ANNEX 25
DETAILS OF
PRELIMINARY DESIGN

1. EXPANSION OF KAOLIEO WATER TREATMENT PLANT

Expansion of Kaolieo Water Treatment Plant (40,000 m^3/d)

Ⅰ Expanded Design Capacity

		Expansion				Existing
Planed Daily Maximum Supply Water	:	**40,000** m^3/d				**20,000** m^3/d
Planed Daily Maximum Treated Water	:	**44,000** m^3/d	(Loss:	**10.0**	%)	**22,000** m^3/d
Planed Hourly Maximum Supply Water	:	**52,000** m^3/d	(P.F.:	**1.30**	)	**26,000** m^3/d

Ⅱ Calculation on Demensions of Intake and Treatment Facilities

1 Intake Facility

1) Intake

a) Design Water Level in Mekong River

HWL	+ **171.50**	:	same as existing
LWL	+ **159.50**	:	same as existing
Average	+ **162.04**		

2) Intake Pump

a) Actual and Total Head of Pump

LWL of Mekong River : 159.50 m　　Average W.L. of Mekong River : 162.04 m

W.L. of Receiving Well : 175.00 m

Actual Head of Pump : 15.50 **m** → **Total Head of Pump :** **18.50** **m**

b) Pump Capacity and Pump Unit

Pump Capacity : **Q=** 15.3 m^3/min

Pump Unit : **N=** **3** unit (1 unit Stand-by)

c) Specifications of Pump

Dia **350** **mm** * Cap **15.3** m^3/min * H **18.5** m * **70** Kw * **3** unit

d) Inlet Pipe

DIP-1:	N.Dia.φ **1.000**	m *	Length **20.0**	m *	Section Area	0.7854 m^2
DIP-2:	N.Dia.φ **1.000**	m *	Length **15.0**	m		
DIP-3:	N.Dia.φ **1.000**	m *	Length **5.0**	m		

e) Flashing Piping System

SP: N.Dia.φ **0.300** m * Section Area 0.0707 m^2

2 Raw Water Transmission Pipe

a) Pipe Material

*** Steel Pipe & Ductile Cast Iron Pipe**

b) Demension

DIP: N.Dia.φ **0.700** m * Length **40.0** m * Section Area 0.385 m^2

c) Flow Velocity

V_1= Flow Capacity ／ Section Area ＝ 0.509 ／ 0.385 ＝ 1.323 m/sec

d) Raw Water Flow Meter

Type: Ultrasonic Flow Meter

Diameter: 0.700 m * Section Area 0.385 m^2

Velocity: Flow Capacity ／ Section Area ＝ 0.509 ／ 0.385 ＝ 1.323 m/sec

e) Flow Control Valve

Type: Butterfly Valve

Diameter: 0.700 m * Section Area 0.385 m^2

Velocity: Flow Capacity ／ Section Area ＝ 0.509 ／ 0.385 ＝ 1.323 m/sec

3 Treatment Facilities

1) Receiving Well and Coagulation Basin

a) Type of Structure

*** Reinforced Concrete**

b) Type of Coagulation Method

*** Utilizing the Energy of Broad Crested Rectangular Weir**

c) Dimension

Receiving Well	Width 2.80	m*	Length 5.60	m*	Depth 5.10	m *	W. D. 4.50	m *	1 basin
Coagulation Basin	Width 2.80	m*	Length 2.80	m*	Depth 5.10	m *	W. D. 3.84	m *	1 basin
Weir	Width 2.80	m*	Height 4.22	m *	1 Unit				

d) Capacity of Receiving Well and Coagulation Basin

Receiving Well	V_r=	2.80	m *	5.60	m *	4.50	m =	70.60 m^3
Coagulation Basin	V_f=	2.80	m *	2.80	m *	3.84	m =	30.10 m^3

e) Detention Time of Receiving Well and Coagulation Basin

Receiving Well	T_r=	70.60	m^3 ／	30.56	m^3/min =	2.31 min
Coagulation Basin	T_f=	30.10	m^3 ／	30.56	m^3/min =	0.98 min

f) Perforated Baffle Wall of Receiving Well

* Opening Section Area of Perforated Baffle Wall (m^2) : 6% : asumption

Width 2.80 m* W.D. 4.50 m * 0.06 % = 0.756 m^2

g) Overflow Weir

Weir	Width 1.30	m*	Height 4.60	m *	1 Unit

2) Inlet Channel

a) Type of Structure

* Reinforced Concrete

b) Demension

Junction Channel	Width 1.00	m*	Length 21.00	m*	Depth 2.00	m *	W. D. 1.74
Inlet Weir to Floc	Width 0.90	m*	Height 1.55	m *	1 Unit		

3) Flocculation Basin

a) Type of Structure

* Reinforced Concrete

b) Type of Flocculation Method

* Utilizing of Up and Down Flow Baffle Channel

c) Demension

Width 8.55 m* Length 10.15 m* Depth Up Stream 3.200 Down Stream 4.200 m

m* Length 6.95 m (Except Outlet Zone of Flocculation)

* Water Depth Up Stream 2.74 Down Stream 3.37 m * 4 basin

d) Demension of Up and Down Baffle Channel

*** Detailed Dimension as Shown on the Drawing.**

	Width		Lenghth	
1st Row	0.90	m*	8.55	m
2nd Row	0.95	m*	8.55	m
3rd Row	1.00	m*	8.55	m
4th Row	1.05	m*	8.55	m
5th Row	1.10	m*	8.55	m
6th Row	1.20	m*	8.55	m
	6.20			

* Each Depth of Channel's Wall : 0.15 m

e) Demension of Up and Down Baffle Cell

*** Detailed Dimension as Shown on the Drawing.**

	Width		Lenghth	
1st Cell	0.90	m*	1.30	m
2nd Cell	0.95	m*	1.30	m
3rd Cell	1.00	m*	1.30	m
4th Cell	1.05	m*	1.30	m
5th Cell	1.10	m*	1.30	m
6th Cell	1.20	m*	1.30	m
			7.80	

* Each Depth of Cell'sWall : 0.15 m

f) Weir of Outlet Zone

*** Broad Crested Rectangular Weir**

Width 8.55 m* Height 3.00 m* Length 1.20 m * 1 places/basin * Depth of Weir Wall : 0.15 m

g) Outlet Perforated Baffle Wall from Flocculation Basin

Width 8.55 m* Height 4.05 m* Length 1.50 m * 1 places/basin * Depth of Perforated Wall 0.20 m

*** Opening Section Area of Perforated Baffle Wall (m^2)** 6% : assumption

Width 8.55 m* W.D. 3.84 m * 0.06 % = 1.968 m^2

h) Capacity of Flocculation Basin

C_t= 8.55 m * (6.95 m*1/2*(2.736 + 3.371) m *
+ 3.20 m × 3.84 m) = 286.4 m^3/basin

C_a= 8.55 m * 6.95 m*1/2*(2.736 + 3.371) m * = 181.4 m^3/basin

; exept Outlet zone of Flocculation Basin

i) Detention Time of Flocculation Basin

T_t= 286.40 m^3/unit ／ 7.64 m^3/min = 37.5 min

T_a= 181.40 m^3/unit ／ 7.64 m^3/min = 23.7 min ; exept Outlet zone of Flocculation Basin

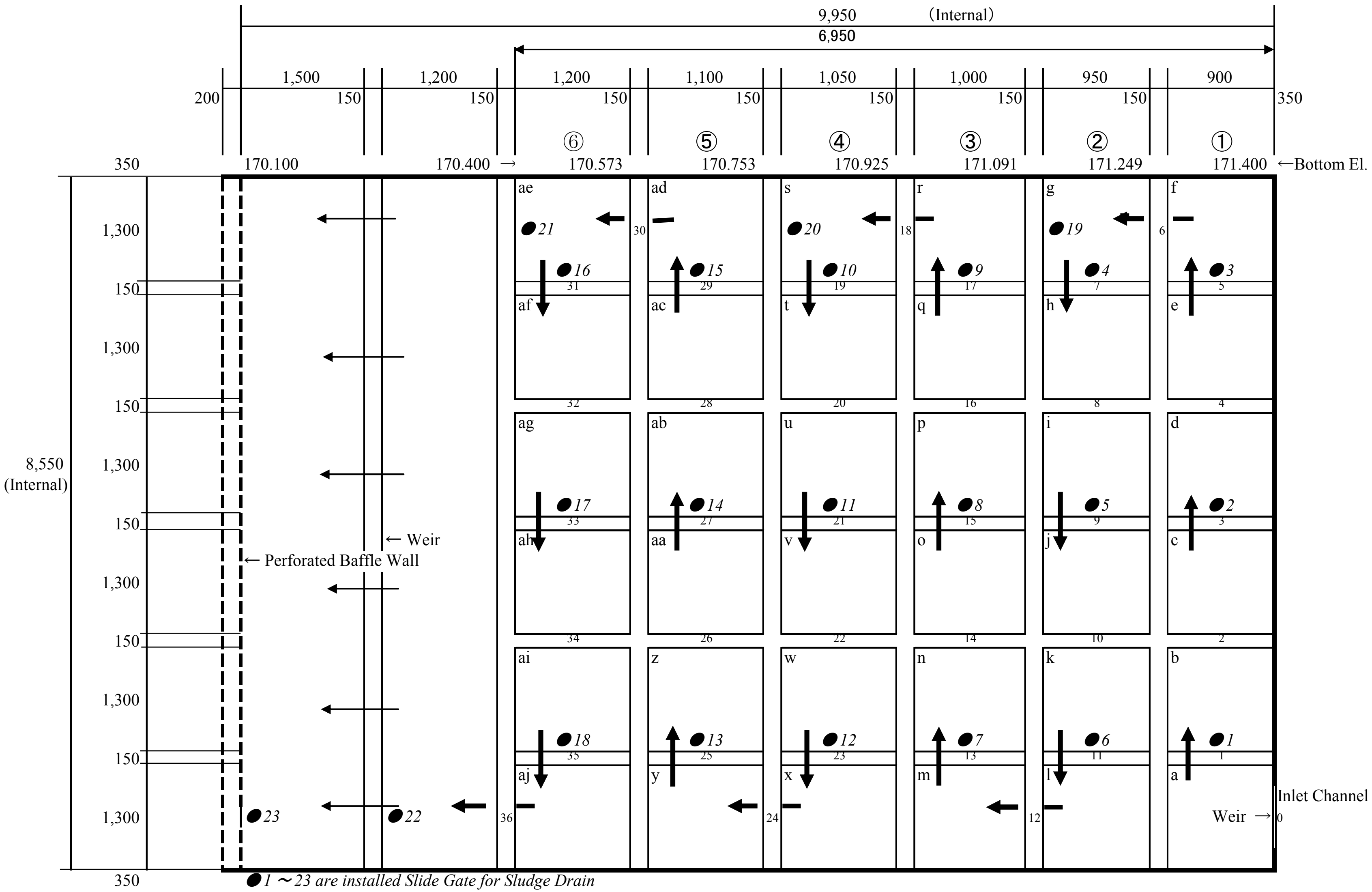
j) Method of Sludge Drain

***Drain out to Flocculation Basin by means of inclination Basement**

*** Sludge Drain Valve**

Dia.φ 0.300 m * 2 places/basin (Outlet Zone of Flocculation Basin)

