C-3 Design of the Pilot Project

C.3.1 JICA Design Review Report for Phlai Chumphon Project

C.3.1.1 Hydrauric Design Statement of Outlet Drainage Structure at km 56+227.563 Case a-1 and Case b-1

Case a-1) Lining Canal WL.42.603 = 90cum/s

⇒ Outlet Drainage (WL.41.744=49.5cum/s) ⇒ Drainage Canal DR.15.8 Design WL.37.919m + 49.5cum/s

& Case b-1) Existing Canal WL.43.293 = 90cum/s

⇒ Outlet Drainage (WL.41.744=49.5cum/s) ⇒ Drainage Canal DR.15.8 Design WL.37.919m + 49.5cum/s

Gate: Full Opened or Regulated Opening **Design Discharge Qd = 49.5 m^3/s**

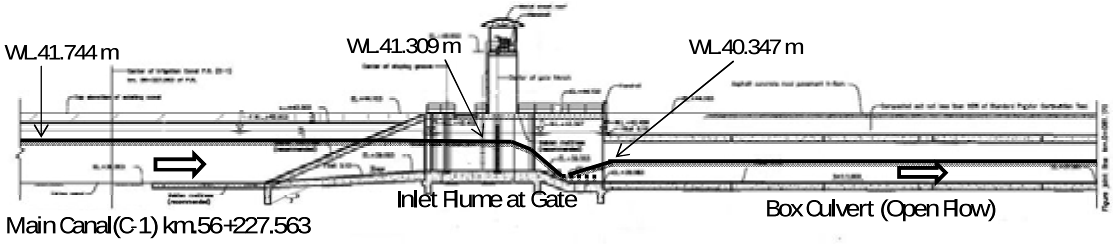
	Structure	Station	Distance L (m)	Discharge Q (m^3/s)	Energy Loss hl (m)	Energy Line EL. EH (m)	Velocity V (m/s)	Velocity Head Vh (m)	Water Level WL (m)	Water Depth d (m)	Bed Elevation BL (m)	Remarks
	Canal P.R. (C-1)											
0		0.00		49.500		41.744	0.000	0.000	**41.744**	2.741	39.003	
	Inlet		0.00		0.145							hl_1 = hi
1		0.00		49.500		41.599	2.385	0.290	41.309	2.306	39.003	
	Inlet Flume at Gate		10.00		0.013							$hl_2 = hf_2$
2		10.00		49.500		41.586	2.404	0.295	41.291	2.288	39.003	
	Inlet Transition of Box Culvert		5.50		0.041							hl_3 = he + hf_3
3	(Hydraulic Jump)	15.50		49.500		41.545	7.580	2.931	38.613	0.610	38.003	
	Inlet of Box Culvert(Open Flow)		0.00		0.966							hl_4 = hsc+hlj
4		15.50		49.500		40.579	2.133	0.232	40.347	2.344	38.003	
	Box Culvert (Open Flow)		85.68		0.080							$hl_5 = hf_5$
5		101.18		49.500		40.499	2.128	0.231	40.268	2.350	37.918	
	Outlet of Box Culvert		0.00		0.002							hl_6 = hse
6		101.18		49.500		40.497	1.938	0.192	40.305	2.387	37.918	
	Outlet Flume		9.35		0.004							$hl_7 = hf_7$
7		110.53		49.500		40.493	1.942	0.192	**40.300**	2.382	37.918	
	End Section of Outlet Flume											
	Total		**110.53 m**		**1.251 m**			**ΣΔWL =**	**1.444 m**			

