service area

Sangkat	Growth rate (%)	Present served population (baseline)	Projected Population				
		2016	2020	2025	2030	2035	2040
Sangkat Mouy	5.27	593	728	941	1,216	1,572	2,033
Sangkat Pir	1.63	7,258	7,743	8,395	9,102	9,868	10,699
Sangkat Buon	1.32	13,490	14,217	15,180	16,209	17,307	18,480
Total	**-**	**21,341**	**22,688**	**24,516**	**26,527**	**28,748**	**31,213**

Based on the current water supply data, the daily water consumption per capita is about 150 liters. The water consumption rate is assumed to gradually increase from 150 lpc in 2016 to 180 lpc in 2030 to reflect increase in the urban standard of living, and it is then projected to level out and remain constant from 2030 to 2040. A wastewater return rate of 80% of the total daily water consumption has been used for the calculation of domestic wastewater generation. The commissioning of the sanitation subproject component is expected by the end of year 2020, and the design flow capacity for the purpose of sizing WWTP and trunk sewers is 20 years (2040). The projected WW generation in each proposed new service block by target year is shown in Table 12.

Table 12 - Domestic WW generation in both new and existing service areas 2016-2040

Service Areas/Block	2016	2020	2025	2030	2035	2040
New Service Areas						
B-1: Occheuteal Beach and serendipity						
Served population	1,137	1,199	1,280	1,367	1,459	1,558
Water consumption rate lpc	150	160	170	180	180	180
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	***136***	***153***	***174***	***197***	***210***	***224***
B-2: Otres Beach						
Served population	833	877	937	1,000	1,068	1,140
Water consumption rate lpc	150	160	170	180	180	180
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	***100***	***112***	***127***	***144***	***154***	***164***
B-3: Meteorology and Victory Beach Areas						
Served population	4,430	4,583	4,781	4,987	5,203	5,428
Water consumption rate lpc	150	160	170	180	180	180
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	***532***	***587***	***650***	***718***	***749***	***782***
B-4: Kampenh Village, Kouch Asia Area, Borey Kamkor and along Ekkareach road						
Served population	10,862	11,423	12,165	12,956	13,797	14,693
Water consumption rate lpc	150	160	170	180	180	180
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	***1,303***	***1,462***	***1,654***	***1,866***	***1,987***	***2,116***
B-5: Independence Beach						
Served population	4,071	4,211	4,393	4,583	4,781	4,988
Water consumption rate lpc	150	160	170	180	180	180
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m3/day)	***488***	***539***	***597***	***660***	***688***	***718***
Total New Service Areas (m3/day)	***2,560***	***2,853***	***3,204***	***3,585***	***3,788***	***4,004***
Existing Service Areas						
Served population	21,341	22,688	24,516	26,527	28,748	31,213
Water consumption rate lpc	150	160	170	180	180	180
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Total Existing Service Areas(m^3/day)	**2,561**	**2,904**	**3,334**	**3,820**	**4,140**	**4,495**
Grand Total	***5,121***	***5,758***	***6,538***	***7,405***	***7,928***	***8,499***

4.2.2 Served commercial/tourism facilities and demands

The five proposed new blocks cover a wide range of areas across Sihanoukville including sloped upland areas and beaches. There are a lot of commercial facilities located in these areas which have been categorised for the calculations as market, restaurant, hotel, guesthouse, and health clinic.

Commercial data[2] is available from 2009 to 2013 and has been used to estimate future growth of the facilities. The annual growth rate is based on the average growth rate from 2009 to 2013. However, the growth rate of market and health clinics was assumed to be zero as the total number of these facilities has fluctuated with more or less the same number over the last five years. The 2013 data has been used as baseline data and projected until 2040. The number of facilities in each category is presented in Table 13 and the growth rate and projected numbers of facilities are shown in Table 14 and Table 15 respectively.

Table 13 - Commercial facilities in each block in 2016

Block	Market-large	Market-medium	Market-small	Health clinic	Hotel	Guesthouse	Restaurant
New service areas							
B-1 (Occheuteal)	5	0	0	1	8	17	6
B-2 (Otres)	0	0	0	0	2	10	1
B-3 (Victory)	0	0	2	0	5	15	3
B-4 (Kuch Asia)	1	2	1	5	4	27	5
B-5 (Independence)	0	0	0	1	1	9	1
Existing service areas							
Existing	6	4	1	8	5	68	20
Total	**12**	**6**	**4**	**15**	**26**	**147**	**36**

Table 14 - Annual growth rate of each category based on data 2009-2013

Categories	Growth rate (%)
Market-Large	0
Market-Medium	0
Market-Small	0
Health Clinic	0
Hotel	9.17
Guesthouse	4.91
Restaurant	5.01

[2] Approved data from the Municipality

Table 15 - Projected commercial facilities in each block by 2040

Block	2016	2020	2025	2030	2035	2040
New service areas						
B-1	37	45	59	78	105	144
B-2	14	17	23	30	40	54
B-3	26	32	42	56	76	103
B-4	44	52	66	83	107	140
B-5	13	15	20	26	34	45
Existing service areas						
Existing	112	133	166	209	266	341
Total	**245**	**295**	**375**	**482**	**628**	**827**

The water consumption unit and rate of each commercial facility are estimated based on reported figures and data from direct interview from local authorities. These are presented in Table 16. A wastewater return rate of 80% of total water consumption has been used for the estimation of wastewater generation from commercial facilities. The results of commercial wastewater generation are shown in Table 17.

Table 16 - Wastewater generation rate and number of unit in each categories

Categories	Unit	Quantity	Generation Rate (L/unit.day)
Market-Large	person	1000	44
Market-Medium	stall	300	57
Market-Small	stall	150	57
Health Clinic	bed	10	833
Hotel	room	50	416
Guesthouse	room	30	144
Restaurant	seat	50	76

Table 17 - Commercial WW generation in both new and existing service areas 2016-2040

Service Areas/Block	2016	2020	2025	2030	2035	2040
New Service Areas						
B-1: Occheuteal Beach						
No. of commercial facilities	37	45	59	78	105	144
Water consumption amount (m3/day)	485	574	733	971	25,567	1,868
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m^3/day)	**388**	**459**	**586**	**777**	**20,454**	**1,494**
B-2: Otres Beach						
No. of commercial facilities	14	17	23	30	40	54
Water consumption amount (m3/day)	95	143	203	291	422	616
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m^3/day)	**76**	**114**	**162**	**233**	**337**	**493**
B-3: Meteorology and Victory Beach Areas						
No. of commercial facilities	26	32	42	56	76	103
Water consumption amount (m3/day)	203	265	376	540	785	1154
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m^3/day)	**163**	**212**	**301**	**432**	**628**	**923**
B-4: Kampenh Village, Kouch Asia Area, Borey Kamkor and along Ekkareach road						
No. of commercial facilities	44	52	66	83	107	140
Water consumption amount (m3/day)	341	292	510	664	887	1,214
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m^3/day)	**273**	**234**	**408**	**531**	**710**	**971**
B-5: Independence Beach						
No. of commercial facilities	13	15	20	26	34	45
Water consumption amount (m^3/day)	80	100	136	187	262	371
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Sub-Total (m^3/day)	**64**	**80**	**109**	**150**	**210**	**297**
Total New Service Areas	**963**	**1,100**	**1,566**	**2,123**	**22,339**	**4,178**
Existing Service Areas						
No. of commercial facilities	112	133	166	209	266	341
Water consumption amount (m^3/day)	881	1005	1211	1497	1898	2464
Water return rate (80%)	0.8	0.8	0.8	0.8	0.8	0.8
Total Existing Service Areas	**705**	**804**	**969**	**1,198**	**1,518**	**1,971**
Grand Total	**1,668**	**1,904**	**2,535**	**3,321**	**23,857**	**6,149**

4.2.3 Served Industry

There are only a few garment factories located in the service area along NH#4, and their wastewater is not allowed to be discharged into the public system. Currently, only the Cambrew factory is discharging its wastewater into the existing WWTP and this will probably continue in the near future. However, the quantity of industrial water primarily from Cambrew needs to be capped. Under the original 2002 design, an allowance of 1000m^3/d from Cambrew was included primarily to provide early revenue for DPWT, but the WWTP site was not intended to be used to cater for industry indefinitely. The quantity of wastewater currently discharged from this factory is estimated to be up to around 4,000m^3/day and this has been taken as the capped limit under this preliminary design.

4.2.4 Summary of demands

The total WW generation are from both domestic and commercial, and the inflow and infiltration rate into the sewerage system as due to the groundwater and/or storm water. The infiltration rate is assumed to be 10% of the total WW generation. The summary of WW generation is shown in Table 18.

Table 18 - Total WW generation in both new and existing services areas 2016-2040

Service Areas/Block	2016	2020	2025	2030	2035	2040
New Service Areas						
B-1: Occheuteal Beach						
Domestic WW	136	153	174	197	210	224
Commercial WW	388	459	586	777	1,273	1,494
Sub-Total (m^3/day)	**525**	**613**	**760**	**973**	**1,483**	**1,718**
B-2: Otres Beach						
Domestic WW	100	112	127	144	154	164
Commercial WW	76	114	162	233	337	493
Sub-Total (m^3/day)	**176**	**226**	**290**	**377**	**491**	**657**
B-3: Meteorology and Victory Beach Areas						
Domestic WW	532	587	650	718	749	782
Commercial WW	163	212	301	432	628	923
Sub-Total (m^3/day)	**694**	**799**	**951**	**1,150**	**1,377**	**1,705**
B-4: Kampenh Village, Kouch Asia Area, Borey Kamkor and along Ekkareach road						
Domestic WW	1,303	1,462	1,654	1,866	1,987	2,116
Commercial WW	273	234	408	531	710	971
Sub-Total (m^3/day)	**1,576**	**1,696**	**2,063**	**2,397**	**2,697**	**3,087**
B-5: Independence Beach						
Domestic WW	488	539	597	660	688	718
Commercial WW	64	80	109	150	210	297
Sub-Total (m^3/day)	**552**	**619**	**706**	**810**	**898**	**1,015**
Total New Service Areas	**3,523**	**3,953**	**4,770**	**5,707**	**6,736**	**8,182**
Existing Service Areas						
Domestic WW	2,561	2,904	3,334	3,820	4,140	4,495
Commercial WW	705	804	969	1,198	1,518	1,971
Industrial WW (Cambrew)	4,000	4,000	4,000	4,000	4,000	4,000
Total Existing Service Areas(m^3/day)	**7,266**	**7,708**	**8,303**	**9,018**	**9,658**	**10,466**
Total (m^3/day)	**10,789**	**11,661**	**13,073**	**14,725**	**16,394**	**18,648**
Infiltration (10%)	**1,079**	**1,166**	**1,307**	**1,473**	**1,639**	**1,865**
Grand Total (m^3/day)	**11,868**	**12,827**	**14,381**	**16,198**	**18,033**	**20,513**

5 Proposed Trunk Sewers and Pump Stations

5.1 Pipe Material Selection

Under the design of the current WWTP (Design report February 2002) uPVC pipe up to and including 400mm diameter, and GRP for larger sizes were specified. The DPWT has requested that, for ease of maintaining assets, the same pipe types be utilized under this proposed subproject. Both pipe types have good resistance against the corrosive nature of wastewater, and good hydraulic flow characteristics. A summary of advantages and disadvantages of both pipe types is provided in Table 19.

Table 19 - Pipe material comparison for use as sewer pipe

Material	Advantage	Disadvantage
uPVC	Superior all round performance	Easily cracked (impact damage)
	Lightweight	UV degradation for lengths above ground
	Resistant to chemical attack	Difficult to locate underground
	Flexible – resistant to fracture & adaptable to earth movements	Leak location difficult
	Watertight gasket joints	Not suitable for large diameters
	Smooth wall surfaces – better flow characteristics and less slime build-up	
	Predominant material for sewer pipes	
	Relatively low installed cost	
	Sufficient longitudinal rigidity to allow easy laying to grade without ponding	
	Long service life expectation (>100 years with no deterioration in bulk properties)	
	Good structural performance of buried sewer pipes over time	
GRP	Long service life >50 years	Low mechanical strength
	High corrosion resistance	Difficult to locate underground
	Low weight	Leak location difficult
	Simple push-on couplings	
	High stiffness available	
	High abrasion resistance	
	High tensile strength	
	Resistant to scaling	
	Flexible	
	Available in longer lengths (less joints)	
	Better hydraulic performance than DI & RC	
	Large diameters available	

The main reason for selecting GRP for larger diameters in addition to the reason given above, is that rigid uPVC is not generally available for larger diameters (most suppliers provide up to 24" or 600mm, and above 300mm is not common) and at these sizes it is a special order and expensive (eg. $360/m for schedule 40 and $540/m for schedule 80). GRP pipe can be supplied fit for purpose to much larger diameters.

5.2 Trunk Sewers

Preah Sihanouk province is a highland zone, and the topography of the land elevation fluctuates along the main road and secondary roads in the city. Topographic survey and field observation was undertaken and the proposed new trunk sewer locations and flow directions identified. The detailed topographical survey data is presented in Appendix A. The design flow directions of the trunk sewers are shown by red arrows in Figure 4.

Figure 4 - Main trunk sewer flow directions

Based on the identified flow direction, the main trunk sewer size has been estimated using the Manning formula. In this subproject, two types of pipes are proposed; uPVC and GRP pipes. This maintains consistency with the existing sewer system. The pipe with diameter of lower than 400mm will be uPVC and the pipe greater than 400mm diameter will be GRP.
Manning formula is used in this calculation with the assumption that the depth of water flow inside the pipe is 90% of the pipe diameter and the slope is constant with 0.15%. The manning formula is:

$$Q = \frac{S}{n} R^{2/3} i^{1/2}$$

where:

- Q : Flow (m^3/s)
- S : Sectorial area of flow (m2)
- n : friction coefficient (-)
- R : Hydraulic radius (m)
- i : slope (m/m)

The total wastewater generation in 2040 has been used for the pipe size calculation. The trunk sewer diameter has been calculated to be in the range from 250mm to 900mm. The total length of the trunk sewers is 32,102m. The detail of the pipe size, flow velocity, and length of the trunk sewer of each block is shown in Table 20.

Table 20 - (5 parts) - Trunk size, length, velocity inside the pipe in each block

Service Areas/Block	Flow (m3/day)	Length (m)	Slope (%)	Velocity (m/s)	Final Diameter (mm)
New Service Areas					
B-1: Occheuteal Beach					
B1TS-01	189	287	0.15	0.777	300
B1TS-02	189	331	0.15	0.777	300
B1TS-03	189	322	0.15	0.777	300
B1TS-04	567	1017	0.15	1.022	450
B1TS-05	567	1005	0.15	1.022	450
B1TS-06	567	146	0.15	1.022	450
B1TS-07	567	914	0.15	1.022	450

Service Areas/Block	Flow (m3/day)	Length (m)	Slope (%)	Velocity (m/s)	Final Diameter (mm)
New Service Areas					
B-2: Otres Beach					
B2TS-01	36	244	0.15	0.514	250
B2TS-02	145	1244	0.15	0.726	400
B2TS-03	145	1061	0.15	0.726	400
B2TS-04	145	652	0.15	0.726	400
B2TS-05	145	820	0.15	0.726	400
B2TS-06	145	870	0.15	0.726	400
B2TS-07	217	2246	0.15	0.804	500
B2TS-08	362	513	0.15	0.913	600
B2TS-09	506	817	0.15	0.994	700
B2TS-10	723	106	0.15	1.086	800

Service Areas/Block	Flow (m3/day)	Length (m)	Slope (%)	Velocity (m/s)	Final Diameter (mm)
New Service Areas					
B-3: Meteorology and Victory Beach Areas					
B3TS-01	375	336	0.15	0.922	400
B3TS-02	563	407	0.15	1.020	450
B3TS-03	563	300	0.15	1.020	450
B3TS-04	563	345	0.15	1.020	450
B3TS-05	563	339	0.15	1.020	450
B3TS-06	563	350	0.15	1.020	450
B3TS-07	563	450	0.15	1.020	450
B3TS-08	563	448	0.15	1.020	450
B3TS-09	1,313	274	0.15	1.261	700
B3TS-10	1,313	796	0.15	1.261	700
B3TS-11	1,313	855	0.15	1.261	700

Service Areas/Block	Flow (m3/day)	Length (m)	Slope (%)	Velocity (m/s)	Final Diameter (mm)
New Service Areas					
B-4: Kampenh Village, Kouch Asia Area, Borey Kamkor and along Ekkareach road					
B4TS-01	679	567	0.15	1.069	500
B4TS-02	679	407	0.15	1.069	500
B4TS-03	679	466	0.15	1.069	500
B4TS-04	1,019	1046	0.15	1.183	600
B4TS-05	1,019	650	0.15	1.183	600
B4TS-06	1,019	384	0.15	1.183	600
B4TS-07	1,019	261	0.15	1.183	600
B4TS-08	1,019	600	0.15	1.183	600
B4TS-09	2,377	558	0.15	1.463	800
B4TS-10	2,377	289	0.15	1.463	800
B4TS-11	2,377	232	0.15	1.463	800
B4TS-12	3,056	623	0.15	1.557	900
B4TS-13	3,396	942	0.15	1.599	900

Service Areas/Block	Flow (m3/day)	Length (m)	Slope (%)	Velocity (m/s)	Final Diameter (mm)
New Service Areas					
B-5: Independence Beach					
B5TS-01	335	357	0.15	0.896	400
B5TS-02	335	327	0.15	0.896	400
B5TS-03	558	822	0.15	1.018	450
B5TS-04	558	214	0.15	1.018	450
B5TS-05	558	793	0.15	1.018	450
B5TS-06	558	804	0.15	1.018	450
B5TS-07	1,116	1920	0.15	1.211	600

A new trunk sewer to deliver the wastewater from pumping stations from Block 03, 04 and 05 is required as the existing trunk from the Golden Lion roundabout to the WWTP (Ø900mm) is too small. The existing trunk sewer from the Golden Lion was only sized for the initial 2008 sub-catchment. The new trunk can be laid in parallel with the existing one (which will continue to carry flow), with diameter 1,200mm and length 2,161m.

The total length of pipe with diameter is shown in Table 21.

Table 21 - Total length of trunk pipe with different diameter

No.	Trunk Diameter	Length (m)
1	uPVC 250mm	244
2	uPVC 300mm	940
3	uPVC 400mm	5,667
4	GRP 450mm	7,347
5	GRP 500mm	3,686
6	GRP 600mm	3,868
7	GRP 700mm	2,742
8	GRP 800mm	1,777
9	GRP 900mm	3,670
10	GRP 1,200mm	2,161
Total		**32,102**

Trunk sewers have been designed for each proposed new area with wastewater from Victory Beach(B3), Kampenh Village/Kuch Asia(B4) and Independence Beach (B5) being delivered from the west to the beginning of a new 1200mm main trunk sewer starting at the Golden Lion roundabout, and Otres Beach(B2) and Occheuteal Beach(B1) being pumped separately from the east directly to the WWTP.

Wastewater from the Victory Hill sub-catchment(B3) is first pumped to the top of a hill, then gravity fed to second pump station at Independence Beach, which also collects wastewater by gravity from the Independence Beach sub-catchment. The combined flow from Victory Hill and Independence Beach sub-catchments is pumped from this second pump station to a third pump station, which also collects by gravity from the Kampenh Village/Kuch Asia. This third pump station then pumps wastewater from all three proposed new western sub-catchments to the Golden Lion roundabout, where it flows into a new 1200mm trunk sewer to the WWTP.

The trunk sewer network is illustrated in Figure 5, and trunk sewer route and size detail for each of the 5 sub-catchments are shown in Appendix B.

Figure 5 - Trunk sewer network overview

5.3 Pump Stations

5.3.1 Physical conditions

Due to the topography of Sihanoukville it is not possible to convey the wastewater by gravity from the five new proposed service areas to WWTP. Five pump stations are required for the five proposed new blocks. The pump stations are located at the lowest point where wastewater from each block can be collected by gravity. The capacity of each pump station is shown in Table 22, and location and route of pumping main is illustrated in Figure 7. Detailed pump calculation sheets and selection are included in Appendix F.

Table 22 - Pumping system physical data

Criteria/Block	Condition	B-1	B-2	B-3	B-4	B-5
Elevation (m)	Inlet (Invert)	+1.7	-2.0	+4.5	+2.5	+3.5
	Outlet (invert)	+4.5	+4.5	+45.0	+17.0	+17.0
	Highest point	+4.5	+12.1	+45.0	+19.9	+19.9
Flow (m3/day)	Year 2030	1,070	415	1,265	2,637	2,156
	Year 2040	1,890	723	1,876	3,396	2,992
Pumping main diameter (mm)	Year 2030	100	60	110	200	150
	Year 2040	150	80	150	200	200
Pumping main distance (m)	-	1,528	4,576	1,131	809	2,249

5.3.2 Pump selection

A submersible pump with auto-coupling is a type of pump which is easy to install and pull out if required during its maintenance. The auto-coupling system submersible pumps enables automatic connection or disconnection from its seat with the discharge elbow under the water to the outside of the sump pit.

When the pump is installed on an auto-coupling system where the base is fixed to the bottom of the pump pit, the pump is lowered into the pit on a dual guide rail system. The pump automatically connects to the base unit in a tilted position in order to evacuate possible air in the pump housing and to prevent clogging or jamming.

When the pump is installed on a hookup auto-coupling system, the base is fixed on a crossbar above the liquid level in the pit. The pump is lowered into the pit with the discharge pipe and the counter part of the coupling. The pump will be fixed in a tilted position when it is connected to the base. With both auto-coupling systems, the weight of the pump in combination with the sealing system will prevent leakage when the pump is operating.

The major advantage of a submersible pump is that, it never needs to be primed, because it is already submerged in the fluid. Submersible pumps are also very efficient because they do not have to expend a lot of energy moving water into the pump. Water pressure pushes the water into a submersible pump, thus saving energy. Furthermore, submersible pumps can easily handle solids, while some are better for liquids only. Submersible pumps are quiet, because they are under water, and cavitation is never an issue, because there is no “spike” in pressure as the water flows through the pump.

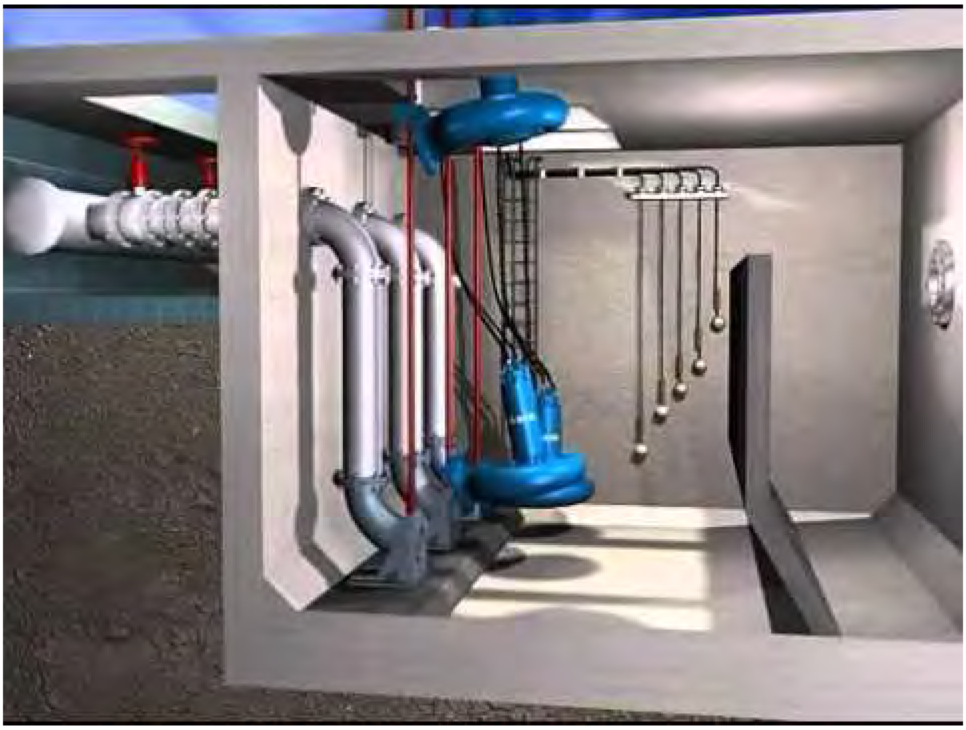
Pump selection has been made based on the requirements laid out in Table 22 and is presented in Table 23 below.