*** 23 places are installed Slide Gate for Sludge Drain as shown on the drawing as marked "●"**


9,950 (Internal)
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170.573
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Inlet Channel
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●1 ～23 are installed Slide Gate for Sludge Drain

4) Sedimentation Basin

a) Type of Structure

* Reinforced Concrete

b) Type of Sedimentation Method

* Horizontal Flow with Long Launders

c) Demension

Width 8.55 m * Length 33.00 m * Depth: Up Stream 4.700, Down Stream 3.400 m

* Water Depth: Up Stream 3.84, Down Stream 3.04 m * 4 basins

d) Capacity of Sedimentation Basin

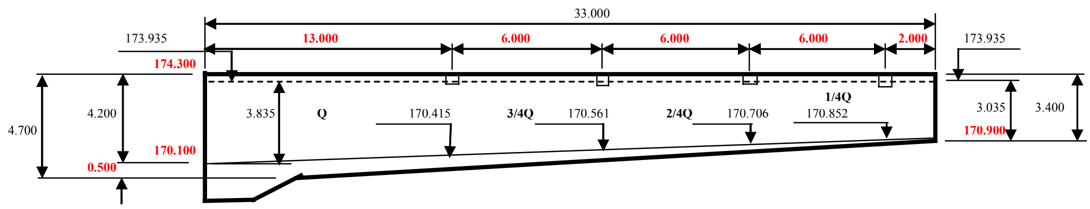
C_t= 8.55 m * 33.00 m *1/2*(3.84 + 3.04) m * = **969.3** m^3/basin

e) Detention Time of Sedimentation Basin

*** Apparent Detention Time**

T_{ta}= 969.3 m^3/unit ／ 458.33 m^3/h = **2.11** h

*** Substantial Detention Time**



***Capacity of Each Section of Sedimentation Basin as shown on the above:**

C_{t1}=	8.55	m *	13.00	m *1/2*(	3.835	+ 3.520) m *	=	**408.8** m^3/section
C_{t2}=	8.55	m *	6.00	m *1/2*(	3.520	+ 3.375) m *	=	**176.9** m^3/section
C_{t3}=	8.55	m *	6.00	m *1/2*(	3.375	+ 3.229) m *	=	**169.4** m^3/section
C_{t4}=	8.55	m *	6.00	m *1/2*(	3.229	+ 3.084) m *	=	**161.9** m^3/section

Therefore, substantial detention time;

T_{ts}= 408.8 m^3/unit ／ 458.33 m^3/h ＋ 176.9 m^3/unit ／ 343.75 m^3/h ＋
169.4 m^3/unit ／ 229.17 m^3/h ＋ 161.9 m^3/unit ／ 114.58 m^3/h ＝ **3.56** h

f) Average Flow Velocity

*** Average Section Area**

A= 8.55 m * 1/2 * (3.84 + 3.04) m = **29.37** m^2/basin

*** Average Flow Velocity**

V = 7.64 m^3/min ／ 29.37 ㎡/basin = **0.26** m/min ; at normal operation 0.4

V_w= 10.19 m^3/min ／ 29.37 ㎡/basin = **0.35** m/min : in case of sedimentation washing/cleanii

g) Surface Loading

*** Surface Area of Sedimentation**

A= 8.55 m * 33.00 m = **282.2** m^2/basin

*** Surface Loading**

S_L= 7.64 m^3/min ／ 282.2 m^2/basin = **27.1** mm/min ; at normal operation 15-25

S_L= 10.19 m^3/min ／ 282.2 m^2/basin = **36.1** mm/min ; in case of sedimentation washing/cleanii

h) Ratio of Width/Length

Ratio of Width/Length = 8.55 ／ 33.00 = 1 ／ 3.9

i) Froude Number (Fr)

Fr= $V_{av}^2 / g \times R$

where, V_{av}; Average flow velocity (m/sec)= 0.00433 m/sec

g; Accelated Gravity (m/sec^2)= 9.81 m/sec^2

R: Hydroulic Radius(m)= A/P = 1.905 m

A = 8.55 m*1/2(3.84 + 3.04) = 29.37 m^2

P = 8.55 m*1/2(3.84 + 3.04)*2 = 15.42 m

Fr= $V_{av}^2 / g \times R$ = **1.005E-06** **OK**

j) Reynolds Number (Re):

Re= $V_{av} \times R / \nu$

Where, V_{av}; Average flow velocity (m/sec)= 0.00433 m/sec

R: Hydroulic Radius (m)= A/P = 1.905 m

ν; Kinematic Viscosity at 25℃ (m^2/s) = **0.898** * 1E-06 m^2/sec

Re= $V_{av} \times R / \nu$ = **9.193E+03** **OK**

k) Intermediate Perforated Baffle Wall

*** Opening Section Area of Intermediate Perforated Baffle Wall (** **6%** : asumption

Width 8.55 m*W.D. **3.52** m * 0.06 % = 1.806 m^2

l) Outlet Launder

***Transverse Type**

Width **0.30** m*Length **3.825** m * Depth **0.20** m * **8** line/basin

***Transverse Type**

Width **0.60** m*Length **20.00** m * Depth Up Stream **0.200** Down Stream **0.400** m * **1** line/basin

m) Weir Loading

L_W= 11,000 m^3/d ／ 61.20 m = 179.7 m^3/d/m

n) Sludge Drain Valve

***Drain out to Sedimentation Basin by means of inclination Basement**

Dia.φ **0.300** m * **2** places/basin

5) Filter Basin

a) Type of Structure
* Reinforced Concrete

b) Type of Filtered Method
* Coarse Sand High-Rate Filters

c) Demension

Width 4.70 m * Length 10.50 m * Depth 3.63 m * 6 basins

d) Filter Area of the Basin

Width 4.70 m * Length 10.50 m = 49.35 m²/basin

e) Filtration Rate

V =	7,333	m³/d/basin ／	49.35	m²/basin =	148.6 m/d	:at normal operation
V_w =	8,800	m³/d/basin ／	49.35	m²/basin =	178.3 m/d	:in case of filter washing/cleaning

f) Depth of Sand and Water Depth above Sand

Depth of Sand : D_s = 1.000 m
Water Depth above Sand : H_{wd} = 0.700 m

g) Specification of Sand

* Effective Size	1.00	mm
* Coefficient of Uniformity	1.40	: less than

h) Underdrain Devices

* Supporting Beam	0.690 m	}
* Perforated Pipe	0.360 m	}
* Honeycomb Slab	0.060 m	} Total 1.200 m
* Diffusing Graval	0.020 m	}
* Porous Concrete	0.070 m	}

i) Wash Water Gutter

Width 1.00 m * Length 10.50 m * Depth Up Stream 0.580 Down Stream 1.580 m * 7 places

j) Method of Washing for Filter

* Rate of Air Blower;	0.8～1.0 m³/min/m	→	1.00 m³/min/m	}→	5 min
* Rate of Back Wash Water with Air;	0.3～0.5 m³/min/m	→	0.36 m³/min/m		
* Rate of Back Wash Water;	0.3～0.5 m³/min/m	→	0.40 m³/min/m	?	10 min

k) Back Wash Pump

Dia.φ 400 * Q 1,184 m³/hr (19.74) m³/min * H 6 m* 30 kw
～ 2 units(1 unit Standby)

Back Wash Rate: 0.400 m³/min/m²

l) Air Blower

Dia.φ 600 * Q 2,961 m³/hr (49.35) m³/min * H 0.3 kg/cm²* 45 kw
～ 2 units(1 unit Standby)

Air Blow Rate: 1.000 m³/min/m²

m) Inlet Gate

Width 0.60 m * Height 0.30 m * 2 place/basin   Section Area= 0.360 m²/basin

Flow Velocity through gates:

V =	0.085	m³/sec/2-gates ／	0.360	m²/basin =	0.200 m/sec	:at normal operation
V_w =	0.102	m³/sec/2-gates ／	0.360	m²/basin =	0.300 m/sec	:in case of filter washing/cleaning

n) Filtered Pipe/Filtered Flow Control Valve

*** Filtered Pipe** 0.5 0.4

SP; N.Dia.φ 0.400 m* N.Dia.φ 0.300 m * m 1 place/basin Section Area= 0.126 ㎡/basin

*** Flow Velocity through Filtered Pipe:**

V = 0.085 m^3/sec/basin ／ 0.126 ㎡/basin = 0.700 m/sec :at normal operation

V_w = 0.102 m^3/sec/basin ／ 0.126 ㎡/basin = 0.800 m/sec :in case of filter washing/cleaning

*** Filtered Flow Control Valve**

Type : Volvoset Diameter: B300/400

Operator: Pneumatic Drive Capacity: 7,333 m^3/d

o) Back Wash Pipe

SP; N.Dia.φ 0.500 m * 1 place/basin Section Area= 0.196 ㎡/basin

Flow Velocity through Back Wash Pipe (Filtered Pipe):

V = 0.329 m^3/sec/basin ／ 0.196 ㎡/basin = 1.700 m/sec

p) Air Wash Pipe

SP; N.Dia.φ 0.250 m * 1 place/basin Section Area= 0.049 ㎡/basin

Flow Velocity through Filtered Pipe:

V = 0.823 m^3/sec/basin ／ 0.049 ㎡/basin = 16.800 m/sec

q) Outlet Weir

Width 1.30 m * Height 1.05 m * 1 place/basin

r) Opening Dimension of Wash Water Drain

Width 1.00 m * Height 0.35 m * 1 place/basin Section Area= 0.350 ㎡/basin

s) Drain Pipe

N.Dia.φ 0.15 m * 1 * place/basin Section Area= 0.018 ㎡/basin

6) Filterd Water Measurement and Chloline Mixing Chamber

a) Type of Structure

*** Reinforced Concrete**

b) Type of Method

*** Utilizing the Water Depth of Weir for Flow Measurement**

*** Utilizing the Energy of Broad Crested Rectangular Weir for Mixing**

c) Dimension

*** Filtered Water Flow Measurement Chamber:**

Width 3.00 m*Length 6.05 m * Depth 3.70 m *W.D. 3.01 m * 1 basin

*** Coagulation for Chloline Mixing Chamber**

Width 3.00 m*Length 3.00 m * Depth 3.70 m *W.D. 2.52 m * 1 basin

*** Broad Crested Rectangular Weir for Mixing**

Width 3.00 m*Height 2.20 m * 1 Unit

*** Perforated Baffle Wall**

*** Opening Section Area of Perforated Baffle Wall (m^2)** 8% : asumption

Width 3.00 m*W.D. 3.01 m * 0.08 % = 0.722 m^2

d) Capacity of Chamber

*** Filtered Water Flow Measurement Chamber:**

V_r= 3.00 m * 6.05 m * 3.01 m = 54.60 m^3

*** Coagulation for Chloline Mixing Chamber**

V_r= 3.00 m * 3.00 m * 2.52 m = 22.70 m^3

e) Detention Time of Chamber

*** Filtered Water Flow Measurement Chamber:**

T_r = 54.60 m^3 ／ 30.56 m^3/min = **1.79** min

*** Coagulation for Chloline Mixing Chamber**

T_r = 22.70 m^3 ／ 30.56 m^3/min = **0.74** min

f) Gate

Width **0.80** m* Height **0.80** m * 1 place　　Section Area= 0.640 ㎡

V_1 = Flow Capacity ／ Section Area = 0.509 ／ 0.640 = 0.795 m/sec

7) Filtered Water Connecting Pipe

*** Filtered Water Pipe-1 (to the New Clear Water Reservoir)**

New DIP; N.Dia.φ **0.700** m * Length **45.0** m　　Section Area= 0.385 ㎡

V_1 = Flow Capacity ／ Section Area = 0.509 ／ 0.385 = 1.322 m/sec

*** Filtered Water Pipe-2 (to the Existing Clear Water Reservoir)**

New DIP; N.Dia.φ **0.600** m * Length **135.0** m　　Section Area= 0.283 ㎡

8) Clear Water Reservoir

a) Existing Reservoir

*** Demension**

Width **15.80** m* Length **31.20** m * Depth **5.00** m * W.D. **4.00** m * **2** basin

*** Capacity of Reservoir**

V_r = 15.80 m * 31.20 m * 4.00 m * 2 Basin = **3,943.7** m^3

*** Detention Time of Reservoir**

T_r = 3,943.7 m^3 ／ 833.33 m^3/min = **4.73** hr

b) New Clear Water Reservoir

*** Required Capacity**

V_{re} = 6 hr * 60,000 m^3/d = 2,500 m^3/h = **15,000** m^3

(Existing + Expantion)

*** Required New Capacity**

V_{nre} = 15,000 m^3 - 3,943.7 m^3 = **11,056** m^3

(New)　(Existing)

c) * Dimension

Width **24.60** m* Length **54.60** m * Depth **5.00** m * W.D. **4.00** m * **2** basin

*** Effective Capacity of New Reservoir**

V_r = 24.60 m * 54.60 m * 4.00 m * 2 basin = **10,745** m^3

*** Detention Time agaist Total Planned Daily Maximum Supply Capacity**

14,689 m^3/2basin ／ 2,500.0 m^3/hr = **5.9 hrs**　　**OK**

(Existing + Expantion)　(Existing + Expantion)

*** Water Level Meter:**

*** Overflow and Drain Pipe:**

New DIP; N.Dia.φ **0.450**　　Section Area= 0.159 ㎡

: Top Elevation of Overflow Pipe = m

9) Distribution Pump

Dia.φ **350** * Q **723** m^3/hr (**12.10**) m^3/min * H **67** m* **195** kw

～ **4** units (**1** unit Standby)

2. ANALYSIS ON CHEMICAL FEEDING AND EFFECTS OF GRAVEL FILTRATION AT THE KAOLIEO TREATMENT PLANT

2. ANALYSIS ON CHEMICAL FEEDING AND EFFECTS OF GRAVEL FILTRATION AT THE KAOLIEO TREATMENT PLANT

2-1 Analysis on Chemical Feeding

Based on the water quality data, analysis on chemical feeding at the Kaolieo Treatment Plant is discussed as follows.

Raw Water Quality

1) Turbidity

The records of turbidity of Chinaimo and Kaolieo Treatment Plants in 2000 – 2002 are shown in table below. The average values of turbidity of raw water at Kaolieo and Chinaimo treatment plants are almost at the same level. However, the maximum turbidity of the raw water at Chinaimo treatment plant is higher than the one of Kaolieo treatment plant. The turbidity is an important parameter for determination of the coagulant dosage rate. Since the turbidity meter equipped at the Kaolieo treatment plant is out of date and the reliability of the turbidity data is low, the chemical dosage rate was considered based on the turbidity data of the Chinaimo Treatment Plant.

Record of Raw Water Turbidity at the Chinaimo and Kaolieo Treatment Plants

Month	Chinaimo Water Treatment Plant			Kaolieo Water Treatment Plant		
	Maximum (mg/l)	Minimum (mg/l)	Average (mg/l)	Maximum (mg/l)	Minimum (mg/l)	Average (mg/l)
Jan	106	28	57	188	121	140
Feb	77	21	39	138	70	99
Mar	53	12	30	100	74	82
Apr	90	2	13	85	41	59
May	544	21	182	550	30	204
Jun	1,910	229	487	1,835	224	444
Jul	4,645	237	1,164	3,521	335	1,107
Aug	4,660	645	1,370	2,415	610	1,407
Sep	1,653	334	782	1,324	275	649
Oct	1,165	270	553	778	204	384
Nov	757	136	494	282	190	230
Dec	875	23	209	365	141	215
	4,660	2	448	3,521	30	418

Based on the record shown above in 2002, the maximum turbidity was 4660 NTU. The turbidity more than 4600 NTU were recorded in two months (July and August). Under such condition the Chinaimo treatment plant was designed based on the maximum turbidity was 6,000 NTU. For the expansion of the Kaolieo treatment plant, same maximum value, 6,000 NTU is recommended to employ for the facility design.

According to the average turbidity during 2000 - 2002 was 432 NTU, therefore, the design average turbidity is set as 450 NTU. Although the minimum turbidity in 2002 was 2 NTU, since it is about 10 NTU during 2000 - 2001, design minimum turbidity is set as 10 NTU. The design turbidity values for the maximum, average, and minimum are summarized as follows:

Maximum	:	6000	NTU
Average	:	450	NTU
Minimum	:	10	NTU

2) Ammonia Nitrogen

According to the results of raw water quality analysis of the Kaolieo treatment plant during 2000 - 2002, concentration ranged from 0.01 to 0.06 mg/litre. There was no significant fluctuation of the concentration during these three years; it is assumed that this tendency would continue at the same level as the present condition. Maximum concentration of the Ammonia Nitrogen was set as 0.1 mg/ litre for the facility design.

3) Iron and manganese

Iron and Manganese are detected in raw water regardless the rainy and dry seasons. There was no significant fluctuation of Iron and Manganese concentration during the last three years, it was assumed that this tendency would continue as same level as the present condition. The maximum concentration of Iron is set as 0.50 mg/litre and the maximum concentration of Manganese is set as 0.10 mg/litre.

Chemical Feeding

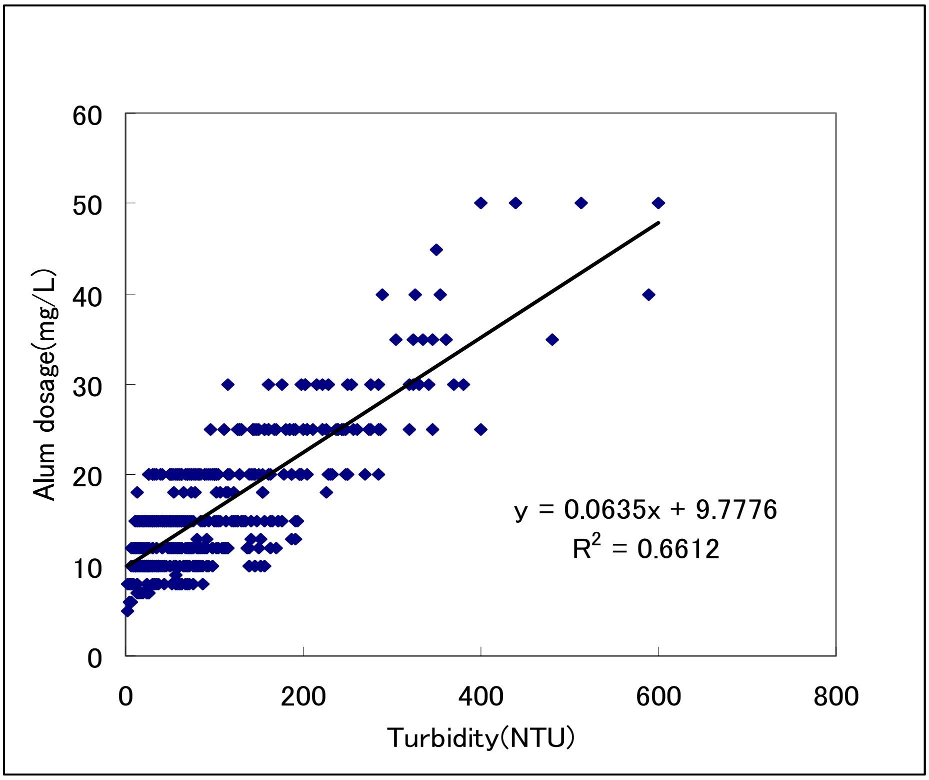
1) Coagulants (Alum (Aluminium Sulphate) and Polymer)

Alum(aluminium sulphate) will be dosed as a coagulant which is applied at the existing treatment plants. Polymer is also applied as a coagulant aid. Since correlation with that the dosage rate of aluminium sulphate and raw water turbidity at the Kaolieo treatment plant is deficient, it is difficult to calculate the dosage rate of coagulants from the actual records of Kaolieo. Then, coagulants dosage rates were derived from the records of the Chinaimo Treatment Plant. At the Chinaimo water treatment plant, polymer is dosed when the turbidity becomes more than 300 NTU, the polymer will be similarly dosed from about 300 NTU at the Kaolieo treatment plant..

The relation between turbidity and aluminium sulphate dosage rate, without polymer case, is expressed with the following formula from figure below.