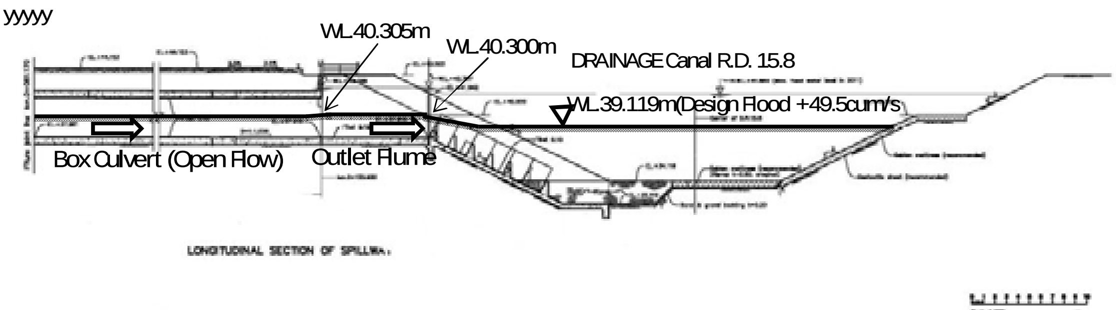
Water level at End Section of Outlet Flume **41.744 -1.444 = 40.300 m > 39.119 m**
OK

Water level at Drainage Canal D.R.15.8 **(Design Drainage + 49.5cum/s) 39.119 m**

Drainage Design+49.5 = WL.37.919 (Q = 61.817 cum/s) + 49.5 cum/s = 111.317 cum /s ⇒WL.39

OUTLET DREINAGE STRUCTURE





Hydraulic Calculation of Outlet Drainage Structure (Design Discharge, Qd=49.5cms)

1. Design Conditions

(1) Design discharge and no. of barre

- Design discharge	Q	=	49.500 m^3/s	= Qd
Design discharge per barrel	Q_1	=	16.500 m^3/s/barrel	
- No. of barrels of condui	N	=	3 nos.	

(2) Hydraulic conditions at beginning section (Canal P.R. (C-1) km.56+227.56

- Water level	FWL	WL_0	=	41.744 m	
- Base elevation		EL_0	=	39.003 m	
- Water depth		h_0	=	2.741 m	
- Velocity		V_0	=	0.000 m/s	
- Roughness coefficien		n	=	0.014	(Concrete)
- Acceleration of gravity		g	=	9.8 m/s^2	

(3) Head loss coefficient at open transitio

Changing condition o open transition formatior	Gradual contractior coefficient fgc	Gradual enlargemen coefficient fge
Rectangular to rectangula	0.10	0.20
Trapezoidal to trapezoida	0.10	0.20
Rectangular to circular with fille	0.20	0.30
Trapezoidal to rectangular with twisted wa	0.20	0.30
Trapezoidal to circular with twisted wa	0.30	0.40
Trapezoidal to rectangular with bended wa	0.30	0.50
Trapezoidal to circular with bended wa	0.40	0.70

2. Inlet Flume at Gate Section

(1) Discharge per barrel of inlet flume Q_1 = 16.500 m^3/s/barrel

(2) Hydraulic conditions at beginning section (Canal P.R. (C-1) km.56+227.56

- Water level	WL_0	=	41.744 m
- Base elevation	EL_0	=	39.003 m
- Water depth	h_0	=	2.741 m
- Velocity	V_0	=	0.000 m/s
- Velocity head	$V_0^2 / 2g$	=	0.000 m

(3) Change of water level due to inflov

$\Delta hi = hi + (V_1^2 / 2g - V_0^2 / 2g)$

$hi = fi (V_1^2 / 2g - V_0^2 / 2g)$

where,

hi : Head loss due to inflov

fi : Head loss coefficient due to inflov

In case, square cu fi = 0.50

a) Assumed Δhi — Δhi = 0.435 m

b) Beginning section of inlet flume (after inflow

- Base elevation	EL_1 =	39.003 m		
- Width	b_1 =	3.000 m		
- Water depth	h_1 =	2.306 m	=	$WL_0 - EL_1 - \Delta hi$
- Flow area	A_1 =	6.918 m	=	$b_1 h_1$
- Wetted perimeter	P_1 =	7.612 m	=	$b_1 + 2h_1$
- Hydraulic radius	R_1 =	0.909 m	=	A_1 / P_1
- Velocity	V_1 =	2.385 m/s	=	Q_1 / A_1
- Velocity head	$V_1^2 / 2g$ =	0.290 m		
	EL_{1e} =	41.599 m		