Table 23 - Pump selection by Block

Pump Location	Pump Type	Total capacity	Total head	Quantity		Pump Rating		Total Rating
		l/s	m	Duty	Stand BY	Hp	Kw	kW
B-1_2030	Submersible sump pump	13	5	1	1	1.5	1.1	1.1
B-1_2040	Submersible sump pump	22	6	1	1	3	2.2	2.2
							Total, kW	**3.3**
B-2_2030	Submersible sump pump	5	16	1	1	3	2.2	2.2
B-2_2040	Submersible sump pump	9	15	1	1	3	2.2	2.2
							Total, kW	4.4
B-3_2030	Submersible sump pump	15	43	1	1	30	22	22
B-3_2040	Submersible sump pump	22	46	1	1	50	37	37
							Total, kW	59
B-4_2030	Submersible sump pump	31	16	1	1	15	11	11
B-4_2040	Submersible sump pump	40	19	1	1	20	15	15
							Total, kW	26
B-5_2030	Submersible sump pump	25	20	1	1	15	11	11
B-4_2040	Submersible sump pump	35	19	1	1	20	15	15
							Total, kW	26

Figure 6 shows a typical pump station layout

Figure 6 - Sample set-up of proposed rectangular lift stations



5.3.3 Duty/standby

The pump system will occur during operation, wherein, one pump will operate when pumping is required while the other pump is on a Stand-by status. Upon reaching the low water level, the duty pump or pump in operation will stop and upon reaching High water level where pumping is required again, previously stand-by pump will now operate as duty pump. In this arrangement of operation, no pump is non-operational or stagnant for a long period of time as the two (2)- pumps, 1-Duty/1-Stand by pumps are operating alternatively. On the other hand, when the water level reached the High-High water level, the stand-by pump will operate to assist the duty pump in pumping water until it reaches the Low water level which will send a signal to the controller that both pumps will stop.

Above mode of operation will be utilized on the 2030 design period while a third pump will be added on the 2040 design period. Thus, a configuration of two (2)-Duty/ 1-Stand-by pump mode of operation will then be utilized. Again, said three (3) pumps will operate alternatively thus making no pumps non-operational or stagnant for a long period of time.

5.3.4 Variable speed drives

Most pump applications do not require full pump performance 24 hours a day. Therefore, it is an advantage to be able to adjust the pump's performance in the system automatically. The best possible way of adapting the performance of a centrifugal pump is by means of speed control of the pumps. Speed control of pumps is normally made by a frequency converter unit.

Variable Speed drives could be utilized in the wastewater lifting station especially with pump motor rating above 5.5kW. These pumps with big pump motor rating as mentioned above does not require full pump performance 24-hours a day, thus when pumping water level or when the water level at the sump pit doesn't change abruptly or remained constant at a longer period, a level sensor which gives information and signal to the motor control with variable speed control should operate at a reduced speed or frequency thus resulting a lesser current consumption and contributes to a lower power consumption.

5.3.5 O&M

Pump/Lifting stations should be regularly monitored and visited on a regular interval to see if it operates normally and no irregularities and abnormalities in its operation is happening. Removal of debris and sediments that cannot be pumped should be removed manually to avoid accumulation of the debris in the sump. Clogging of the pump system and its piping could also occur thus will result to down time in its operation. A by-pass pumping system utilizing an external pump should be integrated in the design of the discharge piping to enable emergency pumping especially during brown outs or non-availability of power thus also preventing overflow of the sump.
Table 24 shows power requirements for each of the pumps described

Table 24 - Electrical requirements

Pump Location	Incoming Capacity Power from EDC	Transformer capacity KVA
B-1_2030	1Pole /32A, 230VAC	N/A
B-1_2040	1Pole /32A, 230VAC	N/A
B-2_2030	1Pole /32A, 230VAC	N/A
B-2_2040	1Pole /32A, 230VAC	N/A
B-3_2030		50
B-3_2040		63
B-4_2030	3Pole /32A, 230VAC	N/A
B-4_2040	3Pole /40A, 230VAC	N/A
B-5_2030	3Pole /32A, 230VAC	N/A
B-4_2040	3Pole /40A, 230VAC	N/A

Figure 7 - Location of pumping station and route of pumping main

In B-05 block (Independence beach area), there are two alternative proposed locations for the pump station, one on the road at a low point where surface water drains into the north east corner of the natural lake Boeng Preak Tub, and the other at the beach. The first option is the topographical preference but it might be difficult to acquire the land for pump station construction[3]. The second option on public land at the beach is slightly higher but is much more straightforward in terms of land availability.

5.4 Collection pipelines

The main trunk sewers are described above. In addition, pipelines along every road in the five proposed service areas will be required, both sides of the road, in order to make domestic connections.

5.5 Household Connections

It is proposed to install free domestic and commercial connections to all existing properties in the five proposed new service areas (4,987 in 2020 from Table 9), plus all those properties from the first 2008 service area that have not yet been connected (5,075 in 2020 minus 1,767 already connected, from Table 11). The original 2002 design, commissioned in 2008, allowed for 3,368 connections in the existing service area by 2010. This will have risen to 5,075 by 2020. The total number of connections in 2020 is therefore 8,295. Commercial connections are additional to these figures and are shown in Tables 13-18.

[3] Based on DPWT report and site visit

6 Treatment Plant Capacity Increase

The current WWTP has a maximum capacity of 6,900m^3/d which has already been reached serving the 1,767[4] connections made to date (approximately 3,000m^3/day) plus wastewater from Cambrew. No further land is available so any retrofitting of the existing WWTP in order to increase capacity to the proposed 20,500m^3/day must be accommodated within the existing WWTP site boundary.

There are several alternative Options proposed for further development of the WWTP site, both to meet the 2040 demand of the existing and proposed 5 additional catchment areas, and to meet the possible future further system expansion to treat all sub-catchments in the wider Sihanoukville urban area.

6.1 Option 1: Increase capacity of WWTP by utilising mixers for aeration ("Aerated lagoons")

This option was the originally intended solution for upgrading process units under the 2002 Design Report[5]. It is also the most technically simple option for ease of operation and maintenance from a staff base of limited capacity and experience.

With the ambient temperatures experienced in Cambodia, aeration by mixing the upper layers of the lagoons has the potential to increase the capacity of the WWTP by 2-2.5 times, or up to 17,250m^3/day. Appropriate mixers should be of the floating surface variety as opposed to submersible so that the depth of mixing can be better controlled in the lagoons. In the facultative lagoons the objective would only to be to aerate the upper layers whilst leaving the deeper anaerobic layers undisturbed. Whilst many brands of mixer are widely available in China, Thailand and Vietnam, they are generally simple and durable, requiring little maintenance, and so source-country is not as important as more complex or high maintenance equipment.

Solar powered mixers have been requested by the DPWT. These have the obvious advantage of reducing power costs, taking advantage of the long hours of sunlight in Sihanoukville. A brand of solar mixers sourced from the US has been investigated and they provide several models, including a very shallow surface water mixer designed for odour reduction in the anaerobic lagoons, which provides the added advantage of removing surface scum and aiding methane release improving improves digestion of wastewater. A second mixer type designed for the facultative and maturation lagoons mixes water to a deeper level and this depth can be controlled and set.

The mixers are proposed to also be connected to mains power for those times of year when it may be overcast, or in case of any of the solar panels being out of service. The manufacturer has proposed the following configuration:

Table 25 - Mixer configuration

Pond type	Pond Qty	Mixer type	Mixer Qty (per pond)	Mixer Qty (total)
Anaerobic	4	Odour capping	1	4
Facultative	2	Aeration	2	4
Maturation	2	Aeration	1	4

Mixer data sheets are contained in Appendix A.

4 3,368 domestic connections were allowed for in the design of which 1,767 have been connected to September 2016. By 2020 this 3,368 will have increased to 5,075.

5 P.39, Provincial Towns Improvement Project, Sewerage System in Sihanoukville, Design Report, ADB Feb 2002

6.2 Option 2: Increase capacity of WWTP by utilising trickling filters

6.2.1 Trickling Filter Description

Trickling filters are designed primarily for BOD removal. They are basically a form of biological filter as opposed to a physical filter in that solids are not removed. Biofilm forms on media inside the filter, which convert pollutants into harmless compounds in the presence of air. Wastewater is sprinkled onto the surface of the media from a rotating arm after which it passes vertically through the filter media. Air enters from both underneath and the top and creates a mostly aerobic environment, except for the centre of the filter which can become anaerobic.

An underdrain system collects treated wastewater, which then usually requires further treatment and settlement in an oxidation pond prior to release into receiving waters. Wastewater needs to be settled in advance of feeding it into the trickling filter, which itself has a very low TSS removal. Filters will clog if solids are not removed upstream of the inlet. Trickling filters are a secondary treatment. The hydraulic and *nutrient* loading rate (i.e., capacity) is determined based on the characteristics of the *wastewater,* the type of filter media, the ambient *temperature*, and the discharge requirements.

The filter is usually above ground, and is usually a cylinder 1-3m deep filled with large surface area media, to which micro-organisms attach. In the past, stones were usually used for media. In more recent times high surface area plastic media have been developed which are much lighter than the traditional stone media and allow the filters to be higher and better ventilated, increasing capacity further. Using these specially designed plastic media can double the capacity when compared to using stone media in the same sized trickling filter.

Sloughing of the micro-organisms from the media into the effluent can occur which requires further settlement downstream of the filter. Sloughing can be managed by a skilled operator finding a balance between bio-film growth and amount of bio-film falling off the media into the effluent, by controlling the organic and hydraulic loads into the filter.

Pumps are required to feed the wastewater to the top of the filter unit, so a constant power supply is required. Whilst trickling filters require more power than a lagoon based system, they require significantly less power than recirculated sludge based plants and can be considered a relatively low-energy green technology. A standard low-rate trickling filter can remove 80-85% of BOD when operated correctly.

Table 26 - Advantages and disadvantages of trickling filters

Advantages	Disadvantages
Can be operated at a range of loading rates	High capital costs
Resistant to shock loadings	Requires expert design and construction
Efficient nitrification	Requires full time O&M staff for efficient operation
High BOD removal	Requires a constant flow of wastewater and source of power
Small land required compared to lagoon systems	Flies and odours are often a problem
	Pre-treatment and sludge treatment are required
	Risk of clogging
	Not all parts & materials available locally

Modern more complex “high rate” trickling filters can have greatly increased capacities compared to the standard configurations that have been used for many decades.

Filter performance can be greatly improved by a skilled professional operator. A constant flow is desired, and this can be achieved during times of varying wastewater inflow by recirculation of effluent to the sedimentation pond upstream of the filter or the filter itself. This also increases dissolved oxygen levels, ensure wetting of the media and helps control sloughing. Trickling filters with one or two stages of recirculation are sometimes known as “high rate” trickling filters. The dosing rate onto the media is also important and can be controlled by the sprinkler arm rotation speed and amount of dilution by recirculation. A good operator can maximise treatment capacity by monitoring incoming wastewater BOD and adjusting the trickling filter operation appropriately. A high rate trickling filter can typically remove 60-80% of BOD.

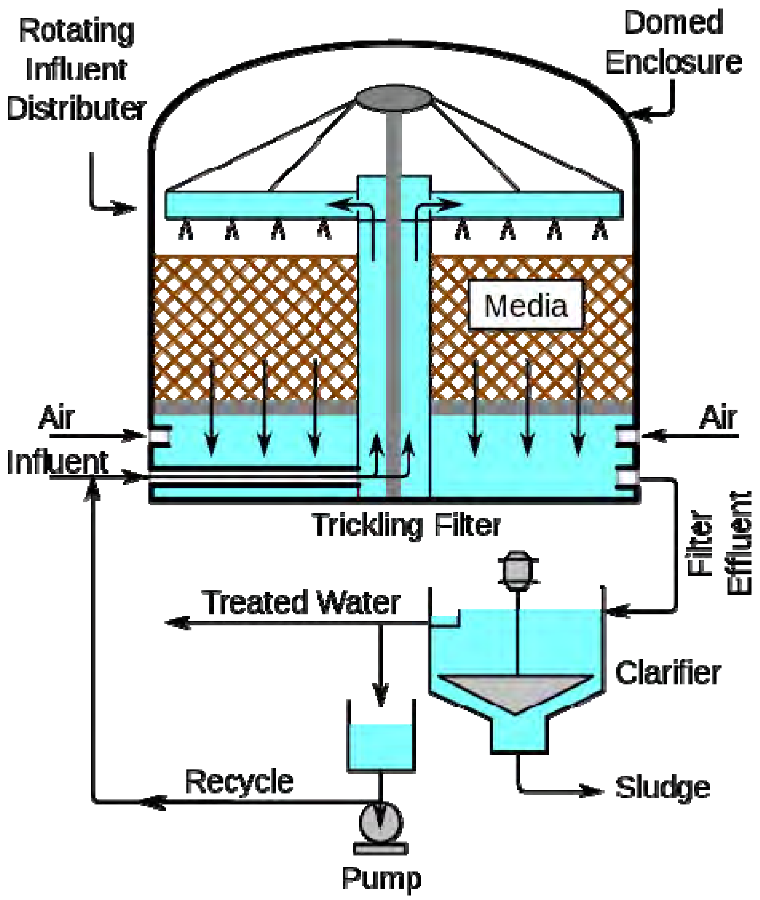
The advantages of recirculation are:

- It allows constant dosage regardless of fluctuation in sewage flow and thus keeps the bed working.
- It dilutes the influent with better quality water and, making it fresh and reducing odour.
- It maintains a uniform rate of organic and hydraulic loading.
- It provides longer contact of the applied sewage with the bacterial film on the contact media and accelerates the biological oxidation process.
- It increases the efficiency by reducing the BOD load generally.

A high rate trickling filter can have a capacity up to 10x that of a standard trickling filter and therefore require less land.

Figure 8 shows the basic features of a trickling filter.

Figure 8 - Trickling filter example



6.2.2 Trickling Filter Retrofitting

For the trickling filter option, it is proposed to retain the existing anaerobic ponds as the pre-filter settlement ponds which are an upstream requirement if trickling filters are to be employed. These can continue to be operated as now with periodical desludging, and will provide some pre-treatment in addition to settlement of solids. These were designed[6] to provide 1 day retention time for a flow of 7,590m^3/d. In order to retain 1 day retention time for settlement, these will require extending into part of the land saved from the facultative ponds. They may also be re-modelled as required into 2 larger ponds instead of the current 4.

The existing maturation ponds will also be retained to provide further settlement of solids, including solids from sloughing of the filter media, prior to release of effluent into the receiving waters.

The area utilised by the existing facultative lagoons can be fully or partially filled and compacted, and used for one or more trickling filters and an extension of the current anaerobic lagoons. It is likely that more than one filter will be required to meet the projected wastewater volumes expected. In order to maximise treatment capacity for a limited amount of land, it is recommended that if this option is selected that a modern plastic media is used. The land available if the facultative lagoons are decommissioned will be 165.75m long and 125m wide (2 x existing ponds of 165.75m x 62.75m).

[6] Appendix C.2.1 "Provincial Towns Improvement Project, Sewerage System in Sihanoukville Design Report", ADB, 2002.

In order to keep the WWTP operating during the construction of this option, the extension of the anaerobic lagoons and reclamation of the remainder of the facultative lagoons will be carried out in two stages, one facultative lagoon at a time. Once the proposed trickling filters are commissioned, the second facultative lagoon can be developed. This will necessarily reduce the capacity of the WWTP during the time of construction by half, and therefore an accelerated construction period, during the dry season, will be preferable.

Table 27 below illustrates some expected capacities from different trickling filter types and configurations for a filter of 2m media depth. Literature[7] has a conventional trickling filter capable of treating 1-4m^3/m^3 media/day and a high rate filter with 10-40m^3/m^3 media/day. For the purpose of this example calculation a median flow of 2.5m^3/m^3/d and 25m^3/m^3/d respectively have been used. The 2040 design flow of 16,000m^3/d plus 4,000m^3/day for Cambrew has been used.

Table 27 - Trickling filter configurations and capacities to design year 2040

Filter configuration	Expected flow range (m3 flow/m3 media/day)	Flow rate used in calculation	2040 Flow/filter (m3/d)	Filter diameter (m)
1 x conventional TF	1-4	2.5	21,000	73.1
2 x conventional TF	1-4	2.5	10,500	51.7
3 x conventional TF	1-4	2.5	7,000	42.2
4 x conventional TF	1-4	2.5	5,250	36.6
1 x high rate TF	10-40	25	21,000	23.1
2 x high rate TF	10-40	25	10,500	16.3

Trickling filters are generally not larger than 60m diameter as the rotating arm distributing influent must be supported by cables and the larger the diameter, the more challenging this becomes.

The construction of the trickling filters may also be staged, with immediate construction allowing for wastewater flows to 2030. Options for this scenario are shown below in Table 28.

Table 28 - Trickling filter configurations and capacities to design year 2030

Filter configuration	Expected flow range (m3 flow/m3 media/day)	Flow rate used in calculation	2030 Flow/filter (m3/d)	Filter diameter (m)
1 x conventional TF	1-4	2.5	16,800	46.2
2 x conventional TF	1-4	2.5	8,400	32.7
3 x conventional TF	1-4	2.5	5,600	26.7
4 x conventional TF	1-4	2.5	4,200	23.1
1 x high rate TF	10-40	25	16,800	14.6
2 x high rate TF	10-40	25	8,400	10.3

From Table 28 it can be seen that one option could be to construct one high rate trickling filter with recirculation and high surface area plastic media of 14.6m diameter (area approx. 200m^2) on the 20,800m^2 that could be made available from redeveloping the facultative lagoons. This could provide secondary treatment to design year 2030, after which a second filter could be constructed, should the technology prove itself to be appropriate for Sihanoukville staff capacity levels.

Should the more advanced high rate trickling filters be selected, periodical ongoing external operational support to DPWT staff is recommended over several years.

[7] *www.idc-online.com/technical_references/pdfs/civil.../**Trickling_Filter**.pdf*

6.3 Mixed process streams: One aerated lagoon stream, one conventional trickling filter stream.

In order to facilitate a seamless switch over between the current lagoon technology and the proposed longer term introduction and use of conventional trickling filters, it is possible to retrofit solar mixers into one lagoon stream as described under Section 6.1, and develop the other stream as described in Section 6.2.2. The lagoon stream, with mixers to increase current capacity, would continue to operate until it reached capacity, and at that time the trickling filters could be brought online one at a time.

The primary advantage of this configuration is that there would be no period of insufficient capacity, which would be experienced if DPWT waited until close to 2040 when the aerated lagoons approached capacity before starting the conversion over to trickling filters one stream at a time. During this construction period all of the wastewater from the town (approximately 20,500m^3/d) would be treated by one stream capable of treating around half of this flow.

The flows that can be treated by one of the two process streams, if aerated and based on a factor 2.5 increase over non-aerated lagoons, are shown in Table 29.

Table 29 - Treatment capability of one single aerated lagoon stream

Lagoon configuration	Design flow (m3/d)	Treatable flow-aerated (m3/d)
Full current lagoon	6,900	17,250
Half lagoon (one stream)	3,450	8,625

From Table 18, by the time the WWTP is commissioned in approximately 2020, the flows from the original service area plus the new 5 service areas will be 12,827m^3/d, more than the 8,625m^3/d that can be treated by one single lagoon stream. The advantage of this option is therefore lost as the process would need to go straight to trickling filters, negating the need to keep one lagoon stream running.

6.4 Option 3: Redevelop WWTP site using higher technology to allow for future treatment of all city subcatchments

It is highly unlikely that further land will be acquired for the purposes of wastewater treatment in the urban centre of Sihanoukville, given the rapid expansion over the last 15 years and rapidly increasing land prices. As such, consideration must be given to the long term wastewater treatment coverage requirements for the entire urban area. Whilst this subproject specifically addresses the 5 main populated catchments - both commercial and tourism related - there are other areas to the north and east of the WWTP and near the port which will grow in the future. Whilst sparsely populated now, they can be expected to become developed during and after the design period of this proposed subproject.

Potential options for further development of the current WWTP site to serve more catchment areas and potential pre-treated wastewater from future urban industry (particularly along NR4) are;

- Further high-rate trickling filters
- Oxidation ditch
- Upflow anaerobic sludge blanket (UASB)

6.4.1 Further high-rate trickling filters

As described in 6.2, further filters could be added in 2030-2040 to feed further the remaining few unserved urban catchments as they are developed in the future. Reclamation of the land where the facultative lagoons now sit would allow sufficient area for HR trickling filters to serve all of the

Sihanoukville urban area including the 5 subcatchments proposed under this subproject plus those further as-yet undeveloped areas. This option would be advantageous as:

- it would retain the technlology proposed for the 5 subcatchments under current design
- would allow for any operational difficulties experienced from the first phase of HR trickling filters from the current proposed 5 subcatchments to be addressed by amending design as required
- Allow for continuity in the DPWT learning curve in using tricklng filters
- HR trickling filters to serve the entire urban area can fit into the current WWTP site.

6.4.2 Oxidation ditch technology

An oxidation ditch is used for the low strength WW usually associated with municipal use. The wastewater is circulated around an oval “ditch” by horizontal aerators, and activated sludge is reintroduced into the process. Oxidation ditches require more land than other activated sludge processes, and more land than trickling filters but do not require settlement lagoons upstream as trickling filters do. They also have several stages of sludge pumps, blowers and other moving machinery which require maintenance and correct operation.

An oxidation ditch WWTP in Bac Giang, Vietnam, of capacity 10,000m^3/day utilises a site of 145m x 75m or 1.1ha. The current Sihanoukville WWTP site is 800m x 140m or 11.2ha so could easily accommodate two or more trains of oxidation ditch plants to accomodate future flows for the whole urban area.

6.4.3 UASB technology

The Upflow Anaerobic Sludge Blanket reactor is a single stage process where the wastewater enters the plant from below and flows upwards through a (normally) cylindrical body, passing through a suspended sludge blanket consisting of granules and formed gas bubbles, which acts as both a mechanical filter and provides biological treatment. Biogas is produced which can either be collected or released. The sludge blanket takes several months to form following start-up. Figure 9 below shows the basic components of a UASB reactor.

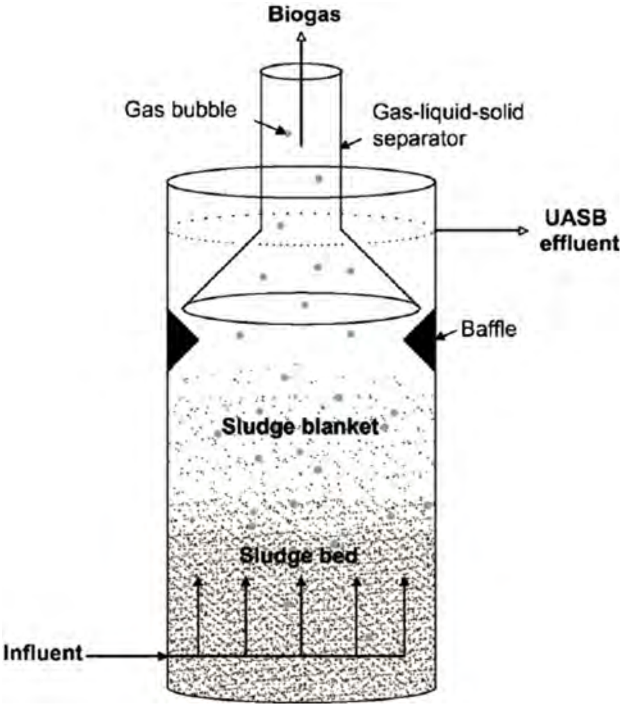
The effluent is nutrient-rich and should ideally be disposed of to agricultural lands rather than natural watercourses.

The UASB plant requires a high level of professional operator skill and understanding. Areas of operation particular to UASB plants are:

- They are more successful with high COD content influent – a minimum of 250ml/l and over 400mg/l for optimum performance. The combined WW influent (from Cambrew plus domestic) to the existing Sihanoukville WWTP was estimated as having a COD of 200mg/l, with the domestic component being only 158mg/l.
- The influent must be consistently distributed across the base of the structure to enable good formation of the blanket and even contact with it. This requires a good influent distribution system that must be monitored.
- The hydraulic load must be set to match the organic load. The upflow velocity must be set to balance the floating sludge blanket to keep it suspended without washing suspended solids out the top of the filter. The control of organic loads is by the operator using a feed pump to match the hydraulic loads to the fluctuating strength of the wastewater coming into the plant.
- A continuous and stable WW flow is required. The UASB has little capacity to handle shock loads.
- The process has a low pathogen reduction

Whilst the UASB technology requires less land than many options, the level of operator skill required, the high influent strength required for efficient anaerobic treatment, and the high levels of nutrients and pathogens in the effluent makes it unsuitable for this application.

Figure 9 - UASB reactor basic layout



6.5 Use of higher technology, energy saving equipment and processes.

There is a Japanese grant fund available (JFCJM) to offset any additional costs associated with purchasing higher technology or more advanced equipment that will have an energy saving or carbon footprint-reducing benefit over a more standard or basic technology selection.

Where possible, preliminary designs selected and recommended under this subproject should take advantage of this potential funding source.

For the above WWTP option of aerated lagoons, the most basic option to increase treatment capacity through mixing/aeration would be banks of submersible or beam-mounted mixers powered from the mains. These are widely available in SE Asia for a relatively low cost, and many are manufactured in China. Utilising higher technology, standalone solar powered floating mixers can be used, which whilst more expensive have no mains energy requirements, and are tailored to provide far more directional mixing specific to the requirements. Based on recent quotes, relative costs of package solar mixers and conventional mains powered mixers are shown in Table 30.

Table 30 - Relative costs of solar and conventional mixers

Mixer type	Location	Qty recommended	Unit cost ($)	Cost ($)	Total cost ($)
Solar[8]	Anaerobic pond	4	61,100	244,400	
	Facultative & maturation ponds	8	61,100	488,800	733,200
Conventional[9]	Anaerobic ponds	24	2,880	69,120	
	Facultative & maturation ponds	48	1,360	65,280	134,400

This example shows a potential JFCJM grant available of $598,800, the difference between the capital cost of conventional mixers and the more advanced, energy saving mixers.