Aluminium sulphate dosage (mg/L) = 0.0635×turbidity (NTU) + 9.7776 (1)

Relation between raw water turbidity and aluminium sulphate dosage (Chinaimo water treatment plant actual result)

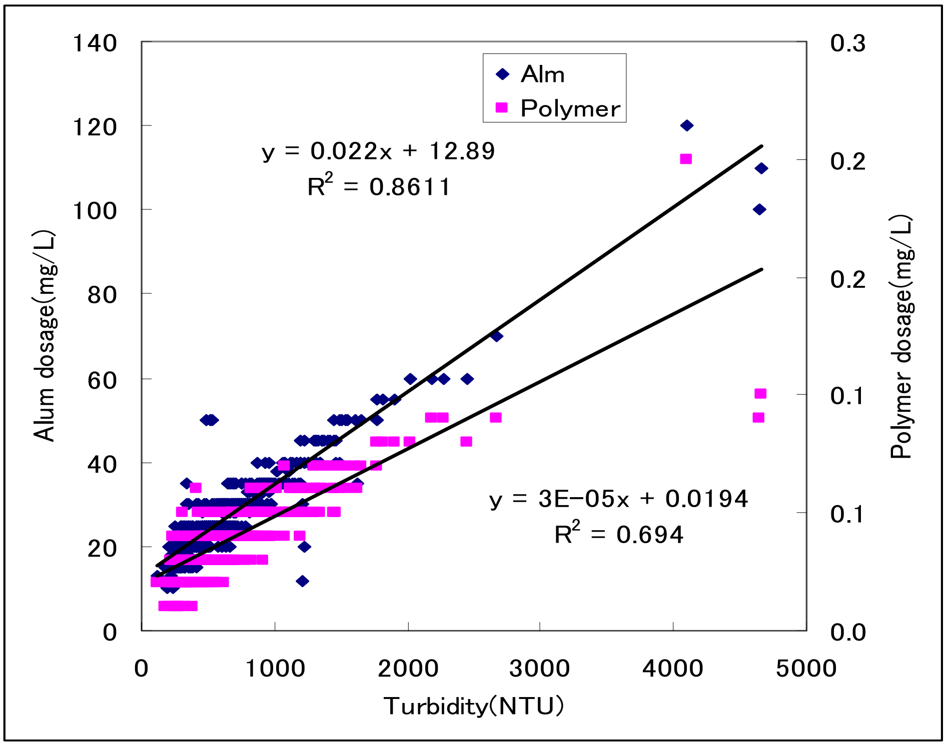


The relation between turbidity and dosage rates of aluminium sulphate and polymer is expressed with the following formula from figure below.

Aluminium sulphate dosage (mg/L) = 0.022× turbidity (NTU) + 12.89 (2)

Polymer dosage (mg/L) = 0.00003× turbidity (NTU) + 0.0194 (3)

Relation between raw water turbidity and dosage rates of aluminium sulphate and polymer dosage.



a) The Maximum Dosage Rates

Since the maximum turbidity of Kaolieo is 6000 NTU, dosage rate will be calculated from formula (2) and (3) above.

Alum dosage $= 0.022 \times 6000 + 12.89$

$= 144.89 \quad 150\text{mg/L}$

The maximum aluminium sulphate dosage rate is derived as 150mg/liter.

The dosage rate of the polymer is calculated as follows.

Polymer dosage $= 0.00003 \times 6000 + 0.0194$

$= 0.1994 \quad 0.2\text{mg/L}$

The maximum polymer dosage rate is as 0.2mg/liter.

b) The Average Dosage Rates

Since the average turbidity of Kaolieo is 450 NTU, dosage rate will be calculated from formula (2) and (3) above.

Alum dosage $= 0.022 \times 450 + 12.89$

$= 22.79 \quad 23.0\text{mg/L}$

The average aluminium sulphate dosage rate is derived as 23 mg/liter.

The dosage rate of the polymer is calculated as follows.

Polymer dosage $= 0.00003 \times 450 + 0.0194$

$= 0.033 \quad 0.03\text{mg/L}$

The average polymer dosage rate is as 0.03mg/liter.

c) The Minimum Dosage Rates

Since the average turbidity of Kaolieo is 10 NTU, dosage rate will be calculated from formula (2) and (3) above.

Alum dosage $= 0.0635 \times 10 + 9.7776$

$= 10.41 \quad 10\text{mg/L}$

The average aluminium sulphate dosage rate is derived as 10 mg/liter.

The results of above mentioned calculation are summarized as follows.

Aluminium sulphate and polymer dosage rate (unit: mg/litre)

Aluminium sulphate			polymer		
Max	Min	Ave	Max	Min	Ave
150	10	25	0.2	0	0.03

The JICA Study Team conducted analysis of acrylamidemonomer which is contained in polymer used by the NPVC. According to the results of the analysis, concentration of acrylamidemonomer will be 0.002μg/liter at the planned maximum dosage rate of polymer. This concentration as of 0.002μg/litre is far lower than the WHO Water Quality Guidelines, 0.5μg/litre and furthermore, much lower than the Ministerial ordinance of Health, Labour and Welfare, Japan, 0.005μg/litre. Therefore, the usage of polymer at the Treatment Plant will not affect the human health.

2) Chlorine (Calcium Hypochlorite: Hypo)

Residual chlorine should be maintained in the treated water. Since chlorine is consumed by substances contained in the raw water and treated water such as Ammonia Nitrogen, Iron, and Manganese, dosage rate of the chlorine will be calculated from the concentrations of these substances.

a) Ammonia Nitrogen

Chlorine consumption will be calculated based on the maximum concentration of ammonia nitrogen, 0.1 mg/litre, as follows.

Ammonia nitrogen ： 1mg/L＝10mg/L

0.1mg/L×10mg/L＝1.0mg/L

b) Iron

Chlorine consumption will be calculated based on the maximum concentration of Iron, 0.5 mg/litre, as follows.

Iron :1mg/L＝0.63mg/L

0.5mg/L×0.63mg/L＝0.315 0.32mg/L

c) Manganese

Chlorine consumption will be calculated based on the maximum concentration of Manganese, 0.1 mg/litre, as follows.

Manganese ： 1mg/L＝1.29mg/L

0.1mg/L×1.29mg/L＝0.129 0.13mg/L

The total chlorine consumption therefore will be calculated as follows.

Total chlorine consumed＝1.0mg/L＋0.32mg/L＋0.13mg/L

＝1.45mg/L 1.5mg/L

Although the total chlorine consumption is calculated as 1.5 mg/litre, the maximum chlorine dosage is set as 5 mg/litre. Chlorine consumption substance and the turbidity will fall at the dry season, intermediate chlorine dosing will be desirable. A chlorine dosage rates are summarized as follows.

Chlorine dosage rate (unit : mg/litre)

	Max	Min	Ave
Pre	5	1	3
Intermediate	5	1	3
Post	2	0.5	1

Pre and intermediate simultaneous dosage will not be allowed

3) Lime

Alkalinity has an effect in pH adjustment and a corrosion prevention of tap water. Raw water from the Mekong River contains enough alkalinity, 90 mg/litre (results at the Chinaimo treatment plant) in raw water. Since it remains 22.5 mg/litre (90-150×0.45=22.5 mg/L) at the maximum aluminium sulphate dosage, it is assumed that the problem of corrosion will not be generated. Moreover, pH of raw water is 8.0-8.4 (2000 - 2002: results at the Kaolieo treatment plant), and even if aluminium sulphate dosage becomes the maximum, pH was maintained about 7.0. Therefore, the post lime dosage is judged not required.

The chemical feeding facility is planned not only for the expansion works but also for the rehabilitation work. The new chemical feeding facilities, which will be constructed as a part of the expansion work, will be shared by both the existing and new facilities. The chemical dosage rates for aluminium sulphate (alum), anionic polymer (polymer) and calcium hypochlorite (hypo) are summarized as shown below.

Summary of Chemical Feeding Rate at Kaolieo Water Treatment Plant (unit: mg/litre)

		Max	Min	Ave	Remark
Aluminium sulphate		150	10	25	Value of Min. is without polymer
Polymer		0.2	0.0	0.03	
Hypo	Pre	5.0	1.0	3.0	Pre- and Intermediate will not dosed simultaneously
	Intermediate	5.0	1.0	3.0	
	Post	2.0	0.5	1.0	

2-2 Effects of Gravel Filtration at the Kaolieo Treatment Plant

The gravel filter layer is installed at the effluent of sedimentation basin at the Kaolieo Treatment Plant. Maintenance work of the gravel layer is conducted by the staff of Kaolieo Treatment Plant and the maintenance works include washing the layer by pressurized water from its surface since layer washing pipe has not been function and sometimes gravel is removed from layer and it is washed manually outside of the basin.

Since the maintenance works are very burden and the effects of the layer is questionable, the Study Team investigated the effects of the gravel layer by measuring turbidity before and after the gravel layer. If the turbidity after the gravel layer is decreased one before the layer, the gravel layer will be evaluated as effective.

Turbidity was measured two times each on October 6 and October 16 and the results of the measurements are shown below.

Results of Turbidity Measurements Before and After the Gravel Filter

	October 6		October 16	
Turbidity measurement	No. 1	No. 2	No. 3	No. 4
Condition of the gravel layer	ordinary condition	gravel was removed from gravel filter	ordinary condition	just after gravel washing by pressurized water from the layer surface
before gravel filtration	5.8	6.3	6.5	6.5
after gravel filtration	8.2	6.5	7.8	5.8

As shown on table above, under the ordinary condition, measurements No. 1 and No. 3, turbidity after the gravel filtration are higher than ones before the filtration. This turbidity increase may caused by discharging turbidity from the gravel layer which has been accumulated in the layer.

It is observed that turbidity is increased even though the gravel was removed from the filter layer. This may be because of the turbulent flow caused by structure of the gravel filter. If the gravel is placed in the layer, water flow will be regulated by the gravel.

Only from the measurement results of No. 4, decrease of turbidity after the gravel filtration. This is because the gravel layer was clean by the recent washing. However, reduction of the turbidity is rather small than expected.

Taking account of frequency of gravel washing and such burden maintenance work, the efficiency of the gravel filtration is judged not so high. Therefore, it is recommended to remove gravel filter during the rehabilitation work on the existing Kaolieo Treatment Plant.