c) Calculation of hi andΔhi — hi = 0.145 m

∴ Δhi = 0.435 m = Assumed Δhi OK

(4) Water level at beginning section of inlet flume (after inflov

WL_1 = 41.309 m = $WL_0 - \Delta hi$

(5) Length and bed slope of inlet flum

- Length	L_2	=	10.000 m
- Bed slope	I_2	=	Level

(6) Change of water level due to frictio

$\Delta hf_2 = hf_2 + (V_2^2 / 2g - V_1^2 / 2g)$

$hf_2 = (n\ Vm_2 / Rm_2^{2/3})^2 L_2$

where,

hf_2 : Head loss due to friction

Vm_2 : Mean velocity — $Vm_2 = (V_1 + V_2) / 2$

Rm_2 : Mean hydraulic radius — $Rm_2 = (R_1 + R_2) / 2$

a) Assumed Δhf_2 — Δhf_2 = 0.018 m

b) end section of inlet flume

- Base elevation	EL_2 =	39.003 m	=	EL_1
- Width	b_2 =	3.000 m	=	b_1
- Water depth	h_2 =	2.288 m	=	$WL_1 - EL_2 - \Delta hf_2$
- Flow area	A_2 =	6.864 m	=	$b_2 h_2$
- Wetted perimeter	P_2 =	7.576 m	=	$b_2 + 2h_2$
- Hydraulic radius	R_2 =	0.906 m	=	A_2 / P_2
- Velocity	V_2 =	2.404 m/s	=	Q_1 / A_1
- Velocity head	$V_2^2 / 2g$ =	0.295 m		

c) Calculation of Δhf_2

Vm_2	=	2.395 m/s
Rm_2	=	0.908 m
hf_2	=	0.013 m

∴ Δhf_2 = 0.018 m = Assumed Δhf_2 OK

(7) Water level at end section of inlet flume

WL_2 = 41.291 m = $WL_1 - \Delta hf_2$

WL2e = 41.586 m =

3. Inlet Transition at Box Culvert

(1) Design discharge of inlet transition Q = 49.50 m^3/s

(2) Length and bed slope

- Length	L_3	=	5.500 m	
- Bed slope	I_3	=	0.182	

(3) Change of water level due to gradual enlargement

$$\Delta hge = hge + hf_3 + (V_3^2 / 2g - V_2^2 / 2g)$$

$$hge = fge (V_2^2 / 2g - V_3^2 / 2g)$$

$$hf_3 = (n\ Vm_3 / Rm_3^{2/3})^2\ L_3$$

where,

hge : Head loss due to gradual enlargement

hf_7 : Head loss due to friction

Vm_7 : Mean velocity $Vm_3 = (V_2 + V_3) / 2$

Rm_7 : Mean hydraulic radius $Rm_3 = (R_2 + R_3) / 2$

fge : Head loss coefficient due to gradual enlargement

In case, straight line formation fge = 0.00

a) Assumed Δhge Δhge = 2.678 m

b) End section of inlet transition

- Bed elevation	EL_3	=	38.003 m	
- Bed width	b_3	=	10.700 m	
- Water depth	h_3	=	0.610 m	= $WL_2 - EL_3 - \Delta hge$
- Flow area	A_3	=	6.530 m	= $b_3\ h_3$
- Wetted perimeter	P_3	=	11.921 m	= $b_3 + 2h_3$
- Hydraulic radius	R_3	=	0.548 m	= A_3 / P_3
- Velocity	V_3	=	7.580 m/s	= Q / A_3
- Velocity head	$V_3^2/ 2g$	=	2.931 m	
	EL_{3e}	=	41.545	
c) Calculation of Δhge	Vm_3	=	4.992 m/s	
	Rm_3	=	0.727 m	
	hf_3	=	0.041 m	
	hge	=	0.000 m	
∴	Δhge	=	2.678 m	= Assumed Δhge OK

(4) Water level at end section of inlet transition (BP of box culvert)