For the option of using high-rate trickling filters, there are further opportunities to increase efficiency and treatment capacity with use of higher technology. Two stages of recirculation and proprietary high surface area media, media support and underdrains can all improve treatment efficiency. However, this level of technology is currently considered beyond the capacity of the DPWT.

Under all of the described WWTP technology options, there is a requirement for 5 pump stations to deliver wastewater to the WTWP from the different sub-catchments. Two key improvements can be made in energy saving at these pump stations.

Firstly, the correct selection of a high quality submersible wastewater pump taking into account the true duty point range from superimposing the duty and operating curves. Whilst a standard method of specifying pumps for many decades, it is often not done, with some engineers and contractors instead relying on a manufacturer's duty data only. Purchasing a more expensive but well known trusted international brand also has long term advantages in power efficiency over cheaper regional brands.

Secondly, the use of variable speed drives (also called variable frequency drives, adjustable speed drive or frequency inverters) can adjust the speed of the pump to suit a variable flow or head requirement, saving power over the more common on/off configuration associated with a fixed speed pump.

[8] Cost based on quote from Solarbee
[9] Cost based on Changzhou Rhongda industries

7 Septage Treatment

7.1 Septage collection

With only 52% of the central city area currently connected to the wastewater treatment plant (less than 15% of the total number of households in the city), the majority of urban properties are reliant on either a septic tank or concrete holding/soakaway tank (pour flush pit latrine).

Septage in Sihanoukville is currently collected by only 2 vacuum trucks, one of 6m^3 owned by DPWT and one privately owned truck of 4m^3. Neither are fully employed. Demand from householders to have their septic tanks de-sludged is based on when they block or cause a smell nuisance rather than by preventative maintenance or periodic checking of sludge depth.

There is no record of numbers, volumes or condition of septic tanks in the city. A septic tank survey and public awareness campaign are recommended and are described in detail in the "Sector Analysis, Development Roadmap and 20 Year Investment Roadmap for Urban Sanitation" part of the Draft Final Report.

7.2 Septage treatment

Septage in Sihanoukville, where collected, is treated in the anaerobic lagoons of the WWTP. There is no current constructed service bay for septage offloading, but instead it is emptied into the side of one of the four lagoons. No septage volume or truckload records are taken.

This co-treatment of septic sludge is an established method of primary treatment as the septic tank sludge mixes with the sludge from the reticulated wastewater from the service area. This increases the loading to the pond which is beneficial for anaerobic decomposition.

If the WWTP upgrade options of either mixing or provision of trickling filters are taken, co-treatment of septage can continue in the anaerobic ponds. However, if oxidation ditches or UASB plants were utilised then either separate new facilities would be required for septage treatment, or the existing anaerobic ponds would need to be modified into stand-alone septage treatment lagoons with 50% redundancy for drying of treated sludge. This is a further disadvantage of selecting the oxidation ditch or UASB options.

8 Septage Treatment

8.1 Current situation

There was no sludge drying area allowed for in the design of the current lagoon system. When sludge has been removed from the anaerobic lagoons it was stockpiled for drying on narrow grassed areas along the sides of the lagoons. However, as it is never disposed of after drying and before the onset of each wet season, this is actually an unplanned, unprotected storage area rather than a drying area.
The anaerobic lagoons were designed with concrete stair access to the bottom for manual desludging (which is feasible if sludge is not allowed to build up too much). An access road for a bobcat or similar was not included in the design due to the 1:3 slope of the anaerobic lagoons. The lagoons were intended to be de-sludged when half full of sludge, by allowing them to be dewatered and partly dried over a period of weeks during the dry season. An alternative method of de-sludging, not intended under this project design, is to decant much of the lagoon to be de-sludged, mix remaining clarified liquid with the settled sludge to a thick liquid consistency, and pump it to trucks at the lagoon edge. Floating barges on the lagoon are usually hired for this purpose, but this service is not available in Cambodia.
Operating staff have not been active in desludging over the last 8 years and one anaerobic lagoon is now out of service due to being full of sludge. The remaining three are reported as 50% full.

8.2 Sludge removal from anaerobic lagoons

Operating staff will not remove sludge manually. This is due to lack of staff, funds to hire temporary labour, equipment and priorities. DPWT have requested portable submersible sludge pumps to aid in desludging but as each of the four lagoons are 61.5m x 32.5m it is unlikely that pumps alone would enable consistent desludging across the footprint of each lagoon without a barge or floating pontoon on which to mount the pumps
It is instead recommended that full desludging be included in the forthcoming construction contract, and following that, concrete access tracks to allow a compact excavator ("bobcat") be constructed into each lagoon.

8.3 Drying options

The degree of "drying" for sludge can be defined as:

- Ordinary sludge – from unsettled lagoon - 2% solids
- Thickened sludge - from settled lagoon – 5% solids
- Dewatered sludge – from drying beds – 20% solids
- Dewatered sludge – from sludge press/dewatering container – 80% solids
- Dried sludge granules – from final drier - 95% or greater solids

There are three main options for sludge drying:

8.3.1 Drying off site

The DPWT have suggested that treated sludge be dried off-site at a location near to the existing solid waste dump, some 26km distant from the WWTP. The site is up a hill along an access road approximately 4km from the main NR#14. There would be problems transporting thickened sludge from the lagoons (being about 5% solids) over this distance. It is more feasible that the thickened sludge be allowed to dry further in the lagoon before being emptied, to a solids content that can easily be handled, and then disposed of direct to landfill. This is an option if the current lagoon system is maintained as there is no possibility to have a suitably sized drying facility on site.

8.3.2 Mechanised sludge press

A mechanized sludge press could be purchased and used on the limited land available on the current lagoon site. This has the advantage of having a small footprint, but the obvious disadvantage of requiring maintenance and requiring power. A sludge drier can further dry the dewatered sludge from the press into dry granules but this is not necessary for landfill disposal.

8.3.3 Sludge dewatering container

This resembles a shipping container but is watertight and contains a filter, often a geotextile with fine holes. Polymer is added to bond small biosolids and stop them from forming a film across the filter that eventually blinds it. Whilst mixers and vacuum pumps can be added to improve results, a more basic model that would be appropriate for Sihanoukville requires no power or moving parts other than the small mixer in the poly storage tank. The container is loaded by an excavator from a truck, and clear filtrate water is piped back into the facultative or maturation lagoon. The dewatered sludge can then be loaded back onto a truck for delivery to landfill or land application. As de-sludging only occurs for each lagoon in rotation every year or longer, a truck can be rented or taken from PWD rather than purchased. This is an appropriate option if the lagoons are fully retained and land is limited. An example of a permanently mounted tipping container is shown in Figure 10.

Figure 10 - Permanently mounted tipping type sludge dewatering container

8.3.4 Incorporation of drying beds into a redeveloped facultative lagoon area

Should the option of trickling filters be selected, all or part of the current area taken up by facultative lagoons will be available for redevelopment (part of this area may be required for extension or re-modelling of the anaerobic lagoons).

The existing facultative lagoons (2) take up 165.75m (L) x 130m (W) of the site. From Table 24, two filters of 16.3m diameter would be required to meet projected 2040 wastewater flows. These two filters plus associated equipment could easily fit into one quarter (40m of the 165m lagoon site length) of the available area. If a second quarter was reserved for further future expansion then all or part of the remaining 80m (L) by 130m (W) could be used for sludge drying.

However, before this investment is ever made, reasons for the current lack of O&M and a commitment by DPWT to carry out proper periodical desludging, and secure budgets to do so, must be clearly established.

8.4 Composting

Composting of final dried, treated sludge is worthwhile if there is a local market for the final product with no cultural aversions against using fertilizer of human origin on food crops. Composting methods are discussed in the "Sector Analysis, Development Roadmap and 20 Year Investment Roadmap for Urban Sanitation" part of the Draft Final Report.

Composting requires a fair amount of attentive manual labour in the form of turning the compost, and ensuring it is covered from rain. There is no existing market for it in Sihanoukville or Cambodia, in particular as there are large food growing areas with naturally fertile soils from the Tonle Sap lake and many river flood plains. Composting of drier treated sludge is therefore not recommended.

8.5 Disposal to landfill

Dewatered sludge can then be used as cover for the proposed landfill rehabilitation project, which is due to be constructed in 2017. Once landfill material has been compacted in layers every 1-2 weeks, it is capped, and treated dewatered sludge is ideal for this purpose.

9 Recommended WWTP Option

The WWTP recommendation has been split into two stages, one for the medium term to serve the areas that are currently developed with a significant population density. These areas are covered by the current and 5 proposed new sub-catchments. A longer term option is proposed that addresses further future development areas outside of the 6 already covered along with further urban industry along NR#4.

9.1 WWTP: medium term

In the short to medium term, during which the 5 prioritised new service areas will be connected to the separated wastewater system, it is proposed that significant changes to the current wastewater treatment regime in order to increase the capacity of the WWTP are minimised, whilst ensuring that any technology changes introduced are energy efficient.

As the current system, after 8 years of operation, has just surpassed 50% of the originally intended connections from the 2002 design, and as there are still clearly challenges with providing adequate O&M to the plant (sludge drying and handling, influent and effluent quality and volume monitoring), it is recommended that floating solar powered mixers be used to increase the capacity of the current WWTP to between 200-300% of the current maximum. These mixers are stand-alone and, whilst more expensive than conventional mixers, use more advanced technology to offer significant lifetime power cost savings, and offer a very low carbon footprint. The solar mixers will also have the option to be ran from mains power during periods of lower sunlight or during maintenance of solar panels.

It is further proposed that anaerobic ponds are to be fully de-sludged, access ramps for a compact excavator installed, and desludging equipment supplied. A 6m^3 septage vacuum truck, compact excavator and sludge dewatering container will be supplied. Having properly operating anaerobic ponds will reduce the COD loading to the facultative ponds, and increase efficiency and capacity overall.

If a 250% increase in capacity is attained, there will be sufficient capacity to treat wastewater from the existing 2008 service area plus the proposed 5 new service areas to year 2040, with the originally planned 1,000m^3/d allowance for wastewater from Cambrew retained.

If the targeted 300% increase in capacity is attained then there will be additional capacity for 4,000m^3/d from Cambrew. Should the increased WWTP capacity fall between 250-300% then the full 4,000m^3/d flow from Cambrew can still be received, but the design capacity lifespan will be reduced from 2040 to between 2035-2040.

Addition of solar powered mixers in a carefully planned configuration has the ability to provide the desired increase in capacity to serve the current and proposed service areas for the next 20 years and has the advantages of;

- Retaining the lagoon system, widely acknowledged as the simplest most effective wastewater treatment method for developing countries, provided sufficient land is available.
- Low maintenance
- Relatively low investment cost to meet the required capacity increase
- Low energy and O&M costs

A concrete receiving bay with bar screens and a service water point will be added to each anaerobic lagoon to facilitate offloading of septage form vacuum trucks into the lagoons for co-treatment.

Portable wastewater quality testing equipment will be provided.

9.2 WWTP: Medium to long term

Once the proposed 5 priority sub-catchments are served, there will be very few developed areas in Sihanoukville that will not be covered by the WW system for the short to medium term. In the longer term, when further areas of the wider urban area which are currently sparsely populated become developed, it is proposed to follow the option described fully under Section 6.2.2 and employ conventional trickling filters. The main (sparsely populated) areas left without coverage pre-2040 will be;

- The area to the immediate east of the original 1998 sub-catchment, part of Sangkhat Buon (Sangkhat 4). This area, to the south of Mittaphab St, has low to moderate density population and is the only current unserved area of any significance in terms of population.
- Sokha beach area – other than the resort this area has little or no housing. The resort has a package plant.
- The port area has little residential housing, and is in a catchment that would require 2 stages of pumping to transport wastewater to the WWTP. Along NH4 from the port to the east (in the direction of Phnom Penh) is lightly developed from outside of the port area up to Cambrew.
- The area north of the port along the railway line is an illegal squat area.
- The large areas starting from 300m inland from Occheuteal beach are currently undeveloped
- The large areas starting from 150m inland from Otres beach are currently undeveloped except for along one access road to the south of Otres which is covered under Block 2.

9.3 Operation and Maintenance requirements

For the medium term option of aerated lagoons, O&M requirements will be largely the same as they are now, focused around sampling of influent and effluent waters and periodical desludging of the anaerobic lagoons. These lagoons should be desludged when they approach 50% depth of settled sludge. This will be made easier for the Operators under the proposed subproject with the remodelling of the anaerobic lagoons with an access ramp and the provision of a compact excavator and portable sludge pumps.

When a lagoon is ready for desludging it will first be dewatered using the sludge pumps. This should always be carried out during the dry season. Once dewatered, the lagoon should be left for several weeks for evaporation to thicken the sludge, before it is removed using the excavator and a truck, and deposited in batches into the sludge dewatering container, which will thicken the sludge further. It can then be carted to landfill for disposal.

For the medium to long term, the current facultative lagoon area will be remodeled, part of it becoming an extension to the anaerobic lagoons, and part being reclaimed for siting conventional trickling filters. In addition to the lagoon maintenance described above, the trickling filters will require ongoing maintenance.

For conventional trickling filters, the main operational requirement is that flow needs to be constant, as if flow is paused the filter drains, which can cause bacteria on the media to dry out and die. This constant hydraulic loading may be difficult during low flow periods, such as night time or during power outages. There are various design methods often utilised to maintain a constant flow, such as dosing siphons or recirculation, but only the lowest technology option is recommended for this application.

It is therefore most important for the Operator to monitor flows and be prepared to repair or replace feed pumps at short notice.

The rotating arm that distributes WW over the media surface may also have mechanical problems from time to time and require rapid maintenance to prevent loss of the filter live bacteria.

The Operator needs to monitor odour. Odour problems can occur when the centre of the filter becomes anaerobic due to not having enough air getting through, or due to excessive or unexpected loads.

The biofilm, or layer of bacteria on the media, must be kept thin to prevent the filter clogging. The filter will need cleaning with a backwash periodically to avoid this.

The main costs of O&M are from paying the salary of one or more full time operators, power costs for the pumps and motor to power the rotating arm, and occasional repairs and replacements of mechanical parts. Under normal circumstances, the power costs are the most significant of these. Estimated O&M costs are provided in Table 31.

Table 31 - O&M cost estimate - both options

Trickling filter option	Estimated annual cost ($)	Aerated Lagoon option	Estimated annual cost ($)
Trickling filter pumps - spares	5,000	Mixer spares	5000
Trickling filter pumps- power[10]	81,000	Mixer mains power	5000
Trickling filter arms - power	15,000		
Fuel for excavator	2,000	Fuel for excavator	2,000
Fuel for vacuum truck	2,000	Fuel for vacuum truck	2,000
Mechanical maintenance for vehicles	5,000	Mechanical maintenance for vehicles	5,000
Permanent operator salary x2	25,000	Permanent operator salary x2	25,000
Labour for desludging	10,000	Labour for desludging	10,000
Total	**145,000**		**54,000**

9.4 Sustainability

The proposal must be seen to be sustainable for the investment to be attractive to all parties with a vested interest. The preliminary design of the Sihanoukville WW collection and treatment system has been carried out with this in mind, and factors illustrating the likelihood of ongoing financial sustainability are;

- Sihanoukville is one of the largest cities in Cambodia, and therefore has one of the largest customer bases.
- Free household connections are proposed for all properties under the project, in both the proposed 5 new service areas and the remainder of the households under the original 2008 service area. This amounts to 8,295 households by 2020 which will be obliged to pay a WW fee.
- Street collection pipelines along both sides of every street are proposed under the preliminary design. This will enable easy connection of both current and future properties
- Sihanoukville has current experience in collecting a WW fee, paid direct to DPWT. The billing process is already in place.

[10] 37kW running full time, $0.25/kWhr

10 Equipment and Vehicle Purchases

The following equipment is recommended for purchase under the contract.

Table 32 - Equipment purchase

Description	Quantity	Purpose
6m^3 vacuum septage truck	1	Septage collection DPWT
High flow portable sludge pumps (min 10l/sec)	2	Lagoon desludging
Compact excavator	1	Lagoon desludging
Portable influent/effluent quality testing equipment	1	Monitoring
Sludge dewatering container	1	Sludge dewatering

11 Preliminary Cost Estimate

11.1 Preliminary cost estimates for recommended option

Table 34 shows the preliminary cost estimate. Breakdowns of costs for pipe are included in Appendix C.

Table 33 - Preliminary cost estimate

Item	Unit	Unit rate	Qty	Total ($)
Solar Aerators with hoses and freight	ea	75,000	12	900,000
Desludging of 4 anaerobic lagoons	m^3	10	20,000	200,000
Disposal of sludge to landfill	m^3	5	20,000	100,000
Concrete pad and effluent pipe for dewatering container	LS	5,000	1	5,000
Shaded storage for sludge pumps, excavator, vacuum truck	m2	100	50	5,000
Receiving bays for anaerobic ponds	ea	20,000	4	80,000
Pump stations/pump wells	ea	150,000	5	750,000
Pumps with VSD's	ea	50,000	10	500,000
Trunk sewers	LS			5,684,814
Main 1200mm trunk sewer	LS			972,450
Pumping mains for 5 new areas	LS			1,490,364
WW mains (for connections)	LS			1,249,290
Household connections	ea	100	8,295	829,500
Supply compact excavator	ea	50,000	1	50,000
Supply 6m^3 vacuum truck	ea	80,000	1	80,000
Supply sludge dewatering container	ea	100,000	1	100,000
Portable sludge pumps	ea	20,000	2	40,000
Supply WQ testing equipment	kit	25,000	1	25,000
WQ testing training	LS	10,000	1	10,000
Public awareness campaign	LS	100,000	1	100,000
Training/capacity building	LS	150,000	1	150,000
Subtotal				**13,321,418**
10% contingency				1,332,142
TOTAL				**14,653,560**

Appendix A: Topo survey

Appendix B: Trunk sewer plans by service area block

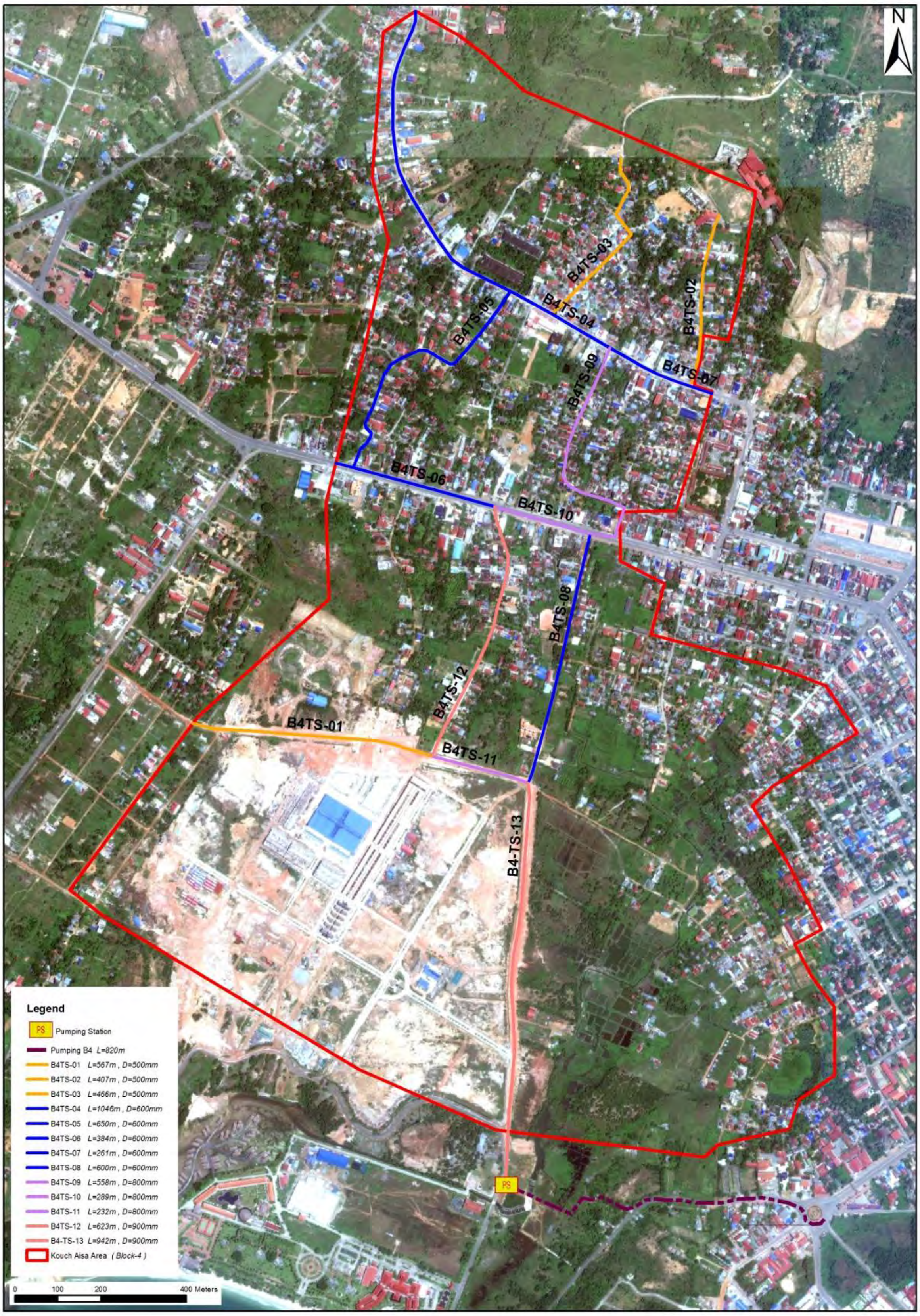
Occheuteal Beach

Otres Beach


N
B2TS-01
B2TS-06
B2TS-02
B2TS-03
B2TS-10
PS
Lowest point for pumping station
B2TS-07
B2TS-09
B2TS-08
B2TS-04
B2TS-05
Legend
Pumping Station
Pumping Main L= 4576m
B2TS-01 L=244m , D=250mm
B2TS-02 L=1244m , D=400mm
B2TS-03 L=1061m , D=400mm
B2TS-04 L=652m , D=400mm
B2TS-05 L=820m , D=400mm
B2TS-06 L=870m , D=400mm
B2TS-07 L=2246m , D=500mm
B2TS-08 L=513m , D=600mm
B2TS-09 L=817m , D=700mm
B2TS-10 L=106m , D=800mm
Otres Beach (Block-2)
0 185 370 740 1,110 1,480 Meters

Victory Beach & Meteorology

Kuch Asia


N
B4TS-01
B4TS-02
B4TS-03
B4TS-04
B4TS-05
B4TS-06
B4TS-07
B4TS-08
B4TS-09
B4TS-10
B4TS-11
B4TS-12
B4-TS-13
PS
Legend
PS Pumping Station
Pumping B4 L=820m
B4TS-01 L=567m , D=500mm
B4TS-02 L=407m , D=500mm
B4TS-03 L=466m , D=500mm
B4TS-04 L=1046m , D=600mm
B4TS-05 L=650m , D=600mm
B4TS-06 L=384m , D=600mm
B4TS-07 L=261m , D=600mm
B4TS-08 L=600m , D=600mm
B4TS-09 L=558m , D=800mm
B4TS-10 L=289m , D=800mm
B4TS-11 L=232m , D=800mm
B4TS-12 L=623m , D=900mm
B4-TS-13 L=942m , D=900mm
Kouch Aisa Area (Block-4)
0 100 200 400 Meters

Independence Beach

Appendix C: Service pipeline maps and lengths by Service area block

Ouccheteal Beach (Block-1)	
Road in Block	Length (m)
R-1	279
R-2	215
R-3	249

Kouch Asia (Block-4)	
Road in Block	Length (m)
R-1	510
R-2	1067
R-3	781
R-4	710
R-5	319
R-6	684
R-7	240
R-8	498
R-9	439
R-10	528
R-11	317
R-12	387
R-13	219
R-14	202
R-15	248
R-16	197
R-17	139
R-18	247
R-19	572
R-20	509
R-21	252
R-22	287
R-23	343
R-24	209

Otres Beach (Block-2)	
Road in Block	Length (m)
R-1	595
R-2	226
R-3	506
R-4	722
R-5	137
R-6	141
R-7	130
R-8	284

Independence Beach (Block-5)	
Road in Block	Length (m)
R-1	465
R-2	228
R-3	275
R-4	254
R-5	500
R-6	495
R-7	252
R-8	378
R-9	516
R-10	214
R-11	144
R-12	208
R-13	492
R-14	560
R-15	324
R-16	145
R-17	128

Victory Beach (Block-3)	
Road in Block	Length (m)
R-1	331
R-2	326
R-3	298
R-4	178
R-5	185
R-6	342
R-7	140
R-8	263
R-9	229
R-10	163
R-11	374
R-12	422
R-13	322
R-14	346
R-15	300

Total road length = 23,135m
Total service pipe length (2 sides road) = 46,270m
Not including trunk sewers or rising mains. These have been considered separately.