Study on Necessity of Grit Chamber

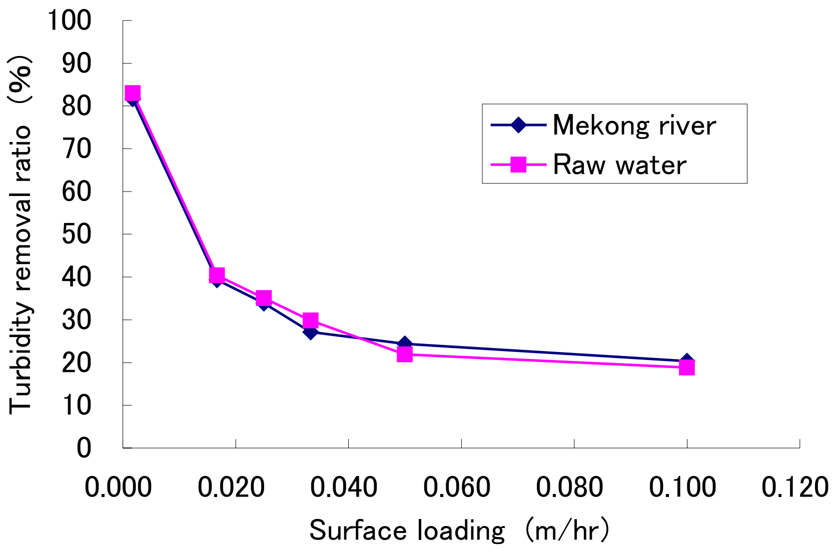
The necessity of the grit chamber is examined from the results of the subsidence test. Characteristics of the subsidence have strong relation with the size of particle of the turbidity. According to the results of subsidence test using water from the Mekong River and raw water of the Kaolieo Treatment Plant, rate of subsidence was 0.038 m/hr and it was very small value because of the majority of turbidity of the Mekong River is very fine silts. The results of the subsidence test are shown on table below.

Results of the Subsidence Test

	Mekong river			Raw water		
Time (min)	Turbidity (NTU)	Removal Ratio (%)	Subsidence Rate m/hr	Turbidity (NTU)	Removal Ratio (%)	Subsidence Rate m/hr
0	295	—	—	265	—	—
30	235	20.3	0.100	215	18.9	0.100
60	223	24.4	0.050	207	21.9	0.050
90	215	27.1	0.033	186	29.8	0.033
120	195	33.9	0.025	172	35.1	0.025
180	179	39.3	0.017	158	40.4	0.017
1680	54	81.7	0.002	45	83.0	0.002
Ave			0.038			0.038

Figure blow shows the relation between turbidity removal ratio and subsidence rate. If the turbidity removal at 30 % is expected at the grit chamber, subsidence rate, in other words surface load, should maintain as 0.025 m/hr from the figure below.

Fig.3 Relation between Turbidity Removal Ratio and Subsidence Rate (Surface Load)



Area required for the grit chamber will be calculated from the results of subsidence test as follows:

Capacity of Treatment Plant:	60,000 m3/day
Surface Load at 30 % removal:	0.025 m/hr
Required area of Grit Chamber:	60,000 m3/day / 24 hour / 0.025 m/hr
	= 100,000 m2 (10 ha)

Even though only 30 % of turbidity removal is expected, huge area, about 10 ha, grit chamber will be required because of low subsidence rate. Therefore, construction of the grit chamber is judged not realistic and not feasible.

Consideration on Sediments inside the Intake Pipe

Intake Pipe Type is adopted as an intake facility for the expansion of Kaolieo Treatment Plant. Although the Intake Pipe Type has great advantages comparing with other type of intake facility, this type has misgivings about sedimentation or accumulation of mud inside the intake pipe which is stopped water intake when raw water introduced through other pipe.

Raw water will be introduced though the pipe which is the nearest from the river surface since surface water contains lower turbidity. Therefore, stoppage of the intake pipe will occur deeper side. At the certain depth of river, water flow will be gentler comparing with surface, and water circulation inside the stopped intake pipe will not occur. This means, that sedimentation of mud will be observed but the sedimentation will be very limited since there will be no intrusion of turbidity from the mouth of the pipe.

Taking account of the characteristics of the turbidity of the Mekong River, they are very fine silt, accumulation is estimated also very limited.

3. NETWORK ANALYSIS
FOR PRIORITY PROJECT IN THE 1ST STAGE

3. NETWORK ANALYSIS FOR PRIORITY PROJECT IN THE 1ST STAGE

3.1 General

Although the improvement of distribution network system was not included in the JICA study originally because of the demarcation of the study between JICA and AFD. To complete the 1st stage project of the master plan, however, the installation of 24.2 km length of distribution mains are required. Selection of priority projects and feasibility study for the distribution system were originally scheduled to be conducted by the AFD study according to the demarcation.

The distribution network system is also indispensable for maintaining the function of the water supply system properly, and should be implemented at the same time as the projects selected by the JICA study. It should be noted that without strengthening the distribution network system, the water supply system will not function properly even though the production capacity will be increased and the transmission system will be developed as a result of the JICA study. It was recommended that the minimum required distribution mains for the system as shown in Figure 3 -1 should be installed at the same time as the expansion of the treatment plant capacity and the development of the transmission system for the Vientiane water supply development. Therefore, the JICA proposed, and the AFD and other agencies concerned i.e. WASA and NPVC, agreed to include the minimum required distribution mains into the JICA feasibility study at the Kick-off Meeting for the feasibility study held on 11th September 2003. It is noted that even though the minimum required distribution mains are included in the JICA study, these mains are not eliminated from the AFD study because the study of the overall distribution network system to be carried out by AFD should consider all the distribution mains including the minimum required distribution mains.

In the analysis to identify the minimum required distribution trunk mains, which was conducted during the master planning, the minimum residual pressure at each junction was not maintained at 15m (1.5 kg/cm2), but the average residual pressure should measure more than zero. This means that water is only available at intermittent periods throughout the day. The purpose of this network analysis is to verify whether the average residual pressure at each junction should be maintained at more than zero based on the data of the investigation results carried out during the feasibility study.

3.2 Results of Field Investigation for Pipeline Routes

During the second field investigation for the Phase III Feasibility Study on the Priority Projects from August to November 2003, the JICA study team conducted a line survey including the longitudinal and cross sections along the proposed pipeline routes for the transmission mains and the minimum required distribution mains. In the Phase II Preparation of Master Plan, the lengths of the proposed pipelines were estimated from the drawings of the NPVC. Based on the results of the survey, considering the detailed field investigation, and referring to the on-going projects, the length of the proposed pipelines for the priority projects are compared in Table 3-1. For the network analysis in the following section, the pipeline lengths obtained from the survey are used.

Table 3-1 Comparison of Pipeline Length

Dia (mm)	Minimum Required Distribution Mains (km)		Transmission Mains (km)	
	M/P	F/S	M/P	F/S
150	4.57	4.57	-	-
250	3.22	3.24	-	-
400	4.89	4.65	-	-
450	-	-	2.22	1.88
600	1.76	1.62	-	-
700	0.68	0.50	0.58	0.72
Total	15.12	14.58	2.80	2.60

Note: Figures in “M/P” were estimated at Phase II of the Preparation of Master Plan.
Figures in “F/S” are adopted for the preliminary design of the priority projects based on the investigations conducted during the Phase III of the Feasibility Study on the Priority Projects.

3.2 Network Analysis for the 1st Stage Project

(1) Conditions of Network Analysis

Methods of the network analysis for the 1st stage project in the target year of 2007 including the priority projects are same as the network analysis for the master plan which is detailed in Section 4.5 of the Comparative Study of Alternatives of Volume II of the Master Plan and Annexes 15 and 20. Differences in the conditions of the network analysis attached in Annex 20 are mainly the pipeline length of the minimum required distribution mains and the transmission mains as shown in Table 3-1.

The network analysis has been conducted using WaterCAD. Conditions of the network analysis are as follows:

a. Formula for friction loss calculation: Hazen-Williams Formula
b. C value for all pipes: 110
c. Velocity Range: 1.0 m/s – 1.5 m/s as the target
d. Hourly peak factor for domestic demand is estimated at 1.4. Half of non-domestic demand is assumed to be the same hourly peak factor as the domestic demand. For the remaining

non-domestic demand, hourly peak factors are not applied since non-domestic customers have their own reservoir. The basis of calculations of the hourly peak factor is shown in Annex-15. The overall hourly peak factor, the average of domestic and non-domestic demand is calculated as 1.3.

Conditions of the pumps, reservoirs and tanks for the network analysis are shown in Figure 3-2.

(3) Results of Network Analysis

As the results of the analysis, it was confirmed that the average residual pressure at each junction will be maintained at more than zero in cases where the minimum required distribution mains were installed at the same time as the expansion of treatment plant capacity and the development of the transmission system. The residual pressure contours are shown in Figures .3-4 to 3-11.

After the completion of the priority project, the transmission and distribution systems of the Vientiane Water Supply System will be developed as shown in Figure 3-3. The behavior of each elevated tank is shown in Figure 3-12.

It is, however, noted that to install only the minimum required distribution mains is not the appropriate development of the distribution network system in 2007. Essentially the appropriate system should follow the master plan prepared in Chapter 4. A detailed study on the distribution network system will be conducted by the AFD study.