	WL_3	=	38.613 m	= $WL_2 - \Delta hge$
	WL3e	=	41.544 m	= WL2e

(5) Hydraulic Jump at end section of inlet transition

- Velocity	V_3	=	7.580 m/s	= Q / A_3
- Water depth	h3	=	0.610 m	
- Frouid Number	Fr_3	=	3.098	= $V3 /(gh_3)^{0.5}$ > Jump
- Hydraulic Jump	h3j	=	2.386 m	= $h_3/2\ x((1+8Fr_3^2)^{0.5}-1)$
- Water Surface of Jump	WL_{3j}	=	40.389 m	= L38.003+h3
- Freeboard of Jump	fb	=	0.997 m	= 0.1 x (V3+h3j)
- Inner Hight of Box Culvert	Hbox	=	3.383 m	< 3.600m ---OK
	EL_{3je}	=	40.581 m	

4. Box Culvert

(1) Discharge per barrel of conduit Q_1 = 16.50 m^3/s/barrel

(2) Head loss due to sudden contraction

A_3 / A_2'	fsc
0.0	0.50
0.1	0.48
0.2	0.45
0.3	0.41
0.4	0.36
0.5	0.29
0.6	0.21
0.7	0.13
0.8	0.07
0.9	0.01
1.0	0.00

$hsc = fsc\ V_4^2 / 2g$

where,
hsc : Head loss due to sudden contractio
fsc : Head loss coefficient due to sudden contractic
(Refer to the right table)

a) Before contractior

- Width per channel	b_3' =	3.567 m	= b_3 / N	
- Water depth	h_3' =	2.386 m	= h_{3j}	
- Flow area	A_3' =	8.510 m	= h_{3j}*'b3	
- Wetted perimeter	P_3' =	8.339 m	= h_{3j} *2+b3	
- Hydraulic radius	R_3' =	1.021 m	= A_3' / P_3'	
- Velocity	V_3' =	1.939 m/s	= Q_1 / V_3'	
- Velocity head	$V_3'^2 / 2g$ =	0.192 m		
	EL3'e	40.581		

b) After contraction (Beginning section of condui

- Base elevation	EL_4 =	38.003 m	= EL_3	
- Width of channel	b_4 =	3.300 m		
- Water depth	h_4 =	2.344 m		
- Flow area	A_4 =	7.735 m	= b_4 h_4	
- Wetted perimeter	P_4 =	7.988 m	= $b_4 + 2h_4$	
- Hydraulic radius	R_4 =	0.968 m	= A_4 / P_4	
- Velocity	V_4 =	2.133 m/s	= Q_1 / A_4	
- Velocity head	$V_4^2 / 2g$ =	0.232 m		
	EL_{4e} =	40.579		
c) Calculation of hsc	A_4 / A_3' =	0.91		
	fsc =	0.01	(Refer to the table above)	
	hsc =	0.002 m	= EL3'e - EL4	ok

(3) Length and bed slope of condui

- Length	L_5 =	85.675 m
- Bed slope	I_5 =	0.0010

(4) Head loss due to frictior

$hf_5 = (n\ Vm_5 / Rm_5^{2/3})^2 L_5$

where,
hf_5 : Head loss due to friction
Vm_5 : Mean velocity $Vm_5 = (V_4 + V_5) / 2$
Rm_5 : Mean hydraulic radius $Rm_5 = (R_4 + R_5) / 2$

a) Beginning section of conduit

- Base elevation	EL_4 =	38.003 m	
- Width of channel	b_4 =	3.300 m	
- Water depth	h_4 =	2.344 m	
- Flow area	A_4 =	7.735 m	
- Wetted perimeter	P_4 =	7.988 m	
- Hydraulic radius	R_4 =	0.968 m	
- Velocity	V_4 =	2.133 m/s	
- Velocity head	$V_4^2 / 2g$ =	0.232 m	2.576
	He =	40.579	

b) End sections of condui (UNIFLOW)

- Base elevation	EL_5 =	37.918 m		0.085
- Width	b_5 =	3.300 m	= b_4	
- Water depth	h_5 =	2.350 m	= h_4	
- Flow area	A_5 =	7.755 m	= b_5 h_5	
- Wetted perimeter	P_5 =	8.000 m	= $2b_5 + 2h_5$	
- Hydraulic radius	R_5 =	0.969 m	= A_5 / P_5	
- Velocity	V_5 =	2.128 m/s	= Q_1 / A_5	
- Velocity head	$V_5^2 / 2g$ =	0.231 m		
	He =	40.499	40.579	
c) Calculation of hf	Vm_5 =	2.131 m/s		
	Rm_5 =	0.969 m		2.581
	hf_5 =	0.080 m	= H4e-H5e OK	
	WL5 =	40.268 m		