Appendix D: Pipe cost estimate breakdowns

D1: Trunk sewers

No.	Trunk Diameter	Length (m)	Approx cost/m	Fittings	Laying	Cost	
1	uPVC 250mm	244	30	12	12	13,176	
2	uPVC 300mm	940	40	16	16	67,680	
3	uPVC 400mm	5,667	60	24	24	612,036	
4	GRP 450mm	7,347	80	32	32	1,057,968	
5	GRP 500mm	3,686	90	36	36	597,132	
6	GRP 600mm	3,868	110	44	44	765,864	
7	GRP 700mm	2,742	130	52	52	641,628	
9	GRP 900mm	3,544	150	60	60	956,880	**4712364**
10	GRP 1200	2,161	250	100	100	972,450	
						5,684,814	

D2: Rising mains

Criteria/Block	Condition	B-1	B-2	B-3	B-4	B-5
Elevation (m)	Inlet	3.7	2	7.5	5.6	6
	Outlet	4.5	4.5	45	17	17
	Highest point	4.5	12.1	45	19.9	19.9
Flow (m^3/day)	Year 2030	1,070	415	1,265	2,637	2,156
	Year 2040	1,890	723	1,876	3,396	2,992
Pumping main diameter (mm)	Year 2030	0.5	0.3	0.5	0.8	0.7
	Year 2040	0.7	0.4	0.7	0.9	0.8
Pumping main distance (m)	-	1,528	4,576	1,131	809	2,249

Rising mains	Diameter	Length (m)	cost/m	Fittings	Laying	Cost
	500	1528	90	36	36	247536
	300	4576	40	16	16	329472
	500	1131	90	36	36	183222
	800	809	140	56	56	203868
	700	2249	130	52	52	526266
						1490364

D3: Service pipes 150mm uPVC

Service pipes	150	46,270	15	6	6	**1,249,290**

Appendix E: Manufacturers Solar Mixer Data

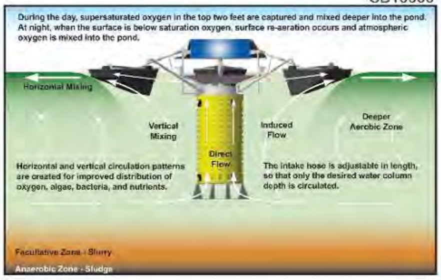
GridBee®
SolarBee®

Wastewater Benefits

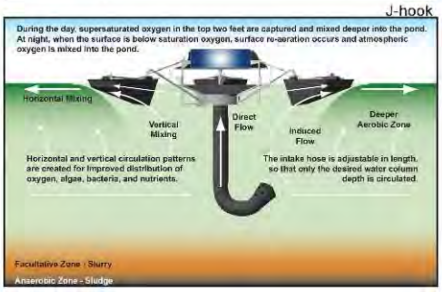
SolarBee and GridBee mixers are designed to solve a variety of municipal and industrial wastewater quality problems including high energy costs, EPA discharge permit violations and odor control. SolarBee and GridBee mixers can operate 24/7 to improve the treatment system, supplying most of the mixing energy required in any treatment pond and reducing the hours that the aeration system must operate.

Flow rates vary by mixer size and range from 1,250 to 10,000 gallons per minute. For each application, a custom proposal is provided at no charge, which shows the expected results.

SB10000

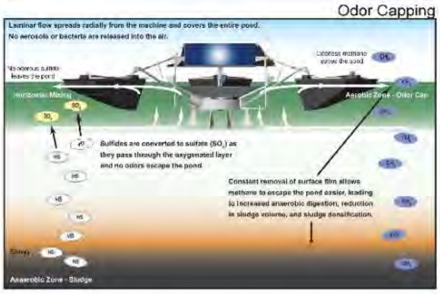


J-hook



Dual Mix

Odor Capping



Wastewater Benefits:

- Reduces energy consumption by reducing run time of existing aeration/mixing equipment
- Provides improved mixing in all systems
- Improves BOD, TSS and ammonia reduction
- Controls odors (odor capping systems available)
- Improves sludge digestion, reduces the need for dredging
- Reduces short-circuiting and fecal coliform counts

Features:

- 316 stainless steel and non-corrosion polymer construction
- Intake designs specific to the application
- 2-year machine warranty, 10-year motor warranty, 25-year service life
- Factory installation, water testing and sludge depth testing available
- Solar units: Day and night operation on solar power
- Solar units: Digital control system programmable with anti-jam and auto-reverse; SCADA outputs available
- Solar units: High-efficiency, high-torque brushless motor with no gearbox for low-speed operation

Applications:

- All aerobic wastewater ponds
- Activated sludge reactor basins
- Water reuse / effluent storage ponds
- Odor capping of anaerobic ponds

SolarBee® and GridBee® are brands of Medora Corporation

Locally Represented By:

Medora Corporation
3225 Hwy 22 • Dickinson, ND 58601
Ph +1 866 437 8076 • www.medoraco.com

SolarBee®
Circulating the World's Water

SB10000 v18 FEATURES

Technology Description:	Floating, solar powered, circulation equipment for wastewater treatment and freshwater applications. Day/night operation on solar only by utilizing a battery to store excess daytime energy for nighttime operation.
Flow Rates:	*Flow rates at full speed at 10 feet (3 meter) diameter.*
Direct Flow Rate - Day	3,000 gallons per minute, (12,000 Liters per minute).
Induced Flow Rate - Day	7,000 gallons per minute. (26,000 Liters per minute).
Combined Flow Rate - Day	10,000 gallons per minute. (38,000 Liters per minute).
Combined Flow Rate - Night	10,000 gallons per minute (38,000 Liters per minute).
Machine Size/Weight:	Assembled machine is 16 feet (5m) in diameter and weighs 850 pounds (385kg).
Shipping Size/Weight:	Machine can be crated and shipped in a 87 inch (2.2 m) wide X 87 inch (2.2 m) long X 65 inch (1.7 m) high crate, estimated shipping weight is 1500 pounds (680 kg). Hose lengths greater than 40 ft (6 m) may require additional crating.
Materials of Construction:	316 stainless steel construction. Foam-filled high-density polyethylene (HDPE) floats. Thermoplastic rubber intake hose. HDPE strainer. Concrete mooring blocks are encapsulated in HDPE.
Drive System:	High torque, direct drive (no gearbox), low voltage brushless D.C. motor.
Power Supply/Control System:	*PV solar panels are protected from bird fouling with bird deterrent kit.*
PV Solar Panels	3 X 80-watt photovoltaic solar panels orientated in triangular pattern. On-board battery storage for day/night operation.
Electronic Controller	Digital solid-state controller, mounted in weather-tight (NEMA 4X) enclosure with externally fused disconnect. SCADA output through RS-232 serial communication (Modbus RTU), DB9 male connection point inside enclosure. Wireless options available, not included.
Wiring	Corrosion-resistant industrial cord with molded watertight connectors that are indexed to prevent improper wiring.
Rotating Assembly:	Removable assembly with easy access to motor and digital controller. Impeller handles 4-inch (10 cm) spherical solids. Oil-filled (food grade) teflon freeze sleeve with o-rings, shaft. Rotational indicator on shaft.
Flotation System:	Three floats in triangular pattern each with an adjustable float arm for proper vertical positioning, total float buoyancy of 1,350 pounds (612kg).
Fluid Intake Assembly - Option 1:	*Hose system bolted to bottom of structural assembly.*
Hose System	10 to 100 feet (3 to 30.5 meters) available in 36 inch (91 cm) diameter X 10, 15, or 20 feet (3, 4.5, 6 meter) sections.
Intake Type	Horizontal plate with 12-inch (31 cm) openings.
Intake Depth Adjustment	15 feet (4.5 m) of field adjustment with three SS chains connected to hose coupling.
Fluid Intake Assembly - Option 2:	Fixed horizontal plate bolted to bottom of structural assembly.
Intake Type	Fixed horizontal plate with 12-inch (31 cm) openings.
Intake Depth Adjustment	No adjustment necessary. Horizontal inflow from 25 in (64 cm) below distribution dish.
Fluid Intake Assembly - Option 3:	*See SB10000DM v18 Dual Mix Features Sheet.*
Anchoring:	(1) Two mooring blocks tethered together with SS chain and attached to structural member on unit or (2) Tethered to shore with SS cable.
Ice Protection:	Freeze sleeve and positive pumping under distribution dish to maintain circulation.
Minimum Operating Depth:	31 inches (0.8 m) with fixed horizontal plate. No damage to machine or bottom of reservoir when run dry in shallow water.
Accessories Available:	(1) Supplemental Shore Power Kit, (2) Chemical Injection Kit, (3) Marker Light Kit
Life/Maintenance/Warranty:	Expect 25-year life, minimal maintenance. Limited 2-year parts and conditional labor warranty. Limited 25 year photovoltaic module manufacturer performance warranty and 10 year motor warranty.

Patent Pending

Subject to change without notice.

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10000v18-FEATURE_20150714

Appendix F: Pump selection and calculations

Contract No.:			Job No.:	-
			Sheet No.:	1 of 1
Project:	SihanoukVille_SANITATION		Revision No.:	
Designed by: F.S.ABRIGO	Date: 3-Oct-16	Checked: -	Date: -	

SHEET TITLE: ***B-1 PS_Sihanoukville, SUMP/PUMP SIZING AT 1,070 cu.M/day (2033)***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow	= 1,070.0 m^3/day	=	12.38	lps	
	0.743 m^3/min	=	44.6	m^3/hr	
		=	196.3	gpm	
No. of Pumps		=	1	+ 1	Stand-By
Recommended Pump Cycle	maximum	=	10	cycle/hour	
		=	6	mins.	
	minimum	=	6	cycle/hour	
		=	10	mins.	
Design Velocity					
at Suction: For Pump Entrance of larger than		=	3.6	m/s	200 mm dia.,
For Pump Entrance of smaller than		=	5.5	m/s	200 mm dia.,
at Discharge: Should be Less Than		=	3.5	m/s	(As per recommendation)
Length of Rectangular Sump		=	2	m	
Width of Rectangular Sump		=	2	m	
Diameter of Circular Sump		=	2	m	

Total Friction Loss in Pipes & Fittings

(Pipe length)	2.06 ft.	K Factor
Piping	0.0378	6.00
Check Valves	3.941	3.00
Ball Valves	0.197	0.15
Elbows	0.236	0.18
Tee	0.788	0.60
Total H_L	**5.20** ft.	
	1.59 m	

Headloss for Valves & Fittings $= k(v^2/2g)$
where: $(v^2/2g) = 0.4$
Headloss for Straight Pipe $= k*(L/100)$
Length of Force main, m = 1,528.0
Diameter of Force main, mm = 0.5
K Factor = 0.01
Total Headloss at Force Main, m: 0.5012

C. DESIGN CALCULATION

Q per pump		=	12.38	lps
		=	44.58	m^3/hr
		=	196.32	gpm
Volume Required:	For Maximum 10 cyc/hour	=	1.11	m^3
	For Minimum 6 cyc/hour	=	1.86	m^3
Volume Required:			*6.05*	m^3
Wet Well Volume Required:		*USE:*	**2.20**	m^3
Height of The Sump:	Rectangular	=	0.55	m
	Circular	=	*0.70*	m
USE:	**Rectangular**	=	**0.55**	m
Riser Pipe Diameter Required:		=	0.067	m say **0.075** m

Volume, $_{Req}$ $=(\Phi*q)/4$ M^3
where:
Φ =minimum time in minutes, min
q =pump capacity, m^3/min

Water in Sump Vol $_{Total}$ = **2.2**
Water in Pipe Vol $_{Total}$ = **3.85**
(incoming)

$$TDH_{Required} = PWL + Discharge\ Head + Headloss_{Total}$$

where:

PWL= HWL-LWL	=	0.55
Discharge Head = $(PS_{GL}-HWL)+(DP\ el.-PS_{GL})+HL_{FM}$	=	2
Total Headloss=	=	2.1
Total Dynamic Head$_{Required}$ = **4.69** m	say =	**5** m
Pump Efficiency	=	68.0%
Power Required	=	1.40 hp say **1.5 hp**
	=	1.12 KW say **1.1 kW**

Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%
*			

Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **0.772 m**

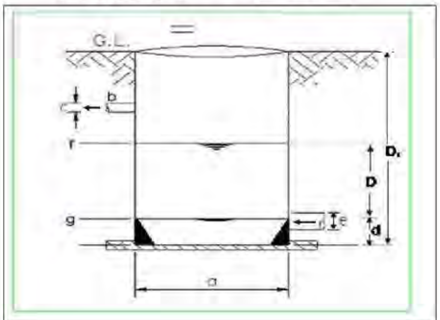
To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: $F= v/(gD)^{1/2}$ = **4.674**
V= vel. @ suction bell, m/s = 3.732
D= O.D of Suction bell, m = 0.065
Submergence= 0.764 **m**

FOR PUMP's DISCHARGE VELOCITY

$Q_{Required}$ = 0.0124 m^3/s
Pipe Area = 0.0044 m^2
Velocity = **2.80** m/s < 3.5 m/s **'PASSED'**

FOR SUCTION VELOCITY:

V= vel.@ suction bell, m/s = **3.73**



SUMP DETAILS

ITEM	DESCRIPTIONS	DIMENSION, m	ELEVATION, m
PS_{GL}	Ground Level	n/a	3.700
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	2.500
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	1.50	
e	Incoming Pipe Diameter	0.7	2.400
	I.L of Incoming Pipe		1.700
f	Pump Start. **(HWL)**		2.950
g	Pump Stop Level **(LWL)**	0.55	2.400
	Pump Min. Water Level Reqt. **(d)**	0.76	
h	Wet well Depth	1.31	
i	Dis. From Incoming Pipe to Bottom	1.31	1.336
BL	Bottom Level	n/a	1.336
DP_{GL}	Discharge Point Ground level		4.50

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS
(For Structural Design Reference)

3.45	**HHWL**, m
2.95	**HWL**, m
6.05	**Volume** $_W$, cu.m
2.40	**LWL**, m
2.10	**LLWL**, m
0.76	**d**, m
1.31	**D + d**, m
2.36	**Depth** $_{TOTAL}$

	Contract No.:			Job No.:	-
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	Project:	SihanoukVille_SANITATION		Revision No.:	
	Designed by: F.S.ABRIGO	Date: 3-Oct-16	Checked: -	Date: -	
SHEET TITLE:	***B-1 PS_Sihanoukville-2040, SUMP SIZING AT 1,890 cu.M/day***				

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 1,890 m^3/day = 21.88 lps
1.313 m^3/min = 78.8 m^3/hr
= 346.8 gpm
No. of Pumps = 1 + 1 **Stand-By**
Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.
Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)
Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	2.60 ft.	K Factor
Piping	0.0475	6.00
Check Valve	3.891	3.00
Ball Valves	0.195	0.15
Elbows	0.233	0.18
Tee	0.778	0.60
Total H_L	**5.14** ft.	
	1.57 m	

Headloss for Valves & Fittings $= k(v^2/2g)$
where: $(v^2/2g)$= 0.395
Headloss for Straight Pipe $= k^*(L/100)$
Length of Force main, m = 1,528.0
Diameter of Force main, mm = 0.7
K Factor = 0.01
Total Headloss at Force Main: **0.5012**

C. DESIGN CALCULATION

Q per pump = 21.88 lps
= 78.75 m^3/hr
= 346.76 gpm
Volume Required: For Maximum 10 cyc/hour = 1.97 m^3
For Minimum 6 cyc/hour = 3.28 m^3
Volume Required: *6.05* m^3
Wet Well Volume Required: *USE:* **2.20** m^3
Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55** m
Riser Pipe Diameter Required: = 0.089 m say **0.1** m

Volume, $_{Req}$ $=(\Phi^*q)/4$ M^3
where:
Φ =minimum time in minutes, min.
q =pump capacity, m^3/min
Water in Sump Vol.$_{Total}$= **2.2**
Water in Pipe Vol.$_{Total}$= **3.85**
(Incoming)

T D $H_{Required}$ = PWL + Discharge Head + $Headloss_{Total}$
where: PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}$-HWL)+(DP el.-PS_{GL}+HL_{FM} = 2
Total Headloss= = 2.1
Total Dynamic Head$_{Required}$ = **4.67** m say = **5** m
Pump Efficiency = 68.0%
Power Required = 2.47 hp say **3** hp
= 2.24 KW say **2.2** kW

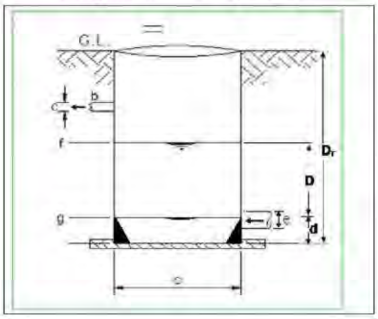
Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%
*			

Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **0.772 m**

To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: $F= v/(gD)^{1/2}$ = **8.255**
V= vel. @ suction bell, m/s = 6.592
D= O.D of Suction bell, m = 0.065
Submergence= 1.299 **m**

FOR PUMP's DISCHARGE VELOCITY
$Q_{Required}$ = 0.0219 m^3/s
Pipe Area = 0.0079 m^2
Velocity = **2.79** m/s < 3.5 m/s **'PASSED'**
FOR SUCTION VELOCITY:
V= vel.@ suction bell. = **6.59**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION, m**
PS_{GL}	Ground Level	n/a	3.700
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	2.500
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	1.50	
e	Incoming Pipe Diameter	0.7	2.400
	IL of Incoming Pipe		1.700
f	Pump Start, **(HWL)**		2.950
g	Pump Stop Level **(LWL)**	0.55	2.400
	Pump Min. Water Level Reqt. **(d)**	1.30	
h	Wet well Depth	1.85	
i	Dis. From Incoming Pipe to Bottom	1.85	0.801
BL	Bottom Level	n/a	0.801
DP_{GL}	Discharge Point Ground level		4.50

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS (For Structural Design Reference)	
3.45	**HHWL**, m
2.95	**HWL**, m
6.05	**Volume** $_W$ cu.m
2.40	**LWL**, m
2.10	**LLWL**, m
1.30	**d**, m
1.85	**D + d**, m
2.90	**Depth** $_{TOTAL}$

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Project:	SihanoukVille_SANITATION			Revision No.:	
Designed by:	Date:	Checked	Date:		
F.S.ABRIGO	4-Oct-16	-	-		

SHEET TITLE: ***B-2.PS_Sihanoukville, SUMP SIZING AT 415 cu.M/day***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 415.0 m^3/day = 4.80 lps
0.288 m^3/min = 17.3 m^3/hr
= 76.1 gpm
No. of Pumps = 1 + 1 **Stand-By**
Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.
Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)
Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	3.64 ft.	K Factor
Piping	0.0665	6.00
Check Valves	0.593	3.00
Ball Valves	0.030	0.15
Elbows	0.036	0.18
Tee	0.119	0.60
Total H_L	**0.84** ft.	
	0.26 m	

Headloss for Valves & Fittings $= k(v^2/2g)$
where: $(v^2/2g)$= 0.06
Headloss for Straight Pipe = k*(L/100)
Length of Force main, m = 4,576.0
Diameter of Force main, mm = 0.3
K Factor = 0.01
Total Headloss at Force Main: **1.5009**

C. DESIGN CALCULATION

Q per pump = 4.80 lps
= 17.29 m^3/hr
= 76.14 gpm

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min.
q =pump capacity, m^3/min

Volume Required: For Maximum 10 cyc/hour = 0.43 m^3
For Minimum 6 cyc/hour = 0.72 m^3
Volume Required: *6.05* ***m^3***
Wet Well Volume Required: ***USE:*** **2.20** ***m^3***
Water in Sump Vol.$_{Total}$ = **2.2**
Water in Pipe Vol.$_{Total}$ = **3.85**
(Incoming)
Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55** **m**
Riser Pipe Diameter Required: = 0.042 m say **0.075** **m**

T D H$_{Required}$ = PWL + Discharge Head + Headloss$_{Total}$
where: PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}\text{-}HWL)+(DP\ el.\text{-}PS_{GL})+HL_{FM}$ = 13
Total Headloss= = 1.8
Total Dynamic Head$_{Required}$ = **15.46** m say = **16** m
Pump Efficiency = 55.0%
Power Required = 2.22 hp say **3** **hp**
= 2.24 KW say **2.2** **kW**

Pump Brand/Model	Q, m^3/min	TDH, m	Eff..%
*			

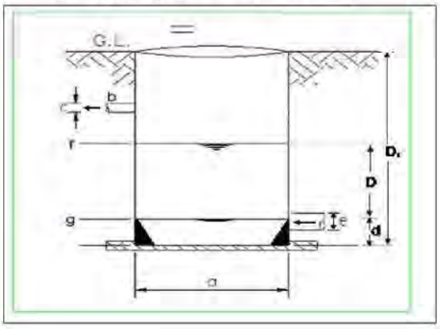
Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **0.772** **m**

To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **F**= $v/(gD)^{1/2}$ = **1.813**
V= vel. @ suction bell, m/s = 1.447
D= O.D of Suction bell, m = 0.065
Submergence= 0.336 **m**

FOR PUMP's DISCHARGE VELOCITY
Q $_{Required}$ = 0.0048 m^3/s
Pipe Area = 0.0044 m^2
Velocity = **1.09** m/s < 3.5 m/s **'PASSED'**

FOR SUCTION VELOCITY:
V= vel.@ suction bell, m/s = **1.45**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION, m**
PS_{GL}	Ground Level	n/a	2.000
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	0.800
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	3.50	
e	Incoming Pipe Diameter	0.7	-1.300
	I.L of Incoming Pipe		-2.000
f	Pump Start. **(HWL)**		-0.750
g	Pump Stop Level **(LWL)**	0.55	-1.300
	Pump Min. Water Level Reqt. **(d)**	0.34	
h	Wet well Depth	0.89	
i	Dis. From Incoming Pipe to Bottom	0.89	-1.936
BL	Bottom Level	n/a	-1.936
DP_{GL}	Discharge Point Ground level		12.10

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS (For Structural Design Reference)	
-0.25	**HHWL**, m
-0.75	**HWL**, m
6.05	**Volume** $_{w}$, cu.m
-1.30	**LWL**, m
-1.60	**LLWL**, m
0.34	**d**, m
0.89	**D + d**, m
3.94	**Depth** $_{TOTAL}$

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	Project:	SihanoukVille_SANITATION			Revision No.:	
	Designed by:	Date:	Checked	Date:		
	F.S.ABRIGO	5-Oct-16	-	-		
SHEET TITLE:	***B-2.PS_Sihanoukville, SUMP SIZING AT 723 cu.M/day***					

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 723.0 m^3/day = 8.37 lps
0.502 m^3/min = 30.1 m^3/hr
= 132.7 gpm

No. of Pumps = 1 + 1 **Stand-By**
Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.

Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)

Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	3.84 ft.	K Factor
Piping	0.0702	6.00
Check Valves	1.799	3.00
Ball Valves	0.090	0.15
Elbows	0.108	0.18
Tee	0.360	0.60
Total H_L	**2.43** ft.	
	0.74 m	

Headloss for Valves & Fittings $= k(v^2/2g)$
where: $(v^2/2g) = 0.18$
Headloss for Straight Pipe $= k*(L/100)$
Length of Force main, m = 4,576.0
Diameter of Force main, mm = 0.4
K Factor = 0.01
Total Headloss at Force Main: 1.5009

C. DESIGN CALCULATION

Q per pump = 8.37 lps
= 30.13 m^3/hr
= 132.65 gpm

Volume Required: For Maximum 10 cyc/hour = 0.75 m^3
For Minimum 6 cyc/hour = 1.26 m^3
Volume Required: *6.05* ***m^3***
Wet Well Volume Required: *USE:* **2.20** ***m^3***

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min
q =pump capacity, m^3/min

Water in Sump Vol.$_{Total}$ = **2.2**
Water in Pipe Vol.$_{Total}$ = **3.85**
(Incoming)

Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55** **m**
Riser Pipe Diameter Required: = 0.055 m say **0.075** **m**

T D H$_{Required}$ = PWL + Discharge Head + Headloss$_{Total}$
where: PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}$-HWL)+(DP el.-$PS_{GL})+HL_{FM}$ = 13
Total Headloss= = 0.7
Total Dynamic Head$_{Required}$ = **14.54** m say = **15** m
Pump Efficiency = 68.0%
Power Required = 2.94 hp say **3.0** **hp**
= 2.24 KW say **2.2.** **kW**

Pump Brand/Model	**Q, m^3/min**	**TDH, m**	**Eff..%**

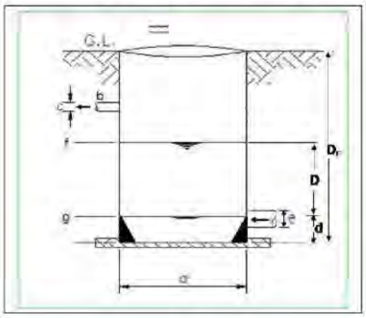
*
Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **0.772 m**

To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **F= v/(gD)$^{1/2}$** = **3.158**
V= vel. @ suction bell, m/s = 2.522
D= O.D of Suction bell, m = 0.065
Submergence= 0.537 **m**

FOR PUMP's DISCHARGE VELOCITY
Q $_{Required}$ = 0.0084 m^3/s
Pipe Area = 0.0044 m^2
Velocity = **1.89** m/s < 3.5 m/s **'PASSED'**

FOR SUCTION VELOCITY:
V= vel.@ suction bell, m/s = **2.52**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION, m**
PS_{GL}	Ground Level	n/a	2.000
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	0.800
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	3.50	
e	Incoming Pipe Diameter	0.7	-1.300
	I.L of Incoming Pipe		-2.000
f	Pump Start, **(HWL)**		-0.750
g	Pump Stop Level **(LWL)**	0.55	-1.300
	Pump Min. Water Level Reqt. **(d)**	0.54	
h	Wet well Depth	1.09	
i	Dis. From Incoming Pipe to Bottom	1.09	-2.137
BL	Bottom Level	n/a	-2.137
DP_{GL}	Discharge Point Ground level		12.10

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS	
(For Structural Design Reference)	
-0.25	**HHWL**, m
-0.75	**HWL**, m
6.05	**Volume** $_W$, cu.m
-1.30	**LWL**, m
-1.60	**LLWL**, m
0.54	**d**, m
1.09	**D + d**, m
4.14	**Depth** $_{TOTAL}$

	Contract No.:			Job No.:	-
				Sheet No.:	1 of 1
	Project:	SihanoukVille_SANITATION		Revision No.:	
	Designed by: F.S.ABRIGO	Date: 5-Oct-16	Checked -	Date: -	

SHEET TITLE: ***B-3.PS_Sihanoukville, SUMP/Pump SIZING At 1,265 cu.M/day (2030)***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 1,265.0 m^3/day = 14.64 lps
0.878 m^3/min = 52.7 m^3/hr
= 232.1 gpm

No. of Pumps = 1 + 1 **Stand-By**

Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.

Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)

Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

		K Factor
(Pipe length)	2.83 ft.	
Piping	0.0518	6.00
Check Valves	5.508	3.00
Ball Valves	0.275	0.15
Elbows	0.331	0.18
Tee	1.102	0.60
Total H_L	**7.27** ft.	
	2.22 m	

Headloss for Valves & Fittings = $k(v^2/2g)$
where: $(v^2/2g)$= 0.56

Headloss for Straight Pipe = k*(L/100)
Length of Force main, m = 1,265.0
Diameter of Force main, mm = 0.4
K Factor = 0.01
Total Headloss at Force Main: **0.4149**

C. DESIGN CALCULATION

Q per pump = 14.64 lps
= 52.71 m^3/hr
= 232.09 gpm

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min.
q =pump capacity, m^3/min

Volume Required: For Maximum 10 cyc/hour = 1.32 m^3
For Minimum 6 cyc/hour = 2.20 m^3

Volume Required: *6.05* ***m^3***
Wet Well Volume Required: ***USE:*** **2.20** ***m^3***

Water in Sump Vol $_{Total}$ = **2.2**
Water in Pipe Vol $_{Total}$ = **3.85**
(Incoming)

Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55** m

Riser Pipe Diameter Required: = 0.073 m say **0.075** **m**

T D $H_{Required}$ = PWL + Discharge Head + $Headloss_{Total}$

where:
PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}$-HWL)+(DP el.-PS_{GL})+HL_{FM} = 40
Total Headloss= = 2.6

Total Dynamic $Head_{Required}$ = **42.83** m say = **43** m
Pump Efficiency = 39.0%
Power Required = 26.39 hp say **30** **hp**
= 22.38 KW say **22** **kW**

Pump Brand/Model	Q, m^3/min	TDH, m	Eff..%

*

Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **1,190.0** **m**

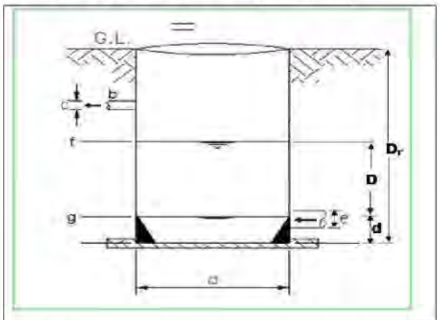
To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **$F= v/(gD)^{1/2}$** = **1.882**
V= vel. @ suction bell, m/s = 1.864
D= O.D of Suction bell, m = 0.1
Submergence= 0.533 **m**

FOR PUMP's DISCHARGE VELOCITY

$Q_{Required}$ = 0.0146 m^3/s
Pipe Area = 0.0044 m^2
Velocity = **3.31** m/s < 3.5 m/s **'PASSED'**

FOR SUCTION VELOCITY:

v= vel.@ suction bell, m/s = **1.86**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION , m**
PS_{GL}	Ground Level	n/a	7.500
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	6.300
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	2.50	
e	Incoming Pipe Diameter	0.7	5.200
	I.L of Incoming Pipe		4.500
f	Pump Start. **(HWL)**		5.750
g	Pump Stop Level **(LWL)**	0.55	5.200
	Pump Min. Water Level Reqt. **(d)**	0.53	
h	Wet well Depth	1.08	
i	Dis. From Incoming Pipe to Bottom	1.08	4.367
BL	Bottom Level	n/a	4.367
DP_{GL}	Discharge Point Ground level		45.00

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS	
(For Structural Design Reference)	
6.25	**HHWL,** m
5.75	**HWL,** m
6.05	**Volume** $_{W,}$ cu.m
5.20	**LWL,** m
4.90	**LLWL,** m
0.53	**d**, m
1.08	**D + d**, m
3.13	**Depth** $_{TOTAL}$

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Designed by:	Date:	Checked	Date:		
F.S.ABRIGO	5-Oct-16	-	-		

SHEET TITLE: ***B-3.PS_Sihanoukville, SUMP/Pump SIZING At 1,876 cu.M/day (2040)***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 1,876.0 m^3/day = 21.71 lps
1.303 m^3/min = 78.2 m^3/hr
= 344.2 gpm

No. of Pumps = 1 + 1 **Stand-By**
Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.

Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)

Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	3.04	ft.	K Factor
Piping	0.0556		6.00
Check Valves	12.115		3.00
Ball Valves	0.606		0.15
Elbows	0.727		0.18
Tee	2.423		0.60
Total H_L	**15.93**	ft.	
	4.86	m	

Headloss for Valves & Fittings $= k(v^2/2g)$
where: $(v^2/2g)$= 1.23
Headloss for Straight Pipe = k*(L/100)
Length of Force main, m = 1,131.0
Diameter of Force main, mm = 0.4
K Factor = 0.01
Total Headloss at Force Main: **0.3710**

C. DESIGN CALCULATION

Q per pump = 21.71 lps
= 78.17 m^3/hr
= 344.19 gpm

Volume Required: For Maximum 10 cyc/hour = 1.95 m^3
For Minimum 6 cyc/hour = 3.26 m^3
Volume Required: *6.05* ***m^3***
Wet Well Volume Required: *USE:* **2.20** ***m^3***
Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55** **m**
Riser Pipe Diameter Required: = 0.089 m say **0.075** **m**

Volume_{Req} **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min
q =pump capacity, m^3/min

Water in Sump Vol._{Total} = **2.2**
Water in Pipe Vol._{Total} = **3.85**
(Incoming)

T D $H_{Required}$ = PWL + Discharge Head + Headloss_{Total}
where: PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}$-HWL)+(DP el.-PS_{GL})+HL_{FM} = 40
Total Headloss= = 5.2
Total Dynamic $\text{Head}_{Required}$ = **45.43** m say = **46** m
Pump Efficiency = 39.0%
Power Required = 41.51 hp say **50** **hp**
= 37.30 KW say **37** **kW**

Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%
*			

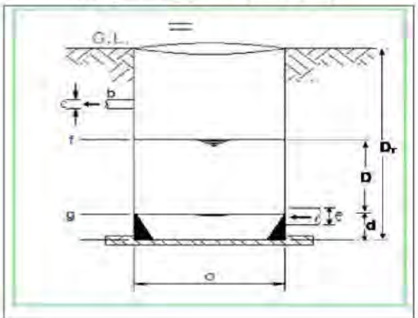
Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **1,190.0** **m**

To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **$F=v/(gD)^{1/2}$** = **2.791**
V= vel. @ suction bell, m/s = 2.765
D= O.D of Suction bell, m = 0.1
Submergence= 0.742 **m**

FOR PUMP's DISCHARGE VELOCITY
$Q_{Required}$ = 0.0217 m^3/s
Pipe Area = 0.0044 m^2
Velocity = **4.91** m/s < 3.5 m/s **REPEAT DESIGN**

FOR SUCTION VELOCITY:
v= vel.@ suction bell, m/s = **2.76**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION, m**
PS_{GL}	Ground Level	n/a	7.500
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	6.300
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	2.50	
e	Incoming Pipe Diameter	0.7	5.200
	I.L of Incoming Pipe		4.500
f	Pump Start, **(HWL)**		5.750
g	Pump Stop Level **(LWL)**	0.55	5.200
	Pump Min. Water Level Reqt. **(d)**	0.74	
h	Wet well Depth	1.29	
i	Dis. From Incoming Pipe to Bottom	1.29	4.158
BL	Bottom Level	n/a	4.158
DP_{GL}	Discharge Point Ground level		45.00

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS (For Structural Design Reference)	
6.25	**HHWL**, m
5.75	**HWL**, m
6.05	**Volume** $_W$ cu.m
5.20	**LWL**, m
4.90	**LLWL**, m
0.74	**d**, m
1.29	**D + d**, m
3.34	**Depth** $_{TOTAL}$

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F.S.ABRIGO	4-Oct-16	- / -		

SHEET TITLE: ***B-4.PS_Sihanoukville, SUMP/PUMP SIZING AT 2,637 cu.M/day (2030)***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 2,637.0 m^3/day = 30.52 lps
1,831 m^3/min = 109.9 m^3/hr
= 483.8 gpm

No. of Pumps = 1 + 1 **Stand-By**

Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.

Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)

Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	3.40 ft.	K Factor
Piping	0.0622	6.00
Check Valves	1.496	3.00
Ball Valves	0.075	0.15
Elbows	0.090	0.18
Tee	0.299	0.60
Total H_L	**2.02** ft.	
	0.62 m	

Headloss for Valves & Fittings = $k(v^2/2g)$
where: $(v^2/2g)$= 0.15
Headloss for Straight Pipe = k*(L/100)
Length of Force main, m = 809.00
Diameter of Force main, mm = 0.8
K Factor = 0.01
Total Headloss at Force Main: **0.2654**

C. DESIGN CALCULATION

Q per pump = 30.52 lps
= 109.88 m^3/hr
= 483.82 gpm

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min
q =pump capacity, m^3/min

Volume Required: For Maximum 10 cyc/hour = 2.75 m^3
For Minimum 6 cyc/hour = 4.58 m^3

Volume Required: *6.05* ***m^3***
Wet Well Volume Required: *USE:* **2.20** ***m^3***

Water in Sump Vol.$_{Total}$ = **2.2**
Water in Pipe Vol.$_{Total}$ = **3.85**
(Incoming)

Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55 m**

Riser Pipe Diameter Required: = 0.105 m say **0.15** m

T D H$_{Required}$ = PWL + Discharge Head + Headloss$_{Total}$
where:
PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}$-HWL)+(DP el-PS_{GL})+HL_{FM} = 17
Total Headloss= = 0.9

Total Dynamic Head$_{Required}$ = **18.38** m say = **19** m
Pump Efficiency = 66.0%
Power Required = 13.95 hp say **15 hp**
= 11.19 KW say **12 kW**

Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%

*
Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **1,198.0 m**

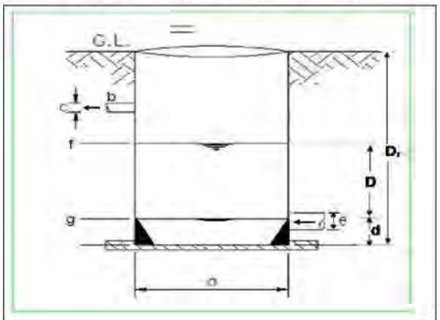
To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **$F= v/(gD)^{1/2}$** = **3.923**
V= vel. @ suction bell, m/s = 3.886
D= O.D of Suction bell, m = 0.1
Submergence= 1.002 **m**

FOR PUMP's DISCHARGE VELOCITY

Q $_{Required}$ = 0.0305 m^3/s
Pipe Area = 0.0177 m^2
Velocity = **1.73** m/s < 3.5 m/s **'PASSED'**

FOR SUCTION VELOCITY:

V= vel.@ suction bell, m/s = **3.89**



SUMP DETAILS

ITEM	DESCRIPTIONS	DIMENSION, m	ELEVATION, m
PS_{GL}	Ground Level	n/a	5.600
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	4.400
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	2.60	
e	Incoming Pipe Diameter	0.7	3.200
	I.L of Incoming Pipe		2.500
f	Pump Start, **(HWL)**		3.750
g	Pump Stop Level **(LWL)**	0.55	3.200
	Pump Min. Water Level Reqt. **(d)**	1.00	
h	Wet well Depth	1.55	
i	Dis. From Incoming Pipe to Bottom	1.55	1.898
BL	Bottom Level	n/a	1.898
DP_{GL}	Discharge Point Ground level		19.90

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS
(For Structural Design Reference)

4.25	**HHWL**, m
3.75	**HWL**, m
6.05	**Volume** $_W$ cu.m
3.20	**LWL**, m
2.90	**LLWL**, m
1.00	**d**, m
1.55	**D + d**, m
3.70	**Depth** $_{TOTAL}$

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	F.S.ABRIGO	4-Oct-16	-	-	
SHEET TITLE:	***B-4.PS_Sihanoukville, SUMP/PUMP SIZING AT 2,637 cu.M/day (2040)***				

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 3,396.0 m^3/day = 39.31 lps
2.358 m^3/min = 141.5 m^3/hr
= 623.1 gpm

No. of Pumps = 1 + 1 **Stand-By**
Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.

Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)

Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	3.66 ft.	K Factor
Piping	0.0670	6.00
Check Valves	2.481	3.00
Ball Valves	0.124	0.15
Elbows	0.149	0.18
Tee	0.496	0.60
Total H_L	**3.32** ft.	
	1.01 m	

Headloss for Valves & Fittings = $k(v^2/2g)$
where: $(v^2/2g)$= 0.25
Headloss for Straight Pipe = $k*(L/100)$
Length of Force main, m = 809.00
Diameter of Force main, mm = 0.9
K Factor = 0.01
Total Headloss at Force Main: 0.2654

C. DESIGN CALCULATION

Q per pump = 39.31 lps
= 141.50 m^3/hr
= 623.07 gpm

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where
Φ =minimum time in minutes, min
q =pump capacity, m^3/min

Volume Required: For Maximum 10 cyc/hour = 3.54 m^3
For Minimum 6 cyc/hour = 5.90 m^3
Volume Required: *6.05* ***m^3***
Wet Well Volume Required: *USE:* **2.20** ***m^3***

Water in Sump Vol.$_{Total}$= **2.2**
Water in Pipe Vol.$_{Total}$= **3.85**
(incoming)

Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55** **m**
Riser Pipe Diameter Required: = 0.120 m say **0.15** **m**

T D H$_{Required}$ = PWL + Discharge Head + Headloss$_{Total}$
where: PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}\text{-}HWL)+(DP\ el.\text{-}PS_{GL})+HL_{FM}$ = 17
Total Headloss= = 1.3
Total Dynamic Head$_{Required}$ = **18.88** m say = **19** m
Pump Efficiency = 66.0%
Power Required = 18.45 hp say **20** **hp**
= 14.92 KW say **15** **kW**

Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%

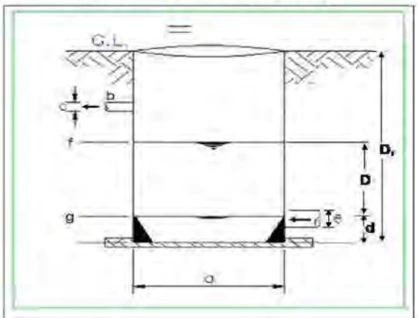
Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **1,198.0** **m**

To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **$F= v/(gD)^{1/2}$** = **5.053**
V= vel. @ suction bell, m/s = 5.004
D= O.D of Suction bell, m = 0.1
Submergence= 1.262 **m**

FOR PUMP's DISCHARGE VELOCITY
Q $_{Required}$ = 0.0393 m^3/s
Pipe Area = 0.0177 m^2
Velocity = **2.22** m/s < 3.5 m/s **'PASSED'**

FOR SUCTION VELOCITY:
V= vel.@ suction bell, m/s = **5.00**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION, m**
PS_{GL}	Ground Level	n/a	5.600
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	4.400
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	2.60	
e	Incoming Pipe Diameter	0.7	3.200
	I.L of Incoming Pipe		2.500
f	Pump Start, **(HWL)**		3.750
g	Pump Stop Level **(LWL)**	0.55	3.200
	Pump Min. Water Level Reqt. **(d)**	1.26	
h	Wet well Depth	1.81	
i	Dis. From Incoming Pipe to Bottom	1.81	1.638
BL	Bottom Level	n/a	1.638
DP_{GL}	Discharge Point Ground level		19.90

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS	
(For Structural Design Reference)	
4.25	**HHWL**, m
3.75	**HWL**, m
6.05	**Volume** $_W$ cu m
3.20	**LWL**, m
2.90	**LLWL**, m
1.26	**d**, m
1.81	**D + d**, m
3.96	**Depth** $_{TOTAL}$

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	Project:	SihanoukVille_SANITATION	Revision No.:	
	Designed by: F.S.ABRIGO	Date: 4-Oct-16 / Checked: - / Date: -		

SHEET TITLE: ***B-5.PS_Sihanoukville, SUMP/PUMP SIZING AT 2,156 cu.M/day (2030)***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 2,156.0 m^3/day = 24.95 lps
1.497 m^3/min = 89.8 m^3/hr
= 395.6 gpm
No. of Pumps = 1 + 1 **Stand-By**
Recommended Pump Cycle maximum = 10 cycle/hour
= 6 mins.
minimum = 6 cycle/hour
= 10 mins.
Design Velocity
at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,
For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,
at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)
Length of Rectangular Sump = 2 m
Width of Rectangular Sump = 2 m
Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	2.64	ft.	K Factor
Piping	0.0483		6.00
Check Valves	5.063		3.00
Ball Valves	0.253		0.15
Elbows	0.304		0.18
Tee	1.013		0.60
Total H_L	**6.68**	ft.	
	2.04	m	

Headloss for Valves & Fittings $= k(v^2/2g)$
where: $(v^2/2g)$ = 0.51
Headloss for Straight Pipe = k*(L/100)
Length of Force main, m = 2,249.00
Diameter of Force main, mm = 0.7
K Factor = 0.01
Total Headloss at Force Main: **0.7377**

C. DESIGN CALCULATION

Q per pump = 24.95 lps
= 89.83 m^3/hr
= 395.57 gpm
Volume Required: For Maximum 10 cyc/hour = 2.25 m^3
For Minimum 6 cyc/hour = 3.74 m^3
Volume Required: *6.05* ***m³***
Wet Well Volume Required: *USE:* **2.20** ***m³***
Height of The Sump: Rectangular = 0.55 m
Circular = *0.70* m
USE: **Rectangular** = **0.55** **m**
Riser Pipe Diameter Required: = 0.095 m say **0.1** **m**

Volume, $_{Req}$ **=(Φ*q)/4** M^3
where:
Φ =minimum time in minutes, min
q =pump capacity, m^3/min

Water in Sump Vol. $_{Total}$ = **2.2**
Water in Pipe Vol. $_{Total}$ = **3.85**
(Incoming)

T D H$_{Required}$ = PWL + Discharge Head + Headloss$_{Total}$
where: PWL= HWL-LWL = 0.55
Discharge Head = $(PS_{GL}\text{-}HWL)+(DP\ el\text{-}PS_{GL})+HL_{FM}$ = 16
Total Headloss= = 2.8
Total Dynamic Head$_{Required}$ = **19.17** m say = **20** m
Pump Efficiency = 66.0%
Power Required = 11.90 hp say **15** **hp**
= 11.19 KW say **12** **kW**

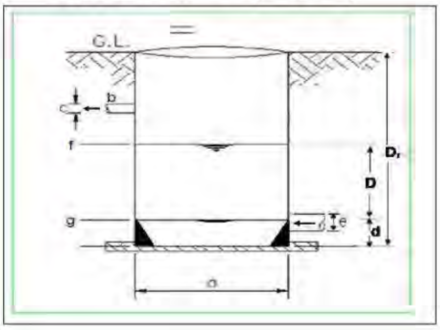
Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%
*			

Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min. Water Level Installation = **1,198.0** **m**

To Check for **Submergence** = **S=D*(1 + 2.3F)**
Where: **F= v/(gD)$^{1/2}$** = **3.208**
V= vel. @ suction bell, m/s = 3.177
D= O.D of Suction bell, m = 0.1
Submergence= 0.838 **m**

FOR PUMP's DISCHARGE VELOCITY
$Q_{Required}$ = 0.0250 m^3/s
Pipe Area = 0.0079 m^2
Velocity = **3.18** m/s < 3.5 m/s **'PASSED'**
FOR SUCTION VELOCITY:
V= vel.@ suction bell, m/s = **3.18**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION , m**
PS_{GL}	Ground Level	n/a	6.000
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	4.800
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	2.00	
e	Incoming Pipe Diameter	0.7	4.200
	I.L of Incoming Pipe		3.500
f	Pump Start, **(HWL)**		4.750
g	Pump Stop Level **(LWL)**	0.55	4.200
	Pump Min. Water Level Reqt. **(d)**	0.84	
h	Wet well Depth	1.39	
i	Dis. From Incoming Pipe to Bottom	1.39	3.062
BL	Bottom Level	n/a	3.062
DP_{GL}	Discharge Point Ground level		19.90

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS (For Structural Design Reference)	
5.25	**HHWL**, m
4.75	**HWL**, m
6.05	**Volume** $_{W,}$ cu.m
4.20	**LWL**, m
3.90	**LLWL**, m
0.84	**d**, m
1.39	**D + d**, m
2.94	**Depth** $_{TOTAL}$

Contract No.:				Job No.:	-
				Sheet No.:	1 of 1
Project:	SihanoukVille_SANITATION			Revision No.:	
Designed by:	Date:	Checked	Date:		
F.S.ABRIGO	5-Oct-16	-	-		

SHEET TITLE: ***B-5.PS_Sihanoukville, SUMP/PUMP SIZING AT 2,156 cu.M/day (2040)***

A. REFERENCES/BOOKS/STANDARDS

A.1. Pumping Station Design Handbook Second Edition

B. DESIGN BASIS

Q_{total} @ peak flow = 2,992.0 m^3/day = 34.63 lps

2.078 m^3/min = 124.7 m^3/hr

= 548.9 gpm

No. of Pumps = 1 + 1 Stand-By

Recommended Pump Cycle maximum = 10 cycle/hour

= 6 mins.

minimum = 6 cycle/hour

= 10 mins.