Figure 3-1 Transmission and Distribution Mains for Priority Project

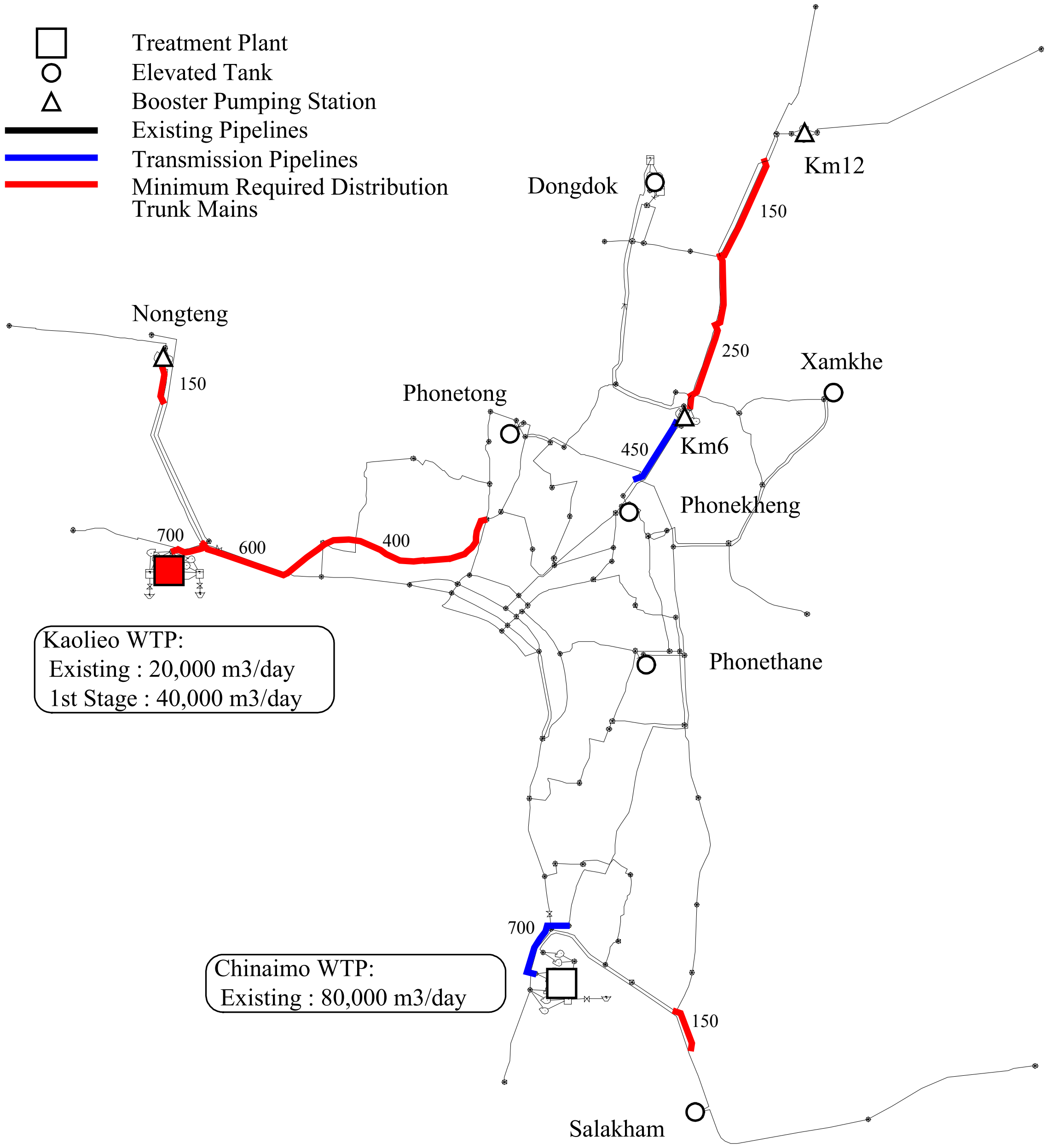


Figure 3-2 Transmission and Distribution System

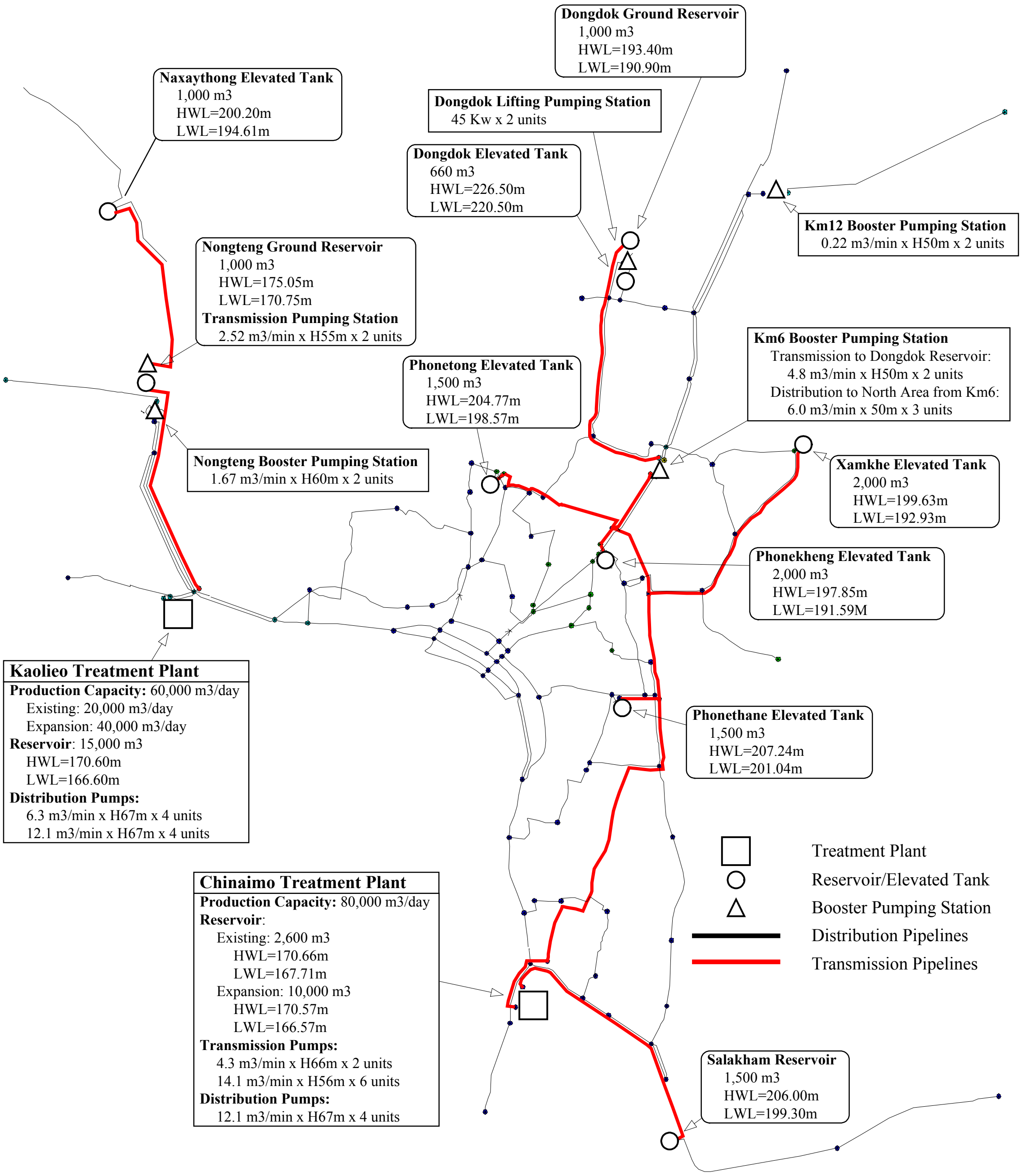


Figure 3-3 Proposed Transmission and Distribution Systems

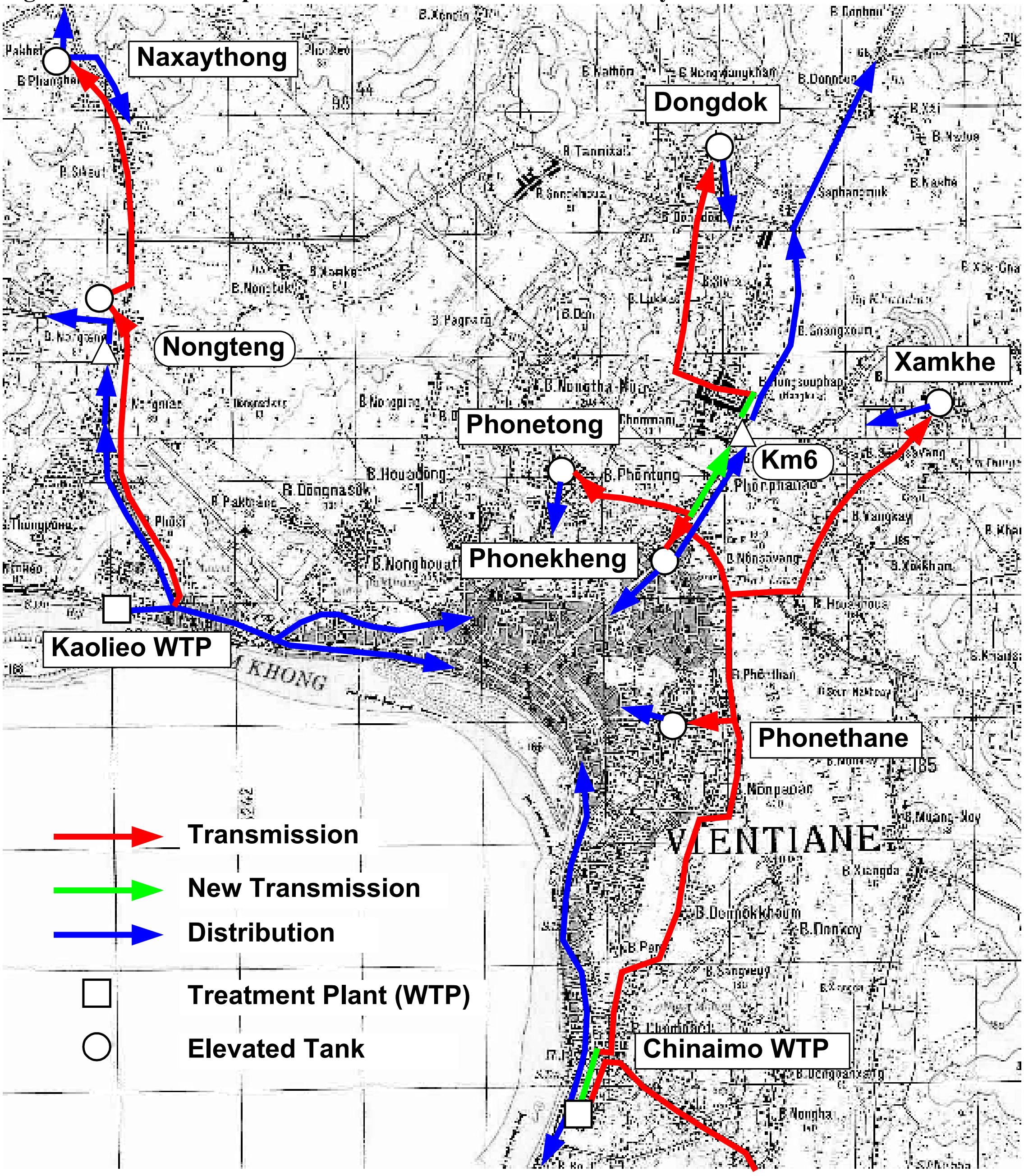


Figure 3-4 Residual Water Pressure Contour at 03:00

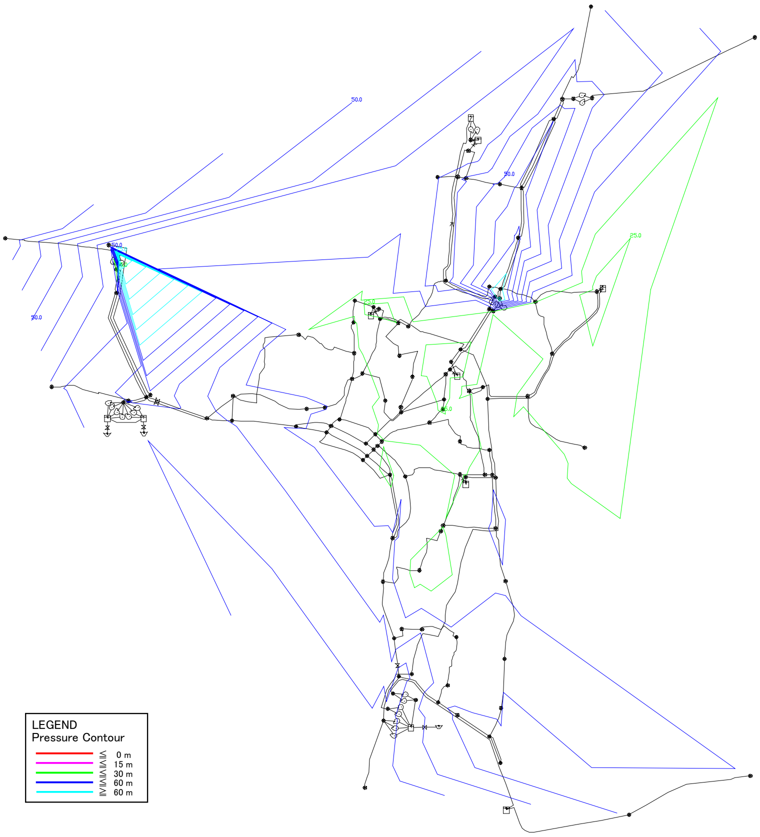


Figure 3-5 Residual Water Pressure Contour at 06:00

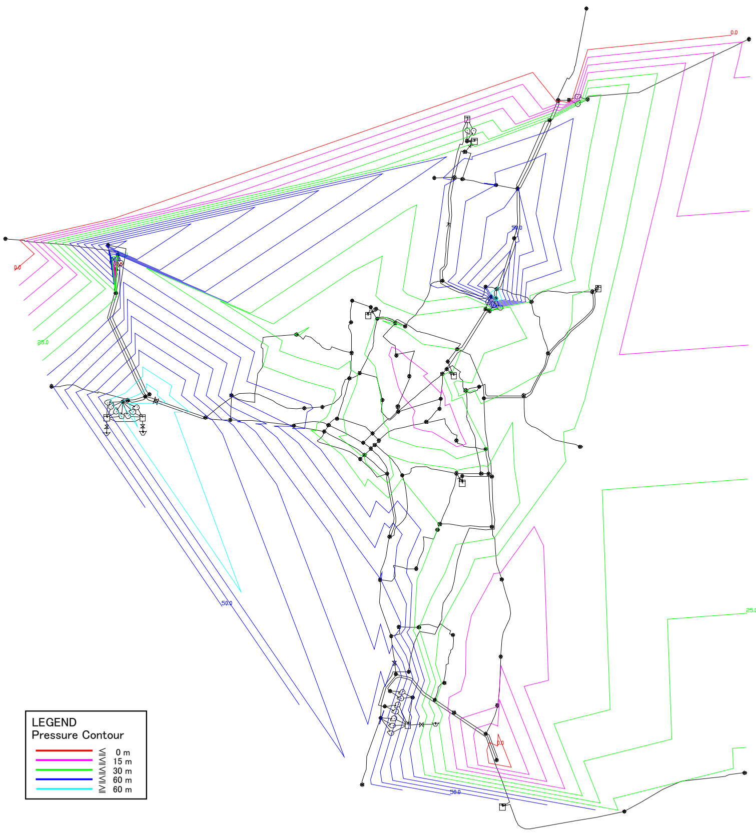


Figure 3-6 Residual Water Pressure Contour at 09:00

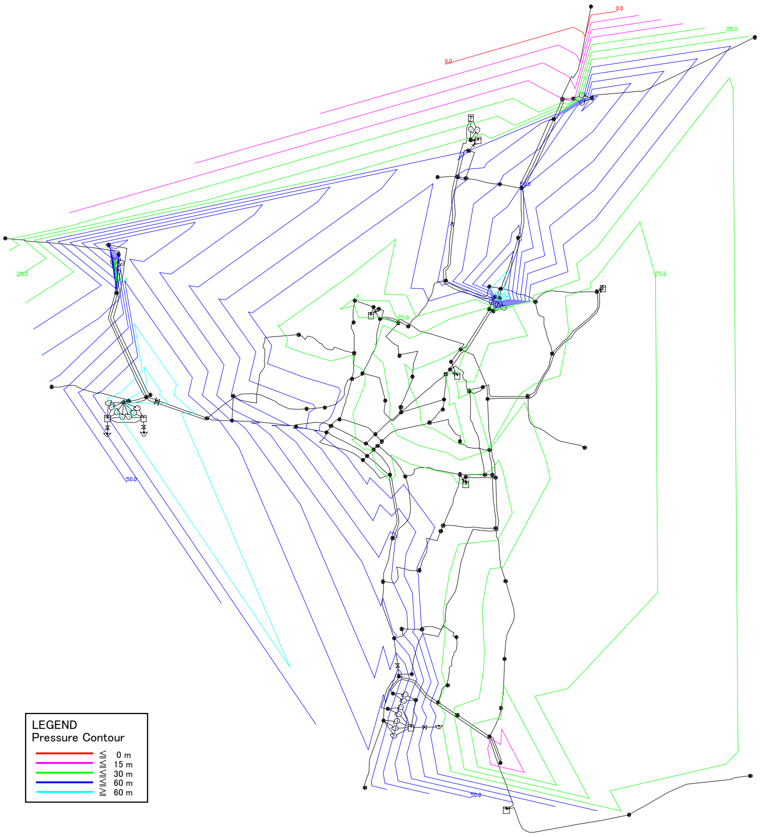


Figure 3-7 Residual Water Pressure Contour at 12:00

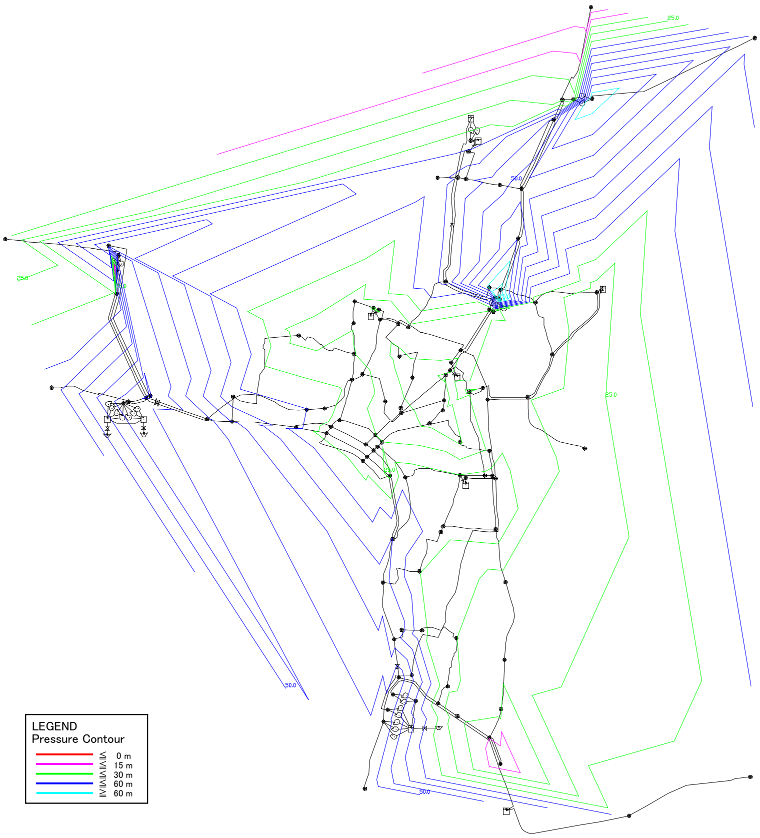


Figure 3-8 Residual Water Pressure Contour at 15:00

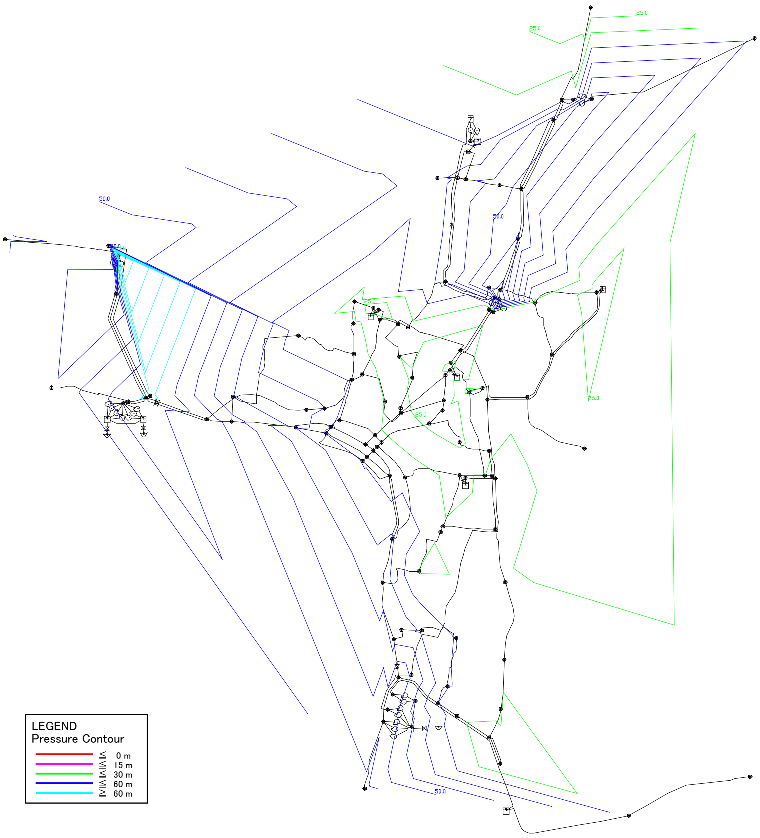


Figure 3-9 Residual Water Pressure Contour at 18:00

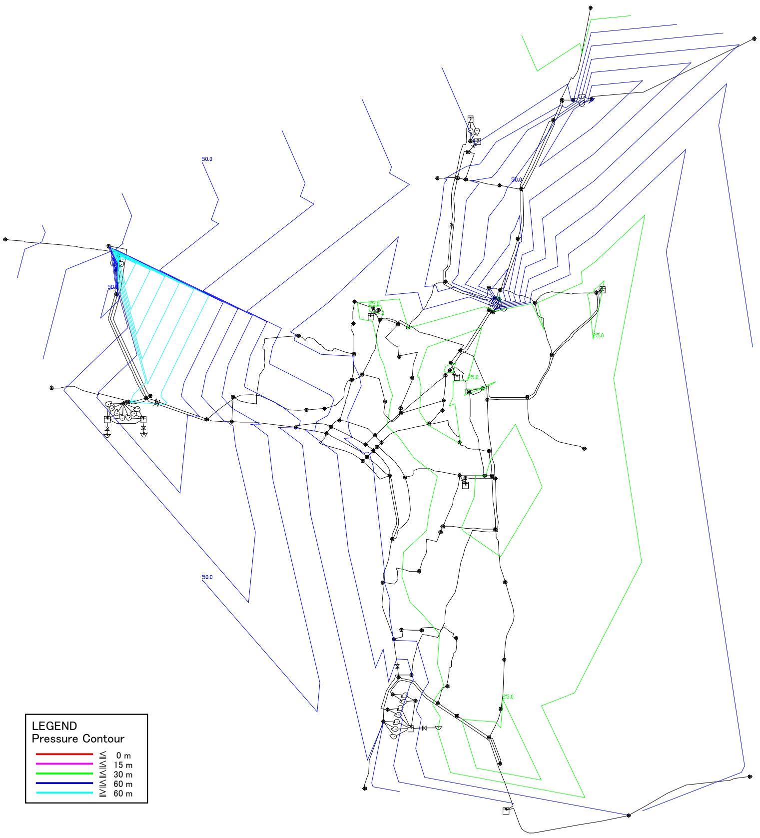


Figure 3-10 Residual Water Pressure Contour at 21:00