5. Beginning Section of Outlet Flume

(1) Design discharge of outlet flume Q = 49.50 m^3/s

(2) Change of water level due to closed conduit

$\Delta h_6 = hf_6 + hse + (V_6^2 / 2g - V_5^2 / 2g)$

$hse = fse\ V_5'^2 / 2g$

where,

hf_6 : Head loss due to friction
hse : Head loss due to sudden enlargemer
fse : Head loss coefficient due to sudden enlargemer
(Refer to the right table)

A_4 / A_5	fse
0.0	1.00
0.1	0.81
0.2	0.64
0.3	0.49
0.4	0.36
0.5	0.25
0.6	0.16
0.7	0.09
0.8	0.04
0.9	0.01
1.0	0.00

a) Assumed Δh_6 Δh_6 = -0.037 m (rising)

b) Before enlargement (End sections of conduit

- Width	b_5' =	9.900 m	= $N\ b_5$
- Water depth	h_5' =	2.350 m	= h_5
- Flow area	A_5' =	23.265 m	= $N\ A_5$
- Wetted perimeter	P_5' =	24.000 m	= $N\ P_5$
- Hydraulic radius	R_5' =	0.969 m	= A_5' / P_5'
- Velocity	V_5' =	2.128 m/s	= Q / A_5'
- Velocity head	$V_5'^2 / 2g$ =	0.231 m	

40.49904

c) After enlargement

- Base elevation	EL_6 =	37.918 m	
- Width	b_6 =	10.700 m	= b_3
- Water depth	h_6 =	2.387 m	= $WL_5 - EL_6 - \Delta h_6$
- Flow area	A_6 =	25.541 m	= $b_6\ h_6$
- Wetted perimeter	P_6 =	15.474 m	= $b_6 + 2h_6$
- Hydraulic radius	R_6 =	1.651 m	= A_6 / P_6
- Velocity	V_6 =	1.938 m/s	= Q / A_6
- Velocity head	$V_6^2 / 2g$ =	0.192 m	

40.497 40.496

d) Calculation of Δhse

	A_5' / A_6 =	0.91	
	fse =	0.01	(Refer to the table above)
	hse =	0.002 m	

$\therefore$ Δh_6 = -0.037 m = Assumed Δh_6 OK

(3) Water level at beginning section of outlet flum

WL_6 = 40.305 m = $WL_5 - \Delta h_6$

6. End Section of Outlet Flume

(1) Length and bed slope

- Length	L_7	=	9.353 m	
- Bed slope	I_7	=	Level	

(2) Change of water level due to friction

$$\Delta hf_7 = hf_7 + (V_7^2 / 2g - V_6^2 / 2g)$$
$$hf_7 = (n\ Vm_7 / Rm_7^{2/3})^2\ L_7$$

where,

hf_7 : Head loss due to friction

Vm_7 : Mean velocity $\quad Vm_7 = (V_6 + V_7) / 2$

Rm_7 : Mean hydraulic radius $\quad Rm_7 = (R_6 + R_7) / 2$

a) Assumed Δhf_7 $\quad \Delta hf_7$ = 0.005 m

b) End section of outlet flume

- Bed elevation	EL_7	=	37.918 m	= EL_6
- Bed width	b_7	=	10.700 m	= b_6
- Water depth	h_7	=	2.382 m	= $WL_6 - EL_7 - \Delta hf_7$
- Flow area	A_7	=	25.486 m	= $b_7\ h_7$
- Wetted perimeter	P_7	=	15.464 m	= $b_7 + 2h_7$
- Hydraulic radius	R_7	=	1.648 m	= A_7 / P_7
- Velocity	V_7	=	1.942 m/s	= Q / A_7
- Velocity head	$V_7^2 / 2g$	=	0.192 m	

c) Calculation of Δhf_7

	Vm_7	=	1.940 m/s	
	Rm_7	=	1.650 m	
	hf_7	=	0.004 m	
∴	Δhf_7	=	0.005 m	= Assumed Δhf_7 OK