Design Velocity

at Suction: For Pump Entrance of larger than = 3.6 m/s 200 mm dia.,

For Pump Entrance of smaller than = 5.5 m/s 200 mm dia.,

at Discharge: Should be Less Than = 3.5 m/s (As per recommendation)

Length of Rectangular Sump = 2 m

Width of Rectangular Sump = 2 m

Diameter of Circular Sump = 2 m

Total Friction Loss in Pipes & Fittings

(Pipe length)	2.92	ft.	K Factor
Piping	0.0535		6.00
Check Valves	1.926		3.00
Ball Valves	0.096		0.15
Elbows	0.116		0.18
Tee	0.385		0.60
Total H_L	**2.58**	ft.	
	0.79	m	

Headloss for Valves & Fittings $= k(v^2/2g)$

where: $(v^2/2g)$ = 0.2

Headloss for Straight Pipe $= k*(L/100)$

Length of Force main, m = 2,249.00

Diameter of Force main, mm = 0.8

K Factor = 0.01

Total Headloss at Force Main: 0.7377

C. DESIGN CALCULATION

Q per pump = 34.63 lps

= 124.67 m^3/hr

= 548.95 gpm

Volume Required: For Maximum 10 cyc/hour = 3.12 m^3

For Minimum 6 cyc/hour = 5.19 m^3

Volume Required: *6.05* ***m^3***

Wet Well Volume Required: *USE:* **2.20** ***m^3***

Height of The Sump: Rectangular = 0.55 m

Circular = *0.70* m

USE: **Rectangular** = **0.55** **m**

Riser Pipe Diameter Required: = 0.112 m say **0.15** **m**

Volume, $_{Req}$ **=(Φ*q)/4** M^3

where:

Φ =minimum time in minutes, min.

q =pump capacity, m^3/min

Water in Sump Vol.$_{Total}$ = **2.2**

Water in Pipe Vol.$_{Total}$ = **3.85**

(Incoming)

T D H$_{Required}$ = PWL + Discharge Head + Headloss$_{Total}$

where: PWL= HWL-LWL = 0.55

Discharge Head = $(PS_{GL}$-HWL)+(DP el.-PS_{GL})+HL_{FM} = 16

Total Headloss= = 1.5

Total Dynamic Head$_{Required}$ = **18.02** m say = **19** m

Pump Efficiency = 66.0%

Power Required = 15.52 hp say **20** **hp**

= 14.92 KW say **15** **kW**

Pump Brand/Model	Q, m^3/min	TDH, m	Eff.,%
*			

Choose: *

Note: Referring From The Pump's Manufacturer's Data, Pump's Min Water Level Installation = **1,198.0** **m**

To Check for **Submergence** = **S=D*(1 + 2.3F)**

Where: **F= v/(gD)$^{1/2}$** = **4.452**

V= vel. @ suction bell, m/s = 4.409

D= O.D of Suction bell, m = 0.1

Submergence= 1.124 **m**

FOR PUMP's DISCHARGE VELOCITY

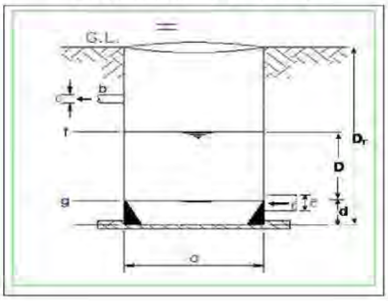
Q $_{Required}$ = 0.0346 m^3/s

Pipe Area = 0.0177 m^2

Velocity = **1.96** m/s < 3.5 m/s **'PASSED'**

FOR SUCTION VELOCITY:

V= vel.@ suction bell, m/s = **4.41**



SUMP DETAILS			
ITEM	**DESCRIPTIONS**	**DIMENSION, m**	**ELEVATION , m**
PS_{GL}	Ground Level	n/a	6.000
a	Tank Dimension (Dia.)	2 x 2	n/a
b	Pipe Discharge Elevation	1	4.800
c	Pipe Discharge Diameter	0.2	
	Ht. of Incoming Pipe to Discharge	2.00	
e	Incoming Pipe Diameter	0.7	4.200
	I.L of Incoming Pipe		3.500
f	Pump Start. **(HWL)**		4.750
g	Pump Stop Level **(LWL)**	0.55	4.200
	Pump Min. Water Level Reqt. **(d)**	1.12	
h	Wet well Depth	1.67	
i	Dis. From Incoming Pipe to Bottom	1.67	2.776
BL	Bottom Level	n/a	2.776
DP_{GL}	Discharge Point Ground level		19.90

Note: Additional submergence for LLWL, m = 0.3

WET WELL DETAILS (For Structural Design Reference)	
5.25	**HHWL**, m
4.75	**HWL**, m
6.05	**Volume** $_W$ cu.m
4.20	**LWL**, m
3.90	**LLWL**, m
1.12	**d**, m
1.67	**D + d**, m
3.22	**Depth** $_{TOTAL}$

Appendix 5
Preliminary Design Report – Siem Reap Sewer Subproject

Appendix H3 Preliminary Design Report –Siem Reap Sewer Subproject

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1 Documents Reviewed

- MTDP - Siem Reap WW Mgmt ADB RRP, Nov 2002
- MTDP - Siem Reap WW Mgmt Inception report, May 2005
- MTDP - Siem Reap WW Mgmt “Basis of Design” report, July 2005
- MTDP - Siem Reap WW Mgmt “Final Design” report, March 2006
- MTDP - Siem Reap WW Mgmt Interceptor sewer As-Built drawings, October 2009
- MTDP - Siem Reap WW Mgmt “Report on Sewer Investigation”, March 2015

- MTDP - ADB draft Project Completion report, June 2011
- MTDP - ADB Project Completion report, August 2011
- MTDP - ADB Validation report, December 2013

- Korean Siem Reap WW Final design report, March 2011
- Korean Siem Reap WW Completion report, September 2014

- Siem Reap Urban Development Project, Wastewater Masterplan (NJS) – June 2010
- Siem Reap Urban Development Project, Stormwater drainage (NJS) – June 2010

2 History of Interceptor Sewer

The design and installation of the interceptor sewer was completed under the Siem Reap Wastewater Management (SRWM) subproject (1 of 4) under the ADB Mekong Tourism Development Project between 2006 and 2009, with construction 2007-09 and commissioning in 2010.

The interceptor sewer consists of 632m of 600mm GRP pipe and 3,043m of 700mm GRP pipe, starting at the upstream end at National Road #6m running north to south down Sivatha St, then turning southwest onto Wat Chork St, crossing the Ring Road, and ending at a pump station. From this pump station wastewater is delivered to the WWTP, also constructed under the SRWM project.
The pipe invert at the upstream end was at 3m depth (600mm dia) and at the downstream end reached its maximum IL depth at 8.3m (700mm dia). The grade of the 632m of 600mm pipe was 0.237% and the downstream 3,043m of 700mm pipe 0.187%.

The original SRWM subproject, of which the interceptor sewer was a part, had the objective of collecting wastewater from central Siem Reap (2.4km2 area), transporting it via the interceptor sewer to a pump station (ADB-PS) from which it was delivered to a new WWTP of capacity 2,776m3/day. This WWTP was intended to serve only the central area of Siem Reap. The 600mm interceptor portion has a capacity of 25,800m3/day and the 700mm interceptor 30,070m3/day. The capacity of the ADB pump station at the downstream end of the interceptor sewer was 20,750m3/day with 3 (Aurora brand) duty pumps. The design horizon for this project was 2020. This is summarised in Table 1.

The original SRWM subproject cost $14.4M, priced in 2006. Of this, $9M was for the interceptor sewer work. Using historical construction cost indices (for SE Asia) of 0.67 between 2006-2016, the $9M 2006 cost is $15M equivalent in 2016.

Between 2011 and 2014, the road surface above the pipeline failed in 5 locations. In the furthest upstream failure the top of the pipe deformed inwards at the top and developed a leak at the top, but did not break. In the four downstream cases the pipe failure was a typical compression failure with cracks at 3,6,9 and 12 o’clock and complete collapse at these 4 road failure locations. Three of these collapse locations have been fully repaired, the upstream deformity has had a temporary repair and backfill, and the fifth failure point around 500m upstream of the pump station has not been repaired due to lack of budget. The pipe remains closed at this location. The interceptor sewer is therefore not functioning. Further failures could happen at any time, and there may be many more deformed areas of pipe that have not been detected yet as there is no road collapse evident. Failure along any part of this pipe means that there is no way for wastewater to reach the WWTP, and all of it is discharged via overflow weirs to the town drainage.

Table 1 - Capacities of infrastructure completed under SRWM (2008)

Infrastructure	Design (max) capacity (m3/day)
600mm sewer	25,800
700mm sewer	30,070
Pump station (ADB-PS)	20,750
WWTP	2,776

3 Further Wastewater Work Completed since Commissioning of the interceptor sewer

Following commissioning of the SRWM subproject in 2010, further work has been carried out under a Korean funded project.

The WWTP was increased to 8,000m3/day under the Korean project, which currently serves the original (ADB) central area, western and eastern areas. Two new pump stations were installed, one east and one west, both pumping to the (now failed) ADB interceptor sewer before being pumped by the ADB pump station to the WWTP. Two further Grundfos pumps were added to supplement the ADB funded Aurora pumps in the Interceptor sewer pump station.

The western “Korean” subcatchment is delivered into the head of the interceptor sewer, whilst the eastern “Korean” subcatchment is delivered into a manhole on the 700mm diameter section of the pipe at Manhole 13 at the intersection of St.7 and St.11 with Sivatha St. These will require reconnection once the proposed new interceptor sewer is installed.

Further land is available to duplicate the WWTP to 16,000m3/day as a second stage, which is the capacity required to serve the central and western (i.e. eastern not included) areas of Siem Reap to 2030. If both eastern and western areas are served then the current WWTP will reach 16,000m3/day capacity in 2018 – and the proposed second stage of another 8,000m3/day has not been constructed as yet.

The current 8,000m3/day WWTP can therefore only treat about 50% of wastewater generated at the current time (2016). The two pump stations constructed under the Korean project and the WWTP extension have not been able to be commissioned yet due to the failed interceptor sewer.

The current WWTP, even when extended to 16,000m3/day, will not be of sufficient capacity to also treat the eastern side of the city to 2030. As has been suggested by the Korean design report and by the Director of DPWT, a separate eastern WWTP will be required to serve the eastern zone. Once this is constructed, flow from the eastern pump station that currently delivers WW to the interceptor sewer will need to be diverted to this new eastern WWTP.

However, advice from MPWT is that, with the current high and increasing cost of land it is unlikely that land will be purchased for this proposed eastern WWTP, and that treatment for the whole city will have to be limited within the boundary of the current 32ha.

An Indian funded team have recently carried out a feasibility study for WW treatment of the eastern side of Siem Reap but there is no solid proposal to date. Whether there is a solid approved proposal to carry through with providing both an 8,000m3/day extension to the current WWTP, and a separate WWTP to serve the eastern zone will have an impact on the sizing of the proposed interceptor sewer replacement.

Figure 1 shows a layout showing wastewater infrastructure put in place under the previous ADB and Korean projects. Figure 2 shows the current WWTP layout and land available for expansion.

Figure 1 - Current WW infrastructure in Siem Reap

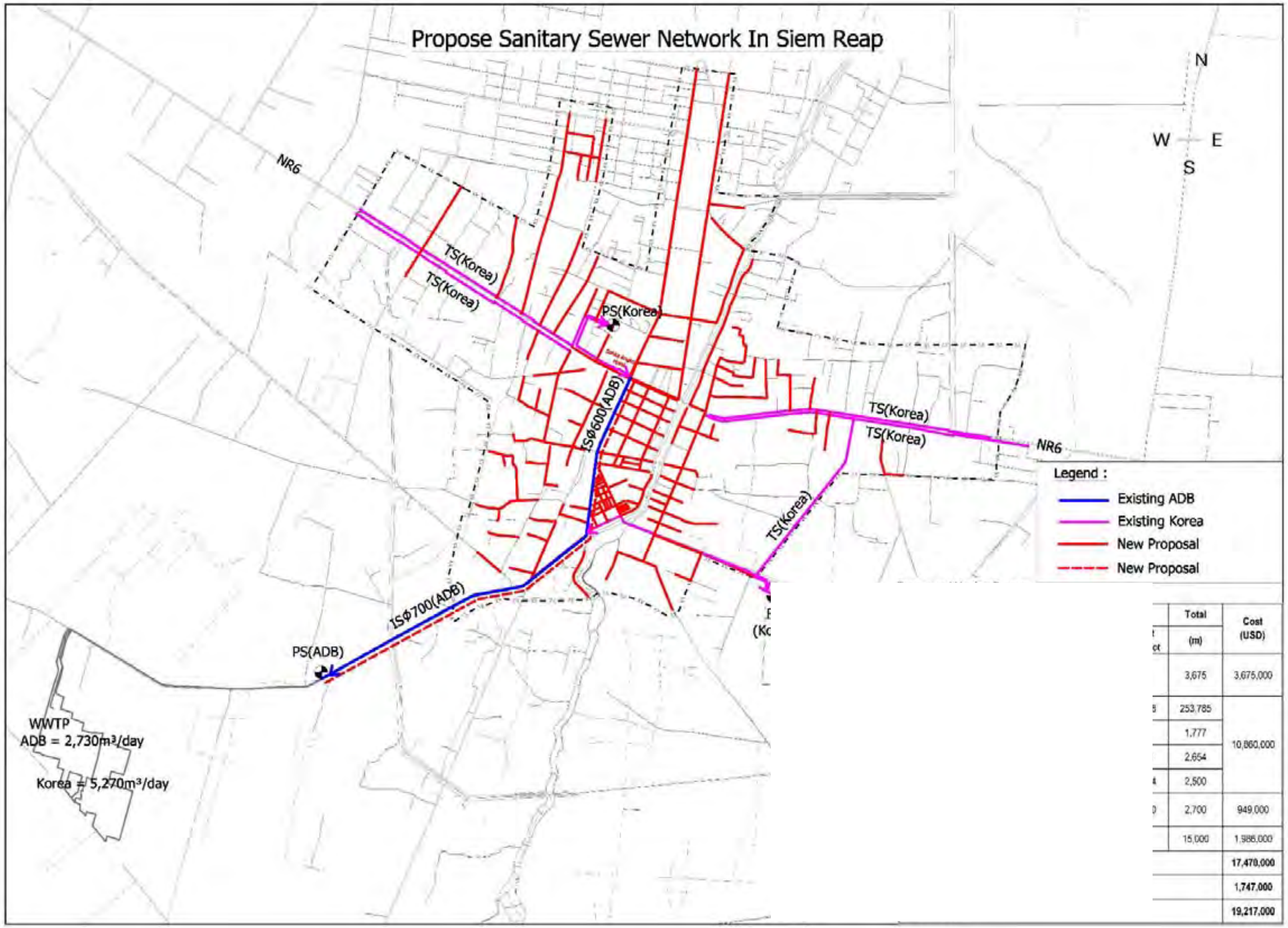


Figure 2 - WWTP layout

4 Options for Replacing Interceptor Sewer

There are three main options for replacing the 3.7km length of interceptor sewer. All options will include increasing the capacity from the current 600/700mm and using a stronger pipe material such as ductile cast iron;

Option 1: Same deep alignment as existing, using trenchless technology for one 1000mm diameter DCI pipe

Option 2: Shallower alignment with multiple (3-4) in-line pump stations, using open cut trenching for one 1000mm diameter DCI pipe.

Option 3: Same deep alignment as existing, using open cut trenching for one 1000mm diameter DCI pipe.

Pipe sizes above are suggested sizes only and will need to be confirmed during detail design.

Option 3 has been discounted due to the serious problems experienced on the original project with dewatering, proper compaction, operating in a deep trench, necessary road closures, disruption and mess in the main tourist area and economic losses from affected businesses and associated tax revenue to the government.

During the initial 2006 design, a cost comparison was made between deep trenching for a gravity-flow pipe against shallow trenching with multiple pump stations, and the outcome was that the lifetime cost effectiveness was better for trenching without pumping up to a depth of 10m.

Currently, if the interceptor was in working condition, the flow would be limited to 25,800m^3/day which is the maximum capacity of the 600mm pipe. This is sufficient for the central and western areas up to and beyond 2030 (a wastewater flow of 16,000m^3/day is projected for these areas for 2030). However, as the pipe needs to be deep due to the flat topography of Siem Reap, is expensive and difficult to construct, it should be sized for a much longer lifetime – a 50 year life to 2070 is suggested. The wastewater generation estimate for the east and west service areas by 2070 based on a 2015 generation of 14,157m^3/day and a growth rate of 3% is **71,950m^3/day**. If the eastern area is disconnected from the trunk sewer and diverted to a new separate WWTP as proposed by the Korean project design, then the required 2070 wastewater capacity of the proposed trunk sewer replacement will reduce to **47,000m^3/day**. A 1000mm diameter pipe will be able to take this flow.

4.1 Option 1: Trenchless technology options

There are 2 main trenchless technology methods that could be applied.

4.1.1 Pipejacking and microtunneling

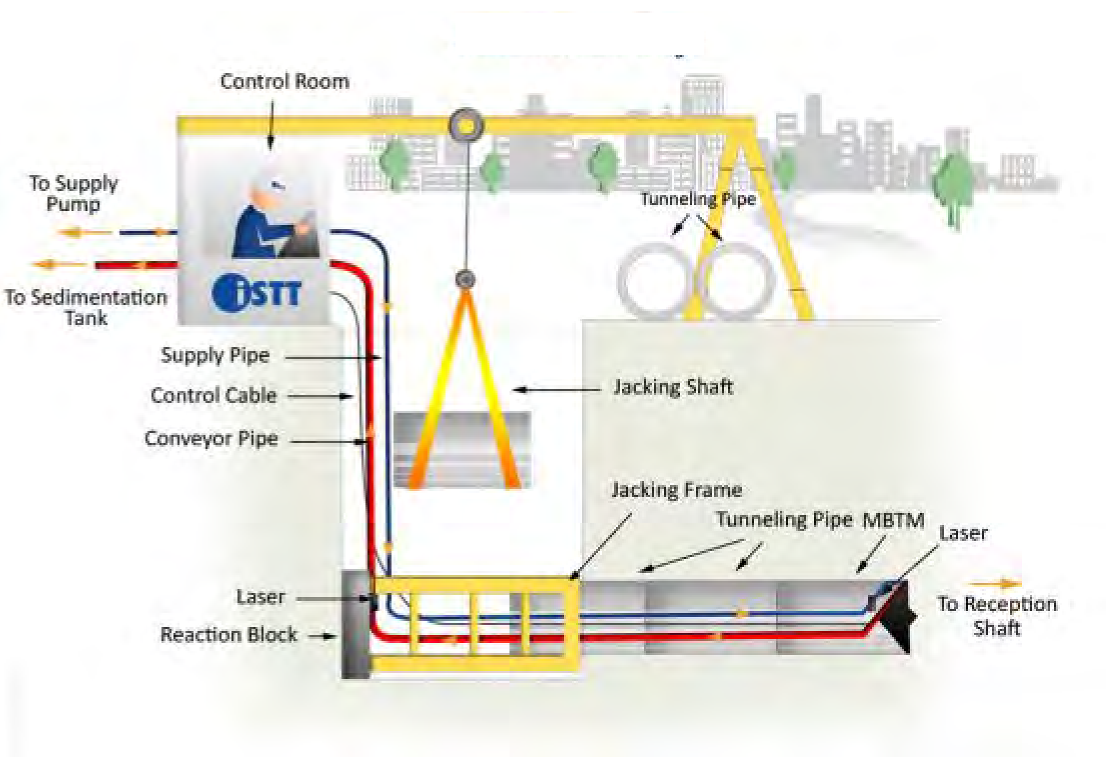
Pipejacking was originally designed for sewer construction by the Japanese in the 1970's. High tolerances for alignment and grade are achievable and diameters up to 3m or more are possible.

A pipejacking boring machine or "tunnel boring machine (TBM)"is set up in a *thrust shaft* at the start of the alignment and pushed into the earth by hydraulic jacks to the *reception shaft*. A well anchored *thrust wall* of sufficient mass is required to thrust against. Shafts are usually located at manhole

positions. The pipe to be installed is then inserted between the jack and boring machine and pushed into the tunnel. Friction between the boring machine and pipeline, and surrounding earth, are reduced by pumping bentonite slurry into the tunnel between pipe and earth outside.
The tunnelling shield at the front of the operation removes earth for the pipe to be jacked through. The shield can be manual for workers inside the tunnel to manually excavate, but it is more common in the present day for the shield to be mechanical and not require men inside the pipe.
As this application will be below the water table, a head wall and seal will be required in the thrust pit and receiving pit in order to keep groundwater out, and to retain lubricating fluids.

Microtunneling is a type of pipe jacking, defined as a remotely-controlled, guided, pipe-jacking operation that provides continuous support to the excavation face by applying mechanical or fluid pressure to balance groundwater and earth pressures. Support at the excavation face is a key feature of microtunneling, distinguishing it from traditional open-shield pipe-jacking. Microtunneling is normally used for smaller diameter pipes.

Figure 3 - Typical example of microtunneling



4.1.2 Pipe bursting

This is a commonly used trenchless technology method, first developed in the UK in the 1970's for gas main replacement. Pipe bursting fractures an existing pipe and displaces the fragments outwards whilst a new pipe the same size or larger is drawn in behind to replace the old pipe. The bursting is achieved through the insertion of a conically shaped "bursting head" tool into the old pipe. The bursting head base is larger than the pipe diameter being replaced, and slightly larger than the new pipe diameter and the rear of the bursting head is attached to the new pipe. The front of the bursting head is connected to a cable or rod, which pulls the head through from an insertion pit to a reception pit. There are two main categories of pipe bursting, pneumatic and static.

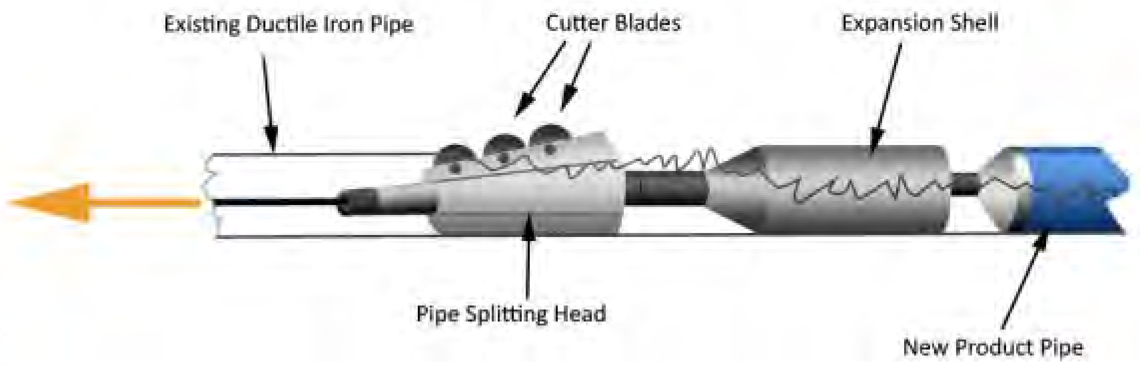
With pneumatic pipe bursting, the bursting head is a pneumatic hammer driven by compressed air. The expander head is positioned just behind the hammer, and both are kept under constant tension from a winch in the receiving pit. As the pneumatic hammer acts, it pulls the expander further into the old pipe, breaking it and pushing the pieces and earth outwards, making space for the new larger pipe that is being pulled in behind.

With static pipe bursting, there is no hammer action to assist in busting the old pipe, but instead the bursting force comes from being pulled by rods or a cable inserted through the existing pipe. More force is available using rods, but if a cable is used the operation is continuous as the operator does not have to keep stopping to remove retracted rod sections in the receiving pit. In the proposed instance, bursting out a 600mm to a 1000mm diameter, much force will be required so it is likely that rods will be required.

Pipe bursting is generally done for replacement pipes between 50mm up to about 1000mm diameter. The majority of pipe bursting is employed for upsizing from 150 to 200mm, 200 to 250mm or 250 to 300mm, but larger sizes are feasible. Larger diameters than 1000mm could in theory be accomplished but are generally not cost effective. Pipe bursting alone is only applicable when the pipe to be burst is of a brittle material such as concrete, PVC, cast iron and clay. Materials with more structural strength like HDPE, GRP and ductile iron cannot be replaced with pipe bursting alone. For these materials a “pipe splitter” must be used, which is a different type of head to the bursting head but is pulled through the pipe in a similar manner, as shown in Figure 4. Pipe splitting - roller blade cutting wheels attached to the bursting head - can be used in conjunction with static pipe bursting but not with pneumatic pipe bursting methodologies.

In pipe bursting or splitting, the replacement pipe can be larger than the existing pipe but there are limits to how much bigger it can be. A general rule is that the replacement pipe can be up to three pipe sizes larger. An experienced trenchless technology company based in Singapore contacted about this Siem Reap proposal commented that the maximum size increase from 700mm they can do is 1000mm.

Figure 4 - Typical pipe splitting equipment



4.1.3 Advantages of pipe bursting or pipe jacking over open-cut

Table 2 describes the benefits of pipe bursting over open-cut. Most of these advantages also apply to pipe jacking.

Table 2 - Benefits of pipe bursting over open-cut

Direct costs	Indirect costs
Lower overall costs	Less time
Less dewatering	Less impact to businesses & residents
Utilises existing utility corridor	Less disturbance to traffic
Less material removed & replaced	Smaller work footprint
Less equipment & labour	Less emissions – green benefits

4.1.4 Pipe material & characteristics

The pipe material must be durable for the long design life proposed, such as ductile cast iron. This must be protected with a liner to avoid corrosion, particularly under the anaerobic conditions caused by sulphide generation in the wastewater. For protection from corrosion from wastewater a high alumina content cement lining is recommended.

The pipe grade should be sufficient to give a self-cleaning flow velocity, although the pipe will be flushed by wet season flows, and will have manholes every 100m for further flushing or jetting.

4.1.5 Soil displacement

Ground displacement can be a problem depending on pipe depth, degree of upsizing and soil conditions. This can potentially cause heaving at the surface or damage to other nearby underground services, particularly with installing larger diameter pipes.

The action of pipe bursting with a larger diameter pipe displaces the surrounding earth in all directions, with more displacement upwards then in the other planes. Whether this has an impact on the surface, in this case a road, depends on the degree of pipe enlargement (for example a small increase from 600mm to 700mm or a larger increase from 600mm to 1000mm), the soil conditions, and the depth. This will need further investigation during final design.

The upstream end of the pipe alignment is the shallowest, starting at 3m to invert, with the downstream end at the ADB pump station being 8.3m to invert. The upstream end of the pipeline along Sivatha Street is therefore more susceptible to road surface damage from soil displacement. This section of road has a relatively new concrete seal so an allowance has been made for reinstatement in the budget cost estimate.

Downstream from the junction of Sivatha Street and Wat Chork Street, where the pipe depth to invert is between 5.8 – 8.3m, the current road condition is poor and would benefit from rehabilitation regardless, should the surface be damaged from ground displacement.

4.1.6 Maintenance

With the upstream and downstream levels fixed, the grade of the interceptor sewer will be around 0.2% which is steeper than the generally accepted minimum self-cleaning grade for large diameter sewers. For self-cleaning, a velocity of 0.6-1m/s is usually quoted, which can be achieved from a grade of 0.05-0.08% for 1000mm diameter pipe and using Manning's formula.

However, this is based on the pipe being half full, which may not always be the case in the first years of its operation. In case of blockages or fat build up, accessible manholes are provided every 100m along the length of the pipe which will enable any section of pipe with a blockage to be flushed from a water truck.

4.2 Shallower open cut sewer with multiple pump stations

By splitting the 3,700m gravity sewer into sections with lift pump stations between these sections, the sewer can be kept shallower. The pipe is still a gravity sewer but the downstream end of each section discharges into a sump where it is pumped back up to the shallower upstream end of the next section. There is no cost advantage in using trenchless methods to do this compared to Option 1 so this option is examined based on open-cut installation only. The cost of trenchless installation is almost the same whether operating at 8m or 5m, the only small difference being deeper entry and exit pits to excavate and protect.