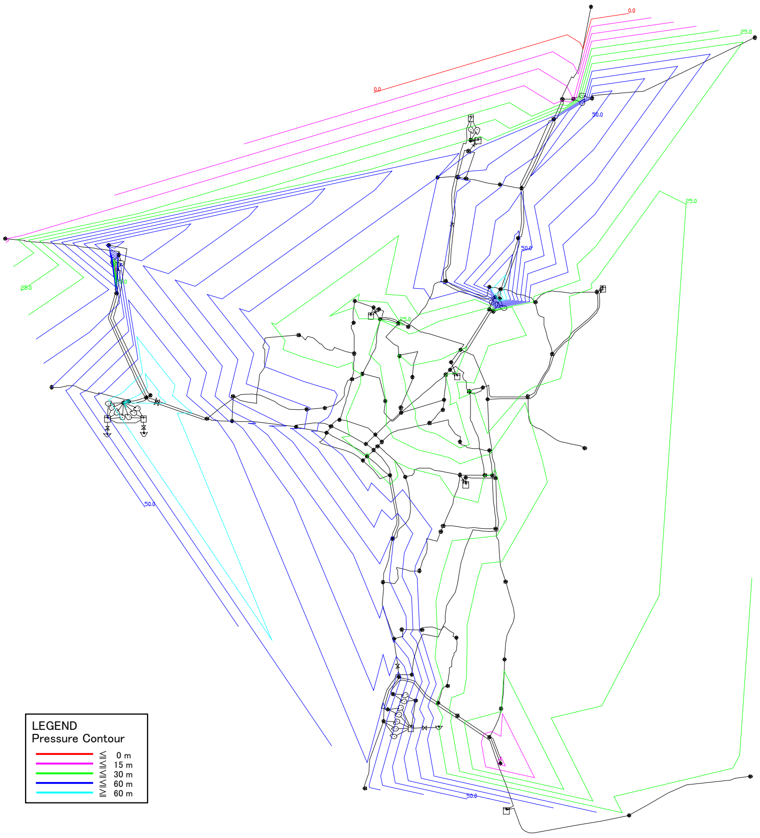


Figure 3-11 Residual Water Pressure Contour at 24:00

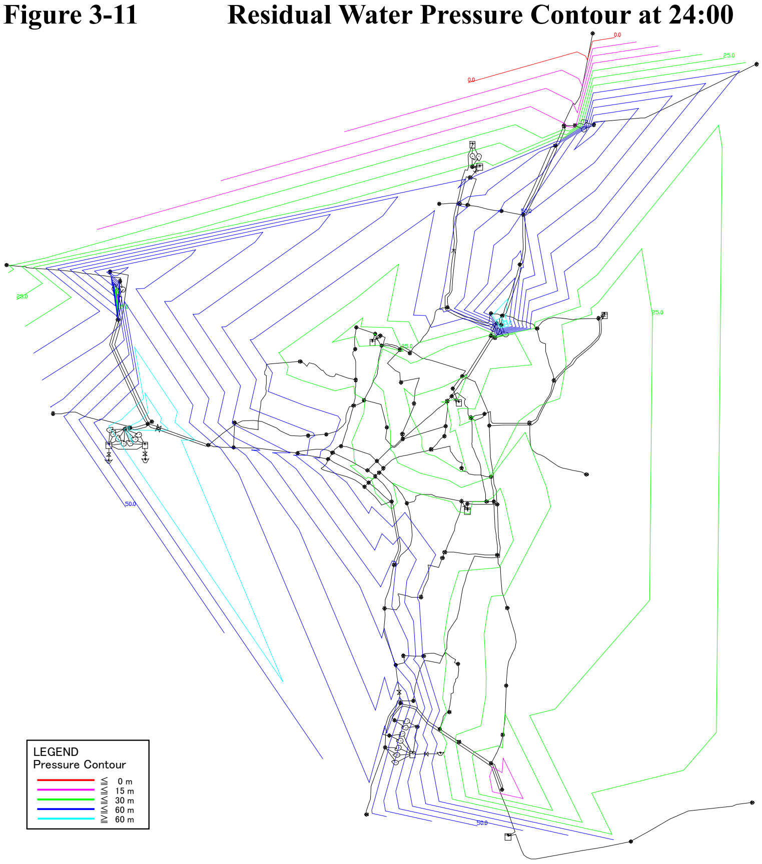
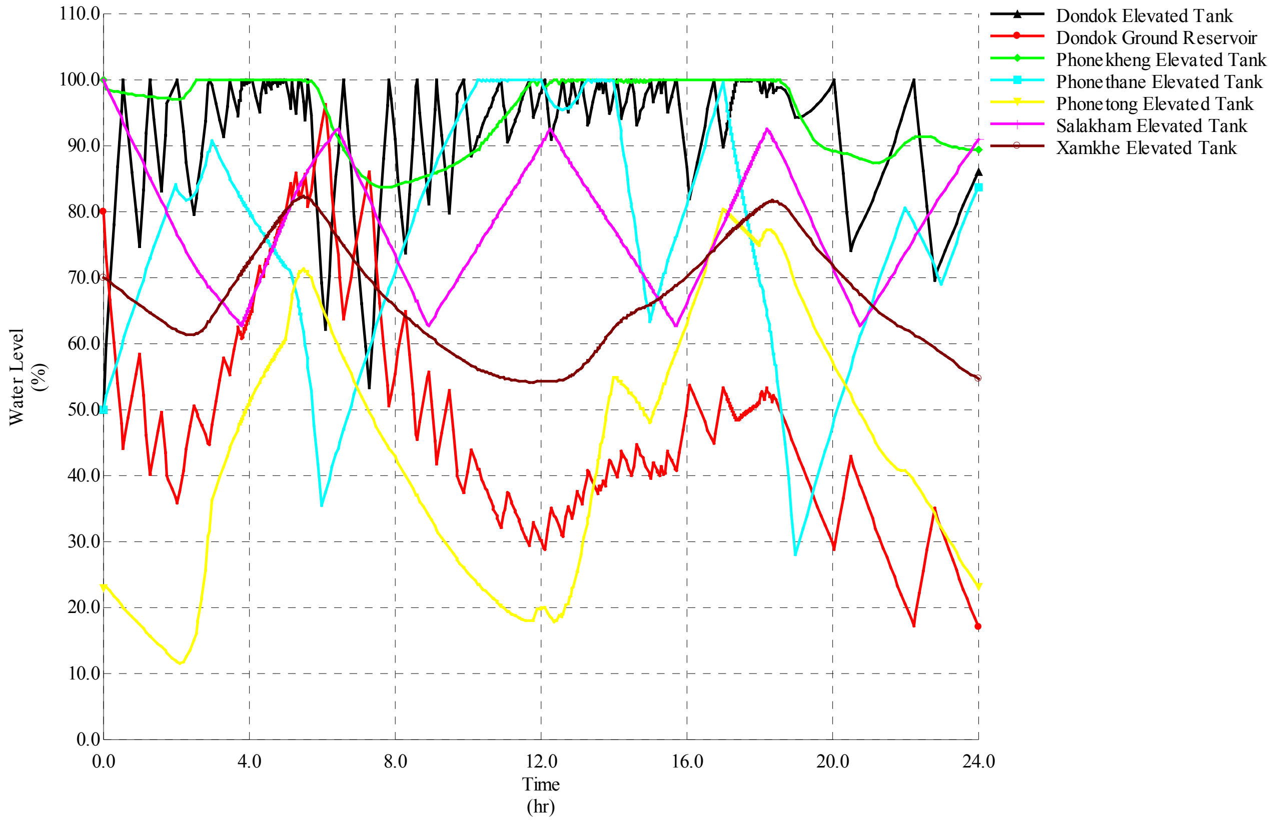


Figure 3-12 Water Level Fluctuations of Elevated Tanks



Attachment A25

Approval Letter from MCTPC

on Raw Water Intake for the Priority Project

from the Mekong River

Attachment A25 Approval Letter from MCTPC on Raw Water Intake for the Priority Project from the Mekong River

Lao People's Democratic Republic
Peace Independence Democracy Unity Prosperity

Ministry of Communication Transport Post and Construction
Department of Housing and Urban Planning
Water Supply Authority

No.: 026 [illegible] /WASA

Date: 21 October, 2003

Mr. Takemasa Mamiya
Team Leader
The JICA Study Team for the Study
on Vientiane Water Supply Development Project

Confirmation on Additional Raw Water Intake from the Mekong River at Kaolieo Treatment Plant.

Dear Mr. Takemasa Mamiya,

Following your letter No. SVWSDP-050 dated 13 October, 2003 regarding the request for confirmation or approval on additional raw water intake from the Mekong River at Kaolieo Treatment Plant, we would like to advise you that:

- Based on article 9, 14 and 15 of the law on water and water resources No 02-96/NA dated 11 October, 1996;
- With reference to the Prime Ministerial Decree No 204/PM dated 9 October, 2001 on the implementation of the law on water and water resources; and
- With reference to the Minutes of Meeting on Mater Plan agreed upon in Vientiane on 12 June, 2003;

it should be understood that the additional raw water intake from the Mekong River for the Vientiane Development Project has already been approved.

Attached please find a copy of the first two legal documents above mentioned we refer to.

Yours sincerely,

Noupheuak Virabouth
Director General

Approved by:

Dr. Somphone Dethoudom
Director General of DHUP