(3) Water level at end section of outlet flume

WL_7 = 40.300 m = $WL_6 - \Delta hf_7$

C.3.1.2 Hydrauric Design Statement of Outlet Drainage Structure at km. 56+227.563 Case b-2

Case b-2) Existing Canal WL.43.293 = 90cum/s

⇒ Outlet Drainage (WL.41.744=49.5cum/s) ⇒ Drainage Canal DR.15.8 Max.2011 WL.41.880m + 49.5cum/s

Inlet Gate: Full Opened or Regulated Opening **Design Discharge Qd = 49.5 m^3/s**

	Structure	Station	Distance L (m)	Discharge Q (m^3/s)	Energy Loss hl (m)	Energy Line EL. EH (m)	Velocity V (m/s)	Velocity Head Vh (m)	Water Level WL (m)	Water Depth d (m)	Bed Elevation BL (m)	Remarks
	Canal P.R. (C-1)											
0		0.00		49.500		43.293	0.000	0.000	**43.293**	4.290	39.003	
	Inlet		0.00		0.045							hl_1 = hi
1		0.00		49.500		43.248	1.323	0.089	43.159	4.156	39.003	
	Inlet Flume at Gate		10.00		0.003							$hl_2 = hf_2$
2		10.00		49.500		43.245	1.324	0.089	43.156	4.153	39.003	
	Inlet Transition of Box Culvert		5.50		0.001							$hl_3 = he + hf_3$
3		15.50		49.500		43.244	0.889	0.040	43.204	5.201	38.003	
	Inlet of Box Culvert(Pipe Flow)		0.00		0.018							hl_4 = hsc
4		15.50		49.500		43.226	1.389	0.098		3.600	38.003	
	Box Culvert (Pipe Flow)		85.68		0.040							$hl_5 = hf_5$
5		101.18		49.500		43.186	1.389	0.098		3.600	37.918	
	Outlet of Box Culvert		0.00		0.098							hl_6 = hv5
6		101.18		49.500		43.088	0.000	0.000	43.088	0.000	0.000	
	Outlet Flume		0.00		0.000							
	Total		**101.18 m**		**0.205 m**			**ΣΔWL =**	**0.205 m**			

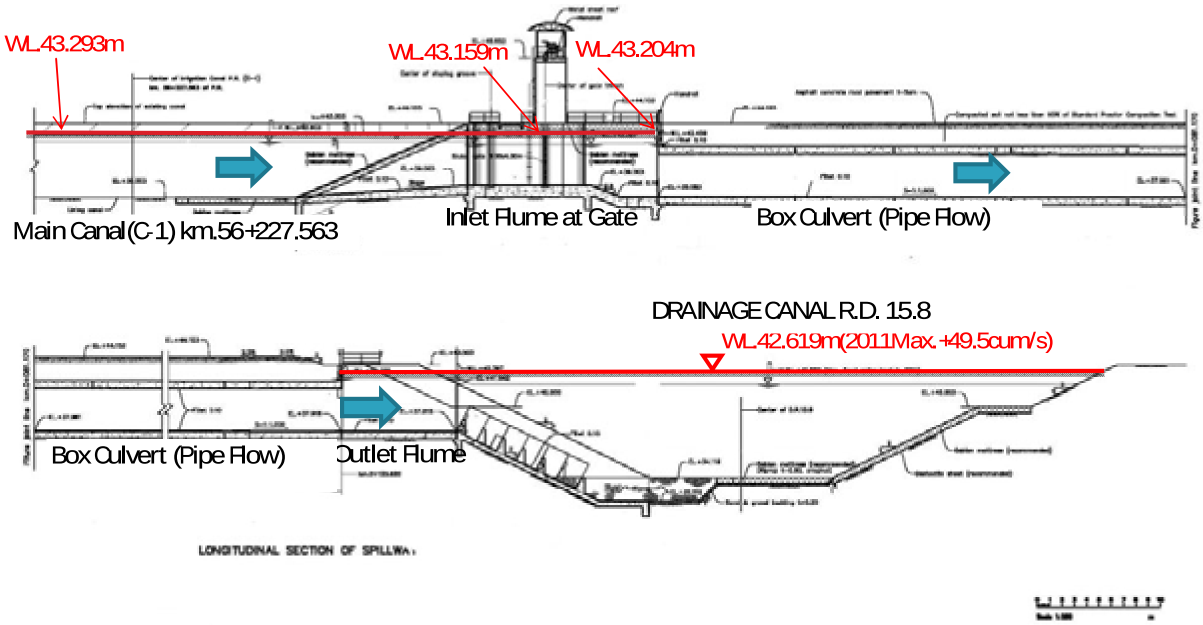
Water level at End Section of Outlet Flume **43.293 -0.205 = 43.088 m > 42.619 m**