Normally a pumped sewer system is only selected over a gravity sewer if the lifecycle cost over a long period (e.g. 40 years in the UK) is lower.
The grade of the existing failed trunk sewer is 0.237% for the initial 600mm diameter section and 0.187% for the downstream 700mm section. Published minimum self-cleaning grade guidelines (based on Manning's n=0.013 and velocity=0.76m/s) allow for a minimum slope of 0.12% for 600mm diameter and 0.11% for 700mm diameter. For this exercise a more conservative 0.2% slope has been adopted.
A sample of three pumping scenarios have been examined and long sections (based on current failed alignment) shown below in Figures 5-7.

The open cut excavation for these options all start at an invert level of 3m depth below ground level. This is to allow 2m cover above the 1000mm diameter pipe which should be sufficient to lay underneath any existing services. At the downstream end the pipe invert levels are lowered to around 5m below ground level. Trench shields will be required during construction. During the wet season the water table is approximately 1m below ground, and during dry season 2m below, so constant dewatering will be required during construction. This was a major problem during the laying of the current GRP pipe in 2005/06, and was possibly a factor in the ultimate failure of the sewer due to the inability to properly compact the saturated bedding material.

4.3 Geotechnical investigation

Under the original pipelaying for the current 600mm and 700mm interceptor sewer a full geotech investigation was carried out[1]. A the alignment will be identical for Option 1 and very close for Option 2, this document is still valid.

[1] Soil Investigation Report, Mekong Toursim Development Project part A1, Sakor Cambodia Co. Ltd, October 2006.

Figure 5 - Three interim pump stations evenly spaced over 3.7km

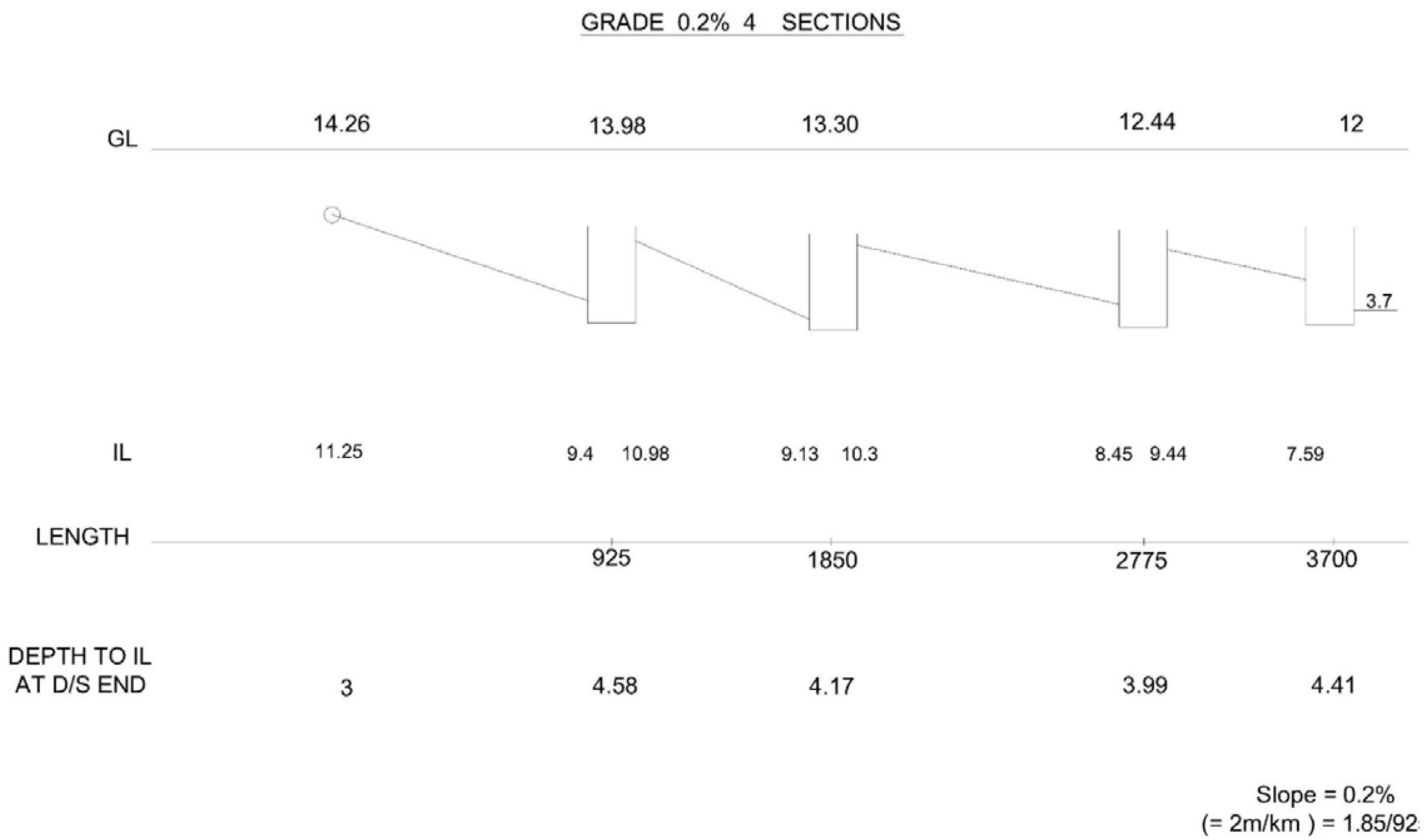


Figure 6 - Three interim pump stations located at 20%, 40% and 80% of 3.7km pipe length

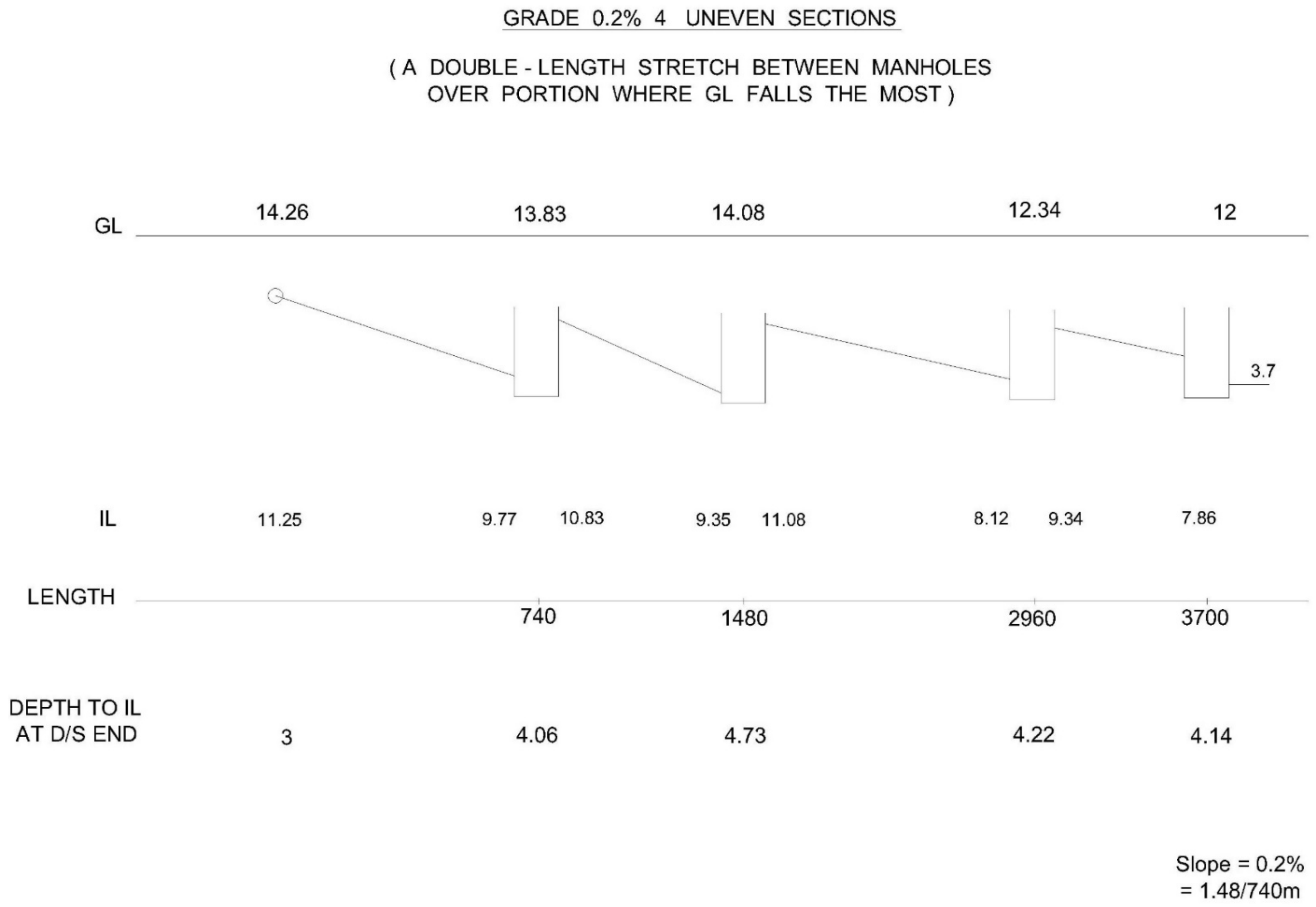
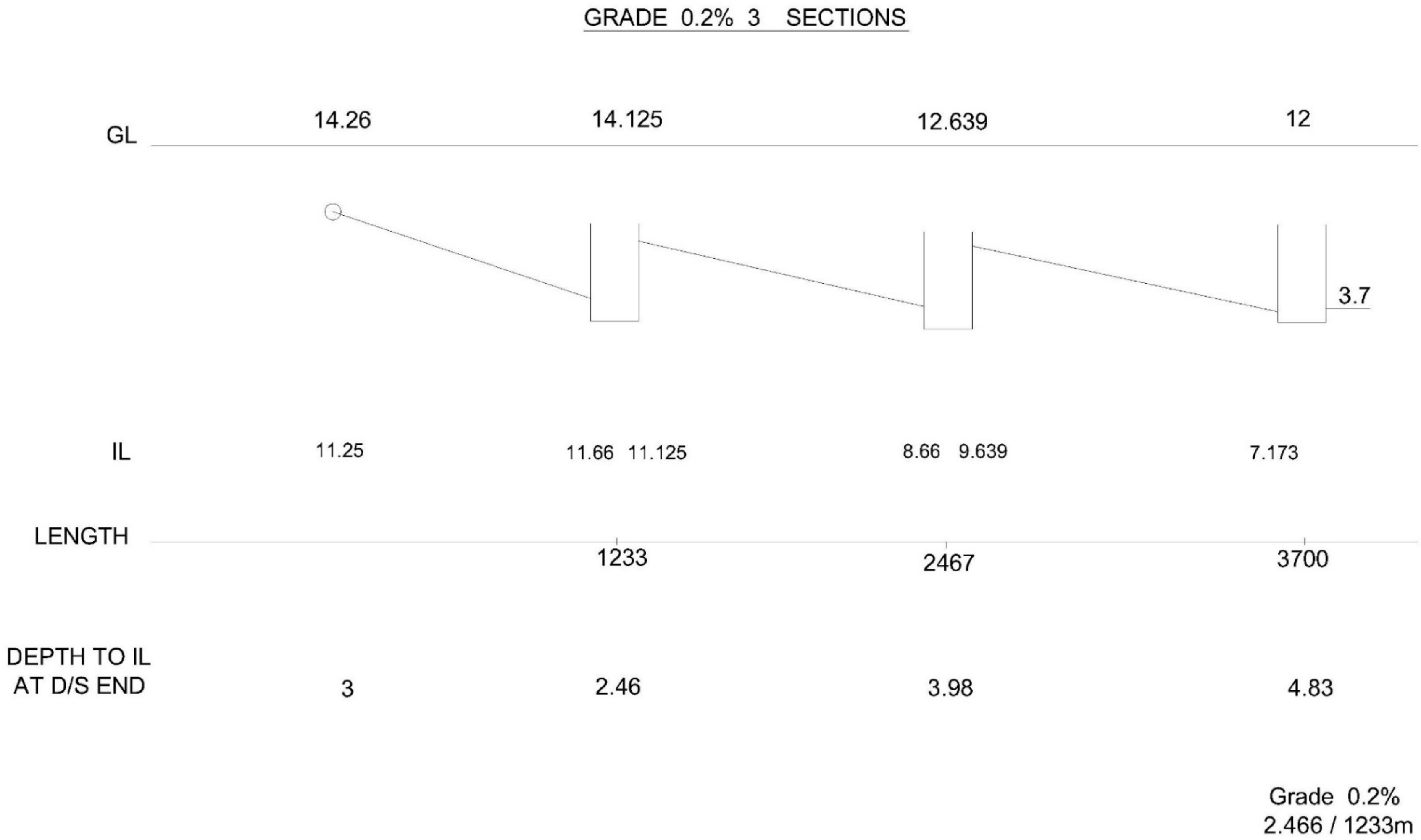


Figure 7 - Two interim pump stations evenly spaced over 3.7km



5 Preliminary Cost

Two options have been costed, one following the existing alignment using pipe bursting/splitting, and the second a shallower alignment using open-cut with 3 pump stations.

5.1 Pipe bursting/splitting

A quote has been received from a Singapore based company for $2000/m for the pipe splitting service alone. All transport, civil works and materials (pipe) are additional. As pipe bursting is rarely carried out at diameters of 1000m, some difficulty was experienced in getting a wider range of cost estimates for the service, following enquiries to trenchless technology companies in Thailand, Vietnam, China and the USA[2].

For the DCI pipe supply, quotes were received from several Chinese manufacturers, all of which were similar at around $320/m for plain 1000mm diameter DCI pipe and $520/m for a high alumina cement lining and the restrained joints required for use with pipe bursting or jacking. Chinese pipe was quoted as class K9 and being to ISO2531. An American quote was received for $957/m again including restrained joints and high alumina cement lining and delivery to ship (FOB).

For the purposes of the preliminary cost estimate a price estimate an intermediate price of $750/m has been used. An approximate capital cost estimate is provided in Table 3. Should pipejacking be selected as the preferred methodology, the cost will likely be an increment higher than the cost for pipe bursting to reflect the hire cost of more sophisticated mechanical and electrical equipment.

[2] Companies were generally not willing to provide quotes or cost estimates outside of a formal bidding process, probably due to it being a small and competitive market, and costs varying widely based on factors such as geology, access, availability of existing service plans, local auxiliary equipment hire etc.

Table 3 - Capital cost estimate for option 1 - Pipe splitting to install a 1000mm diameter DCI pipe

Item	Qty	Unit rate (USD)	Cost (USD)
Pipe splitting	3,700m	2,000	7,400,000
Delivery of equipment	1	50,000	50,000
Alumina cement lined DI pipe 1000mm dia inc. restrained joints, nut & bolts.	3,700m	7503	2,775,000
Shipping China-Sihanoukville	103 containers	5,000	515,000
DI pipe delivery (120 containers S'ville - SR)	120	1,000	120,000
Port fees, clearance	1	100,000	100,000
Entry/exit pits 10mx4m excavation & shoring up to 9m deep	37	5,000	1,850,000
Manholes (same location as pits) inc. landings	37	30000	1,110,000
Road reinstatement for pits (4x10m ea)	37	60	88,800
Reinstatement for possible road haunching of concrete road	1285 X 3m wide = 3,855m2	60	231,300
Dewatering in pits	1	LS	50,000
Health & safety, and public access	1	LS	50,000
Effluent testing equipment	1	LS	25,000
Extension to laboratory building	1	LS	100,000
Subtotal			14,465,100
Contingency @ 10%			1,446,510
Total			**15,911,610**

[3] Based on 3 Chinese quotes and one US quote.

5.2 Shallower open-cut with 3 intermediate pump stations

An approximate capital cost estimate is provided in Table 4.

Table 4 - Capital cost estimate for option 2 – Shallower open-cut alignment with 3 intermediate pump stations

Item	Qty	Unit rate	Cost (USD)
Excavation	4x2x3700=29,600m3	50	1,480,000
Carting of excavated earth to storage area offsite and returning for backfill	11,000 x 8m3 truckloads	50	550,000
Alumina cement lined DI pipe 1000mm dia inc. restrained joints, nut & bolts, delivered to Sihanoukville	3700m	750	2,775,000
Shipping China-Sihanoukville	103 containers	5,000	515,000
DI pipe delivery (120 containers S'ville - SR)	120	1,000	120,000
Port fees, clearance	1	100,000	100,000
Pipe installation/bedding/backfill	3700	400	1,480,000
Pump stations	3	300,000	900,000
Road reinstatement	3700m x 4m width	60/m2	880,000
Manholes	37	30,000	1,110,000
Dewatering	1	LS	100,000
Effluent testing equipment	1	LS	25,000
Extension to laboratory building	1	LS	100,000
Health & safety, and public access	1	LS	100,000
Subtotal			10,235,000
Contingency 10%			1,023,500
Total			**11,258,500**

In addition to capital costs this option will have a significant operating cost from the power consumption at the three pump stations. Whilst the pumping head is low, the flow volumes are high. Approximate annual power costs, based on 18 hours per day pumping, are shown in Table 5 below.

Table 5 - Estimated annual power costs for pumps in Option 2

Pump station	Pump rating	Power tariff ($/kW/hr)	Annual power cost
1:H=4m, Q=50,000m3/d	37	0.25	60,772.5
1:H=4m, Q=50,000m3/d	37	0.25	60,772.5
1:H=4m, Q=50,000m3/d	37	0.25	60,772.5
Total			**182,318**

As an approximate illustrative exercise, the marginally lower cost estimate of the pumped option will increase beyond the trenchless option cost estimate after 5.5 years of operation based on power costs alone, without considering other costs caused by disruption to business, loss of tax revenue and resettlement payments.

6 Future WWTP Capacity Requirements & Costs

6.1 Future WWTP capacity required

The existing WWTP has capacity 8,000m^3/d. It was built in 2 stages, first a 2,776m^3/d plant built under the initial ADB project (commissioned 2010) after which it was expanded to a total of 8,000m^3/d under a Korean project, completed in 2014. The ADB plant worked for a short while until the interceptor sewer failed and the Korean plant has never been commissioned. The ADB project service area was the central part of the western catchment, and the Korean service area the remainder of the western catchment plus the eastern catchment. Figure 8 below shows these catchments.

The Korean design for "Siem Reap Sewage System & Improvement Siem Reap River" (Final Design Report, Kunhwa Consulting, March 2011) allowed for treatment for the western side only up to 2030, and made the assumptions that:

- the eastern area of the city would have a dedicated 9,000m^3/d WWTP constructed.
- A second 8,000m^3/d WWTP would be built next to the current one by 2020

We now know that it is likely that the construction of the eastern WWTP will not occur, and that all treatment for the entire city will need to be on the current 40ha site (32ha government land and 8ha APSARA land).

The current and projected future wastewater generation are shown in Table 6.

Table 6 - Wastewater generation projections

Description	2010	2015	2020	2025	2030	2035	2040
East	3,915	4,907	6,049	7,379	8,909	10,602	12,510
West	7,484	9,250	11,310	13,716	16,526	19,666	23,206
Total	11,399	14,157	17,359	21,095	25,435	30,268	**35,716**

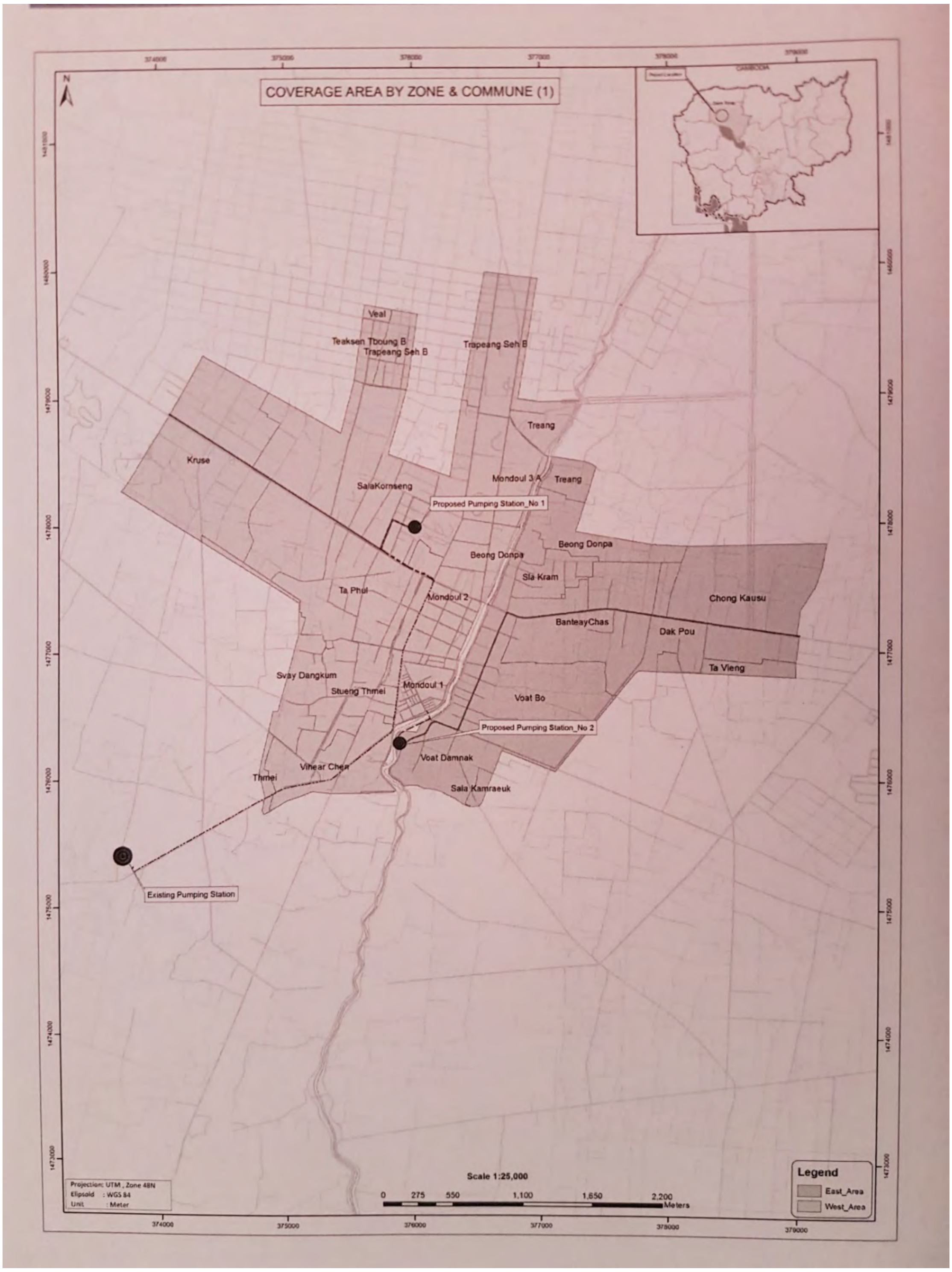
These figures are based on projections from 2011 and should be recalculated using more recent population and commercial property data prior to any final decision on the capacity of a future WWTP. From these figures it can be seen that the current capacity of 8,000m^3/d needs increasing to 36,000m^3/d to serve the city to 2040.
It can be seen that the existing 8,000m^3/d WWTP will be too small to meet expected WW generation before it is even commissioned. It is also reasonable to assume that a separate WWTP for the eastern area will not be constructed due to land prices[4]. Acquiring large amounts of single-site land is difficult in Siem Reap and current land costs in the area of the existing WWTP are in the region of \$30/m^2. This land is low and is flooded in the wet season. If the remaining empty 20ha of government land adjacent to the current 8,000m^3/d WWTP is utilised for the same technology then only a total capacity of 16,000m^3/d will be possible to serve the whole city, and clearly this will not be sufficient. The remaining land therefore needs developing with a technology that has a smaller footprint for the

[4] Based on discussions with MPWT.

same or more treatment capacity, but not a technology too advanced for what is still a fledgling wastewater sector.

An evaluation of suitable appropriate technologies has been carried out for the proposed Sihanoukville and Battambang WWTP's under this CDIA TA, and the most appropriate selection from that exercise are conventional trickling filters. Conventional trickling filters require pre-settlement which can be carried out in a pair of future ponds, and the filters themselves will easily be accommodated within the available area. From the work carried out for Battambang WWTP, a conventional trickling filter of depth 2m and diameter 46m is required to treat approximately 8,000m^3/day. Four of these will therefore be required to meet 2040 wastewater production.

Figure 8 - Eastern and western catchments



6.2 Cost estimate for WWTP expansion

Table 7 shows a preliminary cost estimate for a conventional trickling filter WWTP of total capacity 32,000m^3/d to supplement the existing 8,000m^3/d plant. It does not include any household connections or work outside the WWTP grounds.