OK

Water level at Drainage Canal D.R.15.8 (Max.2011 + 49.5cum/s) 42.619 m

Max.Flood 2011+49.5 = WL.41.880 (Q = 251.010 cum/s) + 49.5 cum/s = 300.510 cum /s ⇒WL.42.

OUTLET DREINAGE STRUCTURE



1. Design Conditions

(1) Design discharge and no. of barrel

- Design discharge	Q	=	49.500	m^3/s = Qd
Design discharge per barrel	Q_1	=	16.500	$m^3/s/barrel$
- No. of barrels of conduit	N	=	3	nos.

(2) Hydraulic conditions at beginning section (Canal P.R. (C-1) km.56+227.563)

- Water level	FWL	WL_0	=	43.293	m	
- Base elevation		EL_0	=	39.003	m	
- Water depth		h_0	=	4.290	m	
- Velocity		V_0	=	0.000	m/s	
- Roughness coefficient		n	=	0.014		(Concrete)
- Acceleration of gravity		g	=	9.8	m/s^2	

(3) Head loss coefficient at open transition

Changing condition of open transition formation	Gradual contraction coefficient fgc	Gradual enlargement coefficient fge
Rectangular to rectangular	0.10	0.20
Trapezoidal to trapezoidal	0.10	0.20
Rectangular to circular with fillet	0.20	0.30
Trapezoidal to rectangular with twisted wall	0.20	0.30
Trapezoidal to circular with twisted wall	0.30	0.40
Trapezoidal to rectangular with bended wall	0.30	0.50
Trapezoidal to circular with bended wall	0.40	0.70

2. Inlet Flume at Gate Section

(1) Discharge per barrel of inlet flume Q_1 = 16.500 m^3/s/barrel

(2) Hydraulic conditions at beginning section (Canal P.R. (C-1) km.56+227.563)

- Water level	WL_0	=	43.293	m
- Base elevation	EL_0	=	39.003	m
- Water depth	h_0	=	4.290	m
- Velocity	V_0	=	0.000	m/s
- Velocity head	$V_0^2 / 2g$	=	0.000	m

(3) Change of water level due to inflow

$$\Delta hi = hi + (V_1^2 / 2g - V_0^2 / 2g)$$
$$hi = fi (V_1^2 / 2g - V_0^2 / 2g)$$

where,
hi : Head loss due to inflow
fi : Head loss coefficient due to inflow
In case, square cut fi = 0.50

a) Assumed Δhi Δhi = 0.134 m

b) Beginning section of inlet flume (after inflow)

- Base elevation	EL_1	=	39.003	m	
- Width	b_1	=	3.000	m	
- Water depth	h_1	=	4.156	m	= $WL_0 - EL_1 - \Delta hi$
- Flow area	A_1	=	12.468	m	= $b_1 h_1$
- Wetted perimeter	P_1	=	11.312	m	= $b_1 + 2h_1$
- Hydraulic radius	R_1	=	1.102	m	= A_1 / P_1
- Velocity	V_1	=	1.323	m/s	= Q_1 / A_1
- Velocity head	$V_1^2 / 2g$	=	0.089	m	
	EL_{1e}	=	43.248	m	

c) Calculation of hi and Δhi hi = 0.045 m

∴ Δhi = 0.134 m = Assumed Δhi OK

(4) Water level at beginning section of inlet flume (after inflow)