Table 7 - Preliminary cost estimate for WWTP expansion

Item	Unit	Unit rate	Qty	Total ($)
Preparatory earthworks	LS	100,000	1	100000
Excavate for anaerobic ponds (2/3 below GL)	m3	10	21333	213,333
Compacted bunds around lagoons 2m high x 2m thick at base	m3	30	624	18,720
Compacted bunds around site 2m high x 2m thick at base	m3	30	6400	192,000
Liners	m2	5	6600	33,000
DCI Pipework, valves & meters inside WWTP	LS	200,000	1	200,000
Conventional trickling filter diameter 46m, 8,185m3/day	ea	1,250,000	4	5,000,000
Site access road from main road 6m wide	m	100	1000	100,000
Site roads 4m wide (gravel)	m	50	1850	92,500
Concrete pad and effluent pipe for dewatering container	LS	5,000	1	5,000
Training/capacity building	LS	150,000	1	150,000
Subtotal				**6,104,553**
10% contingency				610,455
TOTAL				**6,715,009**

7 Siem Reap DPWT Comments

A meeting was held 16 May 2016 with the Director and Wastewater engineers of the DPWT in Siem Reap. They had several comments about the previous project and suggestions on how to improve any future reconstruction project. These were;

- The GRP pipe used not only failed at 12,3,6,9 longitudinal fracture locations but also deformed at the top under soil weight. They believe the pipe was non-specification.
- DPWT want a professional contractor with proven experience in deep pipe laying.
- A 5-10 year defects liability period is required if possible.
- There is no firm plan to have a new WWTP to serve the eastern city and whilst this is their firm preference, one is not expected to be built and commissioned within the next 10 years. The Interceptor sewer should therefore be sized to take all of the town wastewater for the first 10 years, and only the central and west areas after the initial 10 years.
- The current manhole covers are not lockable and flap open during heavy rain. Lockable covers are required.
- Manhole entry equipment and confined spaces training is required.
- 1200mm diameter manhole covers are requested – the current ones are 900mm.
- Pump station – the US Aurora pumps have no regional representation and spares are expensive. One failed after 3 years and mechanical seals need replacing often. Pumps with local representation are requested in future.
- The existing main ADB pump station is 20m underground and has no sump pit at the inlet to collect sand and grit, which builds up inside the chamber. A sump pit is required.
- The ADB pump station inlet chamber (12m deep) has a grating over the top which allows smells to disturb neighbours. A sealed inlet chamber cover is required.
- The ADB pump station inlet conveyor garbage rake is vertical and garbage, once ensnared, drops off as the rake rotates. It should be set back at an angle.

8 Recommendation

The preferred construction method is pipe bursting with pipe splitting to replace the existing 600/700mm diameter GRP pipe with 1000mm lined DCI pipe. Pipe splitting has an advantage over pipe jacking in that the same existing alignment can be used and the amount of excavated spoil requiring disposal offsite is less. Otherwise, pipe jacking is also a good option and is of a similar cost.

The capital cost estimates, whilst preliminary at this stage, show the trenchless option to be around 18% more expensive than shallower open cut with 3 pump stations. This difference is mainly in hiring the pipe bursting equipment and operator team. However, the lifetime cost of the trenchless option is significantly less due to having no ongoing power costs, as illustrated in Table 3 above. The trenchless option also has several practical advantages, being;

- Pipe is deeper, providing more scope for further expansion areas to connect in the future.
- The pipe is a gravity sewer allowing for direct connections at regularly spaced manholes
- Less space is needed for construction
- Large areas in the road for the underground pump stations will be unpractical.
- Far less mess and disturbance is made during the construction period (the pipe passes through the main tourist centre of Siem Reap and construction here would cause much commercial loss for business)
- Whilst the pumped option is shallower than the option using pipe splitting, it is still below the water table and so would still encounter the same dewatering problems as experienced during the original project.

However it is recognised that large diameter pipe bursting is a specialised activity and that the relatively small number of regional companies involved in this method of trenchless technology may cause problems in securing a contractor in the timeframe desired. The Design-Build contract will therefore leave the methodology open to other trenchless technology methods such as pipejacking, both in order to maximise the number of potential interested bidders and take advantage of the expertise of these companies in putting forward alternative solutions.

An extension to the existing laboratory and some further effluent testing equipment has also been requested by DPWT and included in the proposal.

The cost estimate for the interceptor sewer replacement by trenchless technology is $15,911,610.

Appendix 6
Preliminary Design Report – Kampong Cham Septage Management Subproject

Appendix H6 Preliminary Design Report –Kampong Cham Septage Management Subproject

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1 Introduction and Current Situation

Kampong Cham city is the capital of Kampong Cham Province and is on National Road 6 approximately 125km from Phnom Penh, about half way between Phnom Penh and Kratie. Kampong Cham is the third largest city in Cambodia after Phnom Penh and Battambang, but is the most populous Province. Rubber is the primary industry in the Province. Kampong Cham is a flat area on the Mekong flood plain close to the centre of Cambodia. There are no hills in the area.

Kampong Cham is currently at the beginning of its wastewater treatment development. There are an unknown number of working septic tanks in the city centre, with a small number of private vacuum trucks which empty these tanks on demand and dispose of to surrounding agricultural land. Figure 1 shows toilet type in the city centre, but this does not give an indication of the number of true septic tanks associated with each pour-flush or closet (press-flush) toilet. There is no reticulated wastewater collection other than direct or indirect disposal to road drains by households and businesses. Wastewater runs to the Mekong through a series of combined road drains.

As a first step in the development of a sanitation sector it is proposed to examine the feasibility of a septage treatment facility for the town.

Figure 1 - Sanitation facilities by type

Source: Socio-economic survey, July 2013

2 Summary of Proposed Subproject Activities

It is proposed to identify suitable land and construct a septage treatment facility, which can be used by the vacuum trucks that service septic tanks to safely dispose of the septic sludge (septage) generated in the tanks. This septage has already undergone a degree of anaerobic decomposition inside the septic tanks but requires further treatment before it can be dried and disposed of to the environment.

The two simplest and most appropriate methods of treating septage are anaerobic treatment in a lagoon followed by drying of the treated sludge, and treatment in a constructed wetland. A constructed, managed wetland requires large amounts of land and therefore has not been considered further for Kampong Cham.

The treatment process selected for development utilises two facultative lagoons in parallel. Each lagoon is sized to contain one years' volume of septage for a selected future design year, plus one year's rainfall minus evaporation. In the initial years of operation that capacity will therefore be longer than one year. The lagoons are lined with compacted clay and are able to be decanted into a finishing or maturation lagoon or percolation pond. As the water table is known to be very high in Kampong Cham, a finishing lagoon is applicable.

The lagoons are bunded to prevent wet season flood intrusion, and have an access point where vacuum trucks can back up to the lagoon and empty their tanks. One lagoon is used at a time, such that when one lagoon is approaching half full of settled, digested sludge, the other lagoon is then used. The lagoon switch-over should always occur at the end of the wet season, so that the "full" lagoon has 5-6 months to dry sufficiently so that the sludge can be handled. The lagoons will also have 3 or more valved outlets at different levels so that the upper levels of settled liquid can be decanted off to the finishing lagoon, in order to speed up sludge drying.

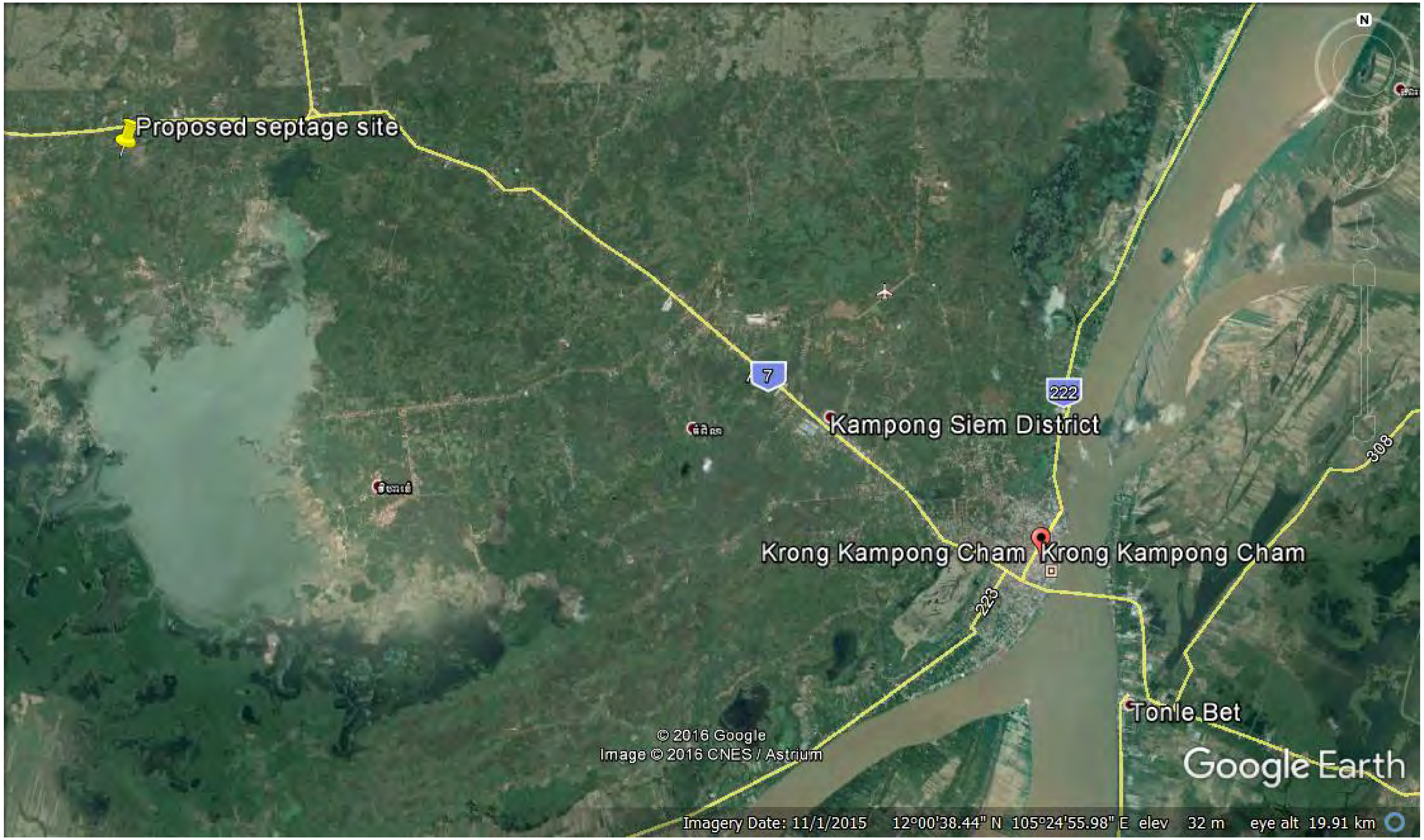
Once sufficiently dried for handling, the treated sludge can be removed from the lagoon using a compact excavator and truck, and either dewatered further using a dewatering container, deposited directly into sludge drying beds, or carried directly to landfill for ultimate disposal.

The subproject proposes to supply equipment to the DPWT for use with the septage treatment plant;

- 6m^3 vacuum truck
- Dewatering container (basic model)
- Compact excavator

The subproject will also include capacity building for DPWT and private vacuum truck operators and a public awareness campaign aimed at increased understanding of septic tanks and the need for maintenance.

Figure 2 - Shows the currently proposed location for the facility



3 Proposed Septage Treatment Facility- Design

3.1 Advantages of lagoons for septage treatment

- Low capital and operating cost
- Best option when land is cheaply available
- Appropriate technology and low level of required training in O&M
- Good level of stabilization for septage
- Many variations on design can be employed
- Primary lagoons are lined
- Effluent to be fed to a maturation/finishing lagoon or percolation pond
- Dried sludge can be composted or landfilled

3.2 Lagoon size

The primary lagoons are sized to hold one year's sludge for a particular design year, the sludge volume being calculated from the population projection to that year. Assuming commissioning in 2020, a 20 year design capacity will serve the population through to 2040. Table 1 shows required lagoon volumes for various design years.

Table 1 - Required primary lagoon volume

Year	2016	2025	2035	2040
Population	41,400	47,060	66,634	73,021
Septic/WC coverage(%)	30	30	30	30
Population with septic tank	12,420	14,118	19,990	21906
Sludge per person (m3)	0.1	0.1	0.1	0.1
Domestic sludge (m3/yr)	1,242	1,412	1,999	2191
Commercial sludge (m3/yr)	124	141	200	219.1
Total sludge (m3)	1,366	1,553	2,199	2410
Total sludge rounded (m3)	1,400	1,550	2,200	2,400

The sludge quantity from septic tanks used in the above calculation refers to the settled sludge. In practice, when a septic tank is emptied this settled sludge is stirred up and mixed with the partially clarified liquid above it. As the lagoon settles out as it is filled over a year or more, this liquid can be drawn off through the multi-level effluent pipes to the maturation lagoon.

3.3 Design Elements

The key design elements for the lagoon based septage treatment facility are summarized in Table 2. An example layout for the facility is provided in Figure 3.

Table 2 - Summary of design elements:

Element	Features
Topo & geotech survey	2ha site, 5 test pits
Site layout	Best fit
Receiving well design	For ease of use by vacuum trucks To minimise odour Provide wash water facility
Lagoon design	1 year storage, clay lined primary lagoon, 3m deep
Effluent outlet design	3 vertically staged outlets to assist clarified effluent drawdown
Maturation pond design	1 year storage, lined, 1.5m deep
Site roads, Operator office, fencing, water supply, cleaning equipment	Road access to all working areas
Main access road	4m wide, sealed

3.3.1 Receiving well design

The receiving well is the position that the vacuum trucks back up to in order to empty their load of septage into the primary lagoon. Because the area is flat, and the lagoons must be bunded to isolate them from the surrounding flood plain, there will be a ramp up to the receiving well at the top of the bund. The receiving well should have a bar screen to prevent solids from entering the lagoons (sanitary pads etc) and a source of pressurized water for washing down the vehicle and concrete pad surrounding the well. The inlet pipe into the lagoon is normally beneath the lagoon liquid level to minimize odours and spillage.

There are many advanced receiving well design examples but a simple, non-mechanised design is recommended for Kampong Cham.

3.3.2 Primary Lagoons

The objective of the primary lagoons is liquid/solid separation with anaerobic and aearobic decomposition of the solids at the bottom of the lagoon. In the early life stages of the lagoon the liquid level will be shallow and the lagoon will act aerobically only. After several years of operation when the lagoon in use approaches full, the clarified liquid may be decanted off to the maturation lagoon. This is done by a series of valved outlet pipes at different levels so that the upper layers of settling liquid can be drawn off without disturbing lower levels.

One primary lagoon is used to deposit septage whilst the other is drying out. At the end of each wet season, the lagoon that has been in use has the settled liquid level decanted off to the maturation lagoon and left to dry out. Once the sludge reaches a consistency that can be handled, it can be moved from the lagoon using an excavator and placed into the dewatering container to thicken it further. This container resembles a shipping container and has a geotextile filter material inside, which captures the sludge solids but allows the clear liquid to run out the bottom. The liquid can be fed back to the maturation pond. Further sun-drying can occur in the sludge drying beds as necessary before disposal to landfill.

The primary lagoons need to be lined, preferably with compacted clay if this material is available locally. When the excavator is removing sludge it must be careful not to remove right down to the clay layer to avoid damaging it. A compacted ramp of minimum grade 1:5 up to the top of the bund and down into the lagoon serves as access for vacuum trucks for filling, and also for an excavator and truck for desludging.

The primary lagoons are sized for 1 year of sludge production for the peak design year of 2040, which amount to 2,400m^3 rounded up to 3,000m^3.This is based on an estimate of 30% of the city having a septic tank, assumes a sludge generation of 0.1m^3/person/year[1], and assumes that households empty their septic tanks every 3-5 years or whenever the sludge depth reaches 30% of tank depth. The lagoons are bunded with a 1:1 bund slope to ground level and the suggested dimensions at the deepest area are 40m x 25m x 3m depth.

3.3.3 Maturation pond

The maturation or finishing pond is intended to provide further anaerobic decomposition to the settled liquids drawn off from the primary lagoons. This pond has a larger surface area than the primary lagoons but is 1.5m deep to allow oxygen diffusion. This pond is also lined with compacted clay and has an access ramp up to the top of the bund and down into the pond for cleaning access.

[1] A generally accepted international quantity

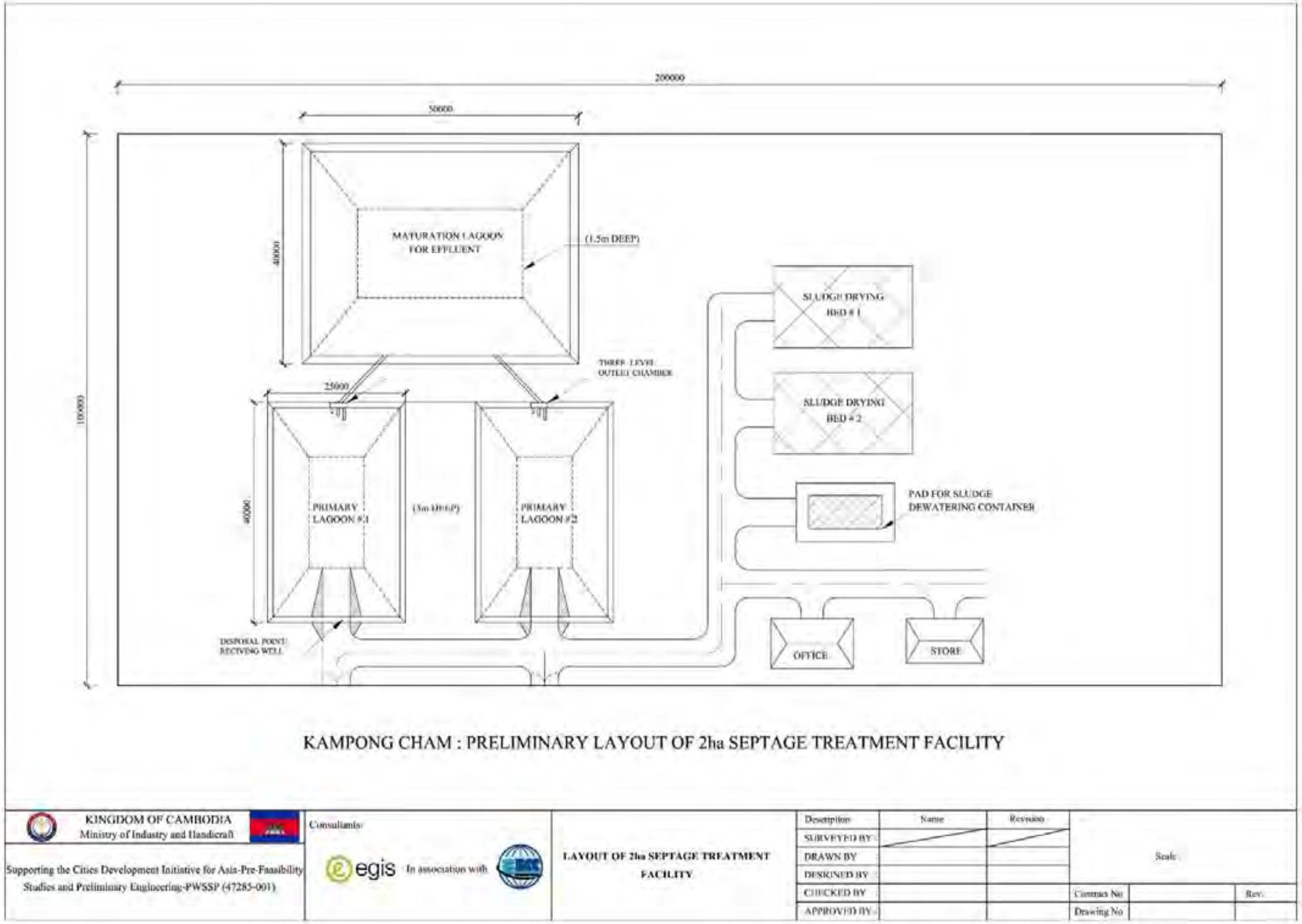
3.4 O&M

The operation of the facility requires that at least one staff operator is present during daylight hours. The main activity required is the desludging of a lagoon at the end of a dry season or once it has dried out sufficiently for handling. This will require several men, for operating the excavator, truck, and distributing sludge into drying beds and/or dewatering container.

Table 3 - Summary of expected O&M tasks

O&M Activity	Frequency
Logging septage delivery trucks	Daily
Fee collection	Per entry or by monthly account
Water blasting of septage receiving area	Weekly
Removal of dried sludge	One week every 1-2 years

Figure 3 - Example layout for septage treatment facility



3.5 Capacity Building

Capacity building for all DPWT staff involved in the wastewater sector is recommended, to improve understanding of septage treatment from collection to disposal of dried treated sludge.
Capacity building core elements to be developed should focus on the following:

- **Collection** – collection by vacuum truck, monthly or annual license fee for private operators, correct truck emptying into receiving well of anaerobic lagoon or dedicated septage treatment lagoon, maintaining cleanliness of receiving area.
- **Private septage collection operators** – Expectation from DPWT, license fees
- **Treatment in WWTP** – action on septage in anaerobic lagoon, effective desludging, placement of sludge into drying beds
- **Treatment in dedicated Septage Treatment Facility (dual lagoons)** – action on septage in facultative lagoon, drying of lagoon during dry season, removal of dry treated sludge to either landfill or for composting. Lagoon level controls and effluent overflow soakaway.
- **Sludge drying in WWTP** – periodic removal of sludge by pumping to drying beds.
- **Sludge drying in dedicated Septage Treatment Facility** – annual removal of dried treated sludge from non-duty lagoon.
- **Composting** – addition of sawdust or vegetable waste in to increase carbon:nitrogen ratio, turning of compost for aeration, correct moisture content, possible markets and cost recovery, composting bed design.
- **Disposal** – If no market for human waste compost, dried treated sludge may be disposed of to landfill as weekly solid waste cover.

3.6 Equipment supply

It is proposed to supply DPWT with a dewatering container, a compact excavator and one $6m^3$ septage vacuum truck under the contract.

4 Septage Collection

Septage collection, in common with other towns in Cambodia, is currently mainly carried out by the private sector. There are currently only a small number of vacuum truck operators in Kampong Cham, but with the capacity building and public awareness proposed under the subproject this should increase. As operators become busier, carrying more loads per day, the incentive to invest further in another truck will arise, in addition to more interest generated for more business people to become involved in the sector.

Capacity building is proposed for both septage treatment facility operators (DPWT staff) and the private sector. The private sector operators will be trained in logging information (volumes of tanks at each address, frequency of emptying) and also in using the treatment facility correctly. Itt is proposed that they will be required to be licensed, and a condition of that license will be that they must drop their septage load at the facility, and not onto agricultural land or anywhere else. The license will come with an annual fee that will aim to cover the annual O&M costs of the treatment facility.

For the sustainability of the septage treatment facility it is vital to have both the support of the private vacuum truck operators through licensing and the increased awareness of the public in having their septic tanks emptied. Across Cambodia it is currently apparent that householders only empty their septic tanks as a last resort when there is an odour problem due to them being overfilled with sludge. As a general rule, if sized correctly for the number of house occupants, septic tanks should be emptied every 3-5 years or when the tank is 30% full of sludge. The sludge depth can be measured with a dip stick through an inspection hole in the roof.

5 Septic Tanks

The coverage of septic tanks in central Kampong Cham, whilst numbers are not known, is thought to be fairly low, and the size, age and condition of septic tanks is unknown. Many households, even in the central city, have concrete ring soakaway tanks which do not provide any treatment and pollute the water table. A standard design and size guidelines for different house occupancies is given in the Sanitation Sector Development Roadmap as part of the proposed National Sanitation Strategy outline, which was produced under this TA.

In order to establish a firm idea of the number, size and condition of septic tanks in Kampong Cham, which will better inform future decisions on septage treatment and viable numbers of vacuum trucks operating, it is recommended that a septic tank survey be carried out. This survey should consist of a series of carefully thought out and phrased questions such that the possible ambiguity of responses from home owners is minimised, and will also include site measurements by the surveyors. It is recommended at least 20% of households are surveyed, and this will most likely be done by several teams working simultaneously in different areas. Similar surveys carried out in other countries have employed college students during holidays, which has the advantage of increasing their knowledge of household sanitation.

6 Cost Estimate

Table 4 - Cost estimates

Item	Unit	Unit rate	Qty	Total ($)
Preparatory earthworks	LS	10000	1	10000
Excavate for 2x facultative ponds (2/3 below GL) 40mx25mx3m deep	m3	10	11734	117,340
Compacted bunds around lagoons 2m high x 4m thick at base	m3	30	1520	45,600
Liners	m2	5	3300	16,500
Maturation pond excavation 50m x 40m x 1.5m	m3	10	2100	21,000
Compacted bunds around lagoons 2m high x 4m thick at base		30	760	22,800
DCI Pipework, valves inside WWTP	LS	50000	1	50,000
Site access road from main road 6m wide	m	100	200	20,000
Site roads 4m wide (gravel)	m	50	600	30,000
Site office	LS	20000	1	20,000
Sludge drying areas (bunded)	ea	10000	2	20,000
Concrete pad and effluent pipe for dewatering container	LS	5,000	1	5,000
Receiving bays for anaerobic ponds	ea	20,000	2	40,000
Supply compact excavator	ea	50,000	1	50,000
Supply 6m3 vacuum truck	ea	80,000	1	80,000
Supply sludge dewatering container	ea	100,000	1	100,000
Public awareness campaign	LS	50,000	1	50,000
Training/capacity building	LS	20,000	1	20,000
Subtotal				718,240
Contingency				71,870
Total				**790,110**