WL_1 = 43.159 m = $WL_0 - \Delta hi$

(5) Length and bed slope of inlet flume

- Length	L_2	=	10.000	m
- Bed slope	I_2	=	Level	

(6) Change of water level due to friction

$$\Delta hf_2 = hf_2 + (V_2^2 / 2g - V_1^2 / 2g)$$
$$hf_2 = (n\ Vm_2 / Rm_2^{2/3})^2 L_2$$

where,
hf_2 : Head loss due to friction
Vm_2 : Mean velocity $Vm_2 = (V_1 + V_2) / 2$
Rm_2 : Mean hydraulic radius $Rm_2 = (R_1 + R_2) / 2$

a) Assumed Δhf_2 Δhf_2 = 0.003 m

b) end section of inlet flume

- Base elevation	EL_2	=	39.003	m	= EL_1
- Width	b_2	=	3.000	m	= b_1
- Water depth	h_2	=	4.153	m	= $WL_1 - EL_2 - \Delta hf_2$
- Flow area	A_2	=	12.459	m	= $b_2 h_2$
- Wetted perimeter	P_2	=	11.306	m	= $b_2 + 2h_2$
- Hydraulic radius	R_2	=	1.102	m	= A_2 / P_2
- Velocity	V_2	=	1.324	m/s	= Q_1 / A_1
- Velocity head	$V_2^2 / 2g$	=	0.089	m	

c) Calculation of Δhf_2

Vm_2	=	1.324	m/s
Rm_2	=	1.102	m
hf_2	=	0.003	m

∴ Δhf_2 = 0.003 m = Assumed Δhf_2 OK

(7) Water level at end section of inlet flume

WL_2 = 43.156 m = $WL_1 - \Delta hf_2$
WL2e = 43.245 m =

3. Inlet Transition at Box Culvert

(1) Design discharge of inlet transition Q = 49.50 m^3/s

(2) Length and bed slope

- Length	L_3	=	5.500	m	
- Bed slope	I_3	=	0.182		

(3) Change of water level due to gradual enlargement

$\Delta hge = hge + hf_3 + (V_3^2 / 2g - V_2^2 / 2g)$

$hge = fge (V_2^2 / 2g - V_3^2 / 2g)$

$hf_3 = (n\ Vm_3 / Rm_3^{2/3})^2 L_3$

where,

hge : Head loss due to gradual enlargement

hf_7 : Head loss due to friction

Vm_7 : Mean velocity $Vm_3 = (V_2 + V_3) / 2$

Rm_7 : Mean hydraulic radius $Rm_3 = (R_2 + R_3) / 2$

fge : Head loss coefficient due to gradual enlargement

In case, straight line formation fge = 0.00

a) Assumed Δhge Δhge = -0.048 m (rising)

b) End section of inlet transition

- Bed elevation	EL_3	=	38.003	m	
- Bed width	b_3	=	10.700	m	
- Water depth	h_3	=	5.201	m	$= WL_2 - EL_3 - \Delta hge$ >3.6m Box H
- Flow area	A_3	=	55.651	m	$= b_3\ h_3$
- Wetted perimeter	P_3	=	21.102	m	$= b_3 + 2h_3$
- Hydraulic radius	R_3	=	2.637	m	$= A_3 / P_3$
- Velocity	V_3	=	0.889	m/s	$= Q / A_3$
- Velocity head	$V_3^2/ 2g$	=	0.040	m	
	EL_{3e}	=	43.244		
c) Calculation of Δhge	Vm_3	=	1.107	m/s	
	Rm_3	=	1.870	m	
	hf_3	=	0.001	m	
	hge	=	0.000	m	
∴	Δhge	=	-0.048	m	= Assumed Δhge OK

(4) Water level at end section of inlet transition
(BP of box culvert)

	WL_3	=	43.204	m	$= WL_2 - \Delta hge$
	WL3e	=	43.244	m	= WL2e

4. Box Culvert

(1) Discharge per barrel of conduit Q_1 = 16.50 m^3/s/barrel

(2) Head loss due to sudden contraction

$$hsc = fsc\, V_4^2 / 2g$$

where,
hsc : Head loss due to sudden contraction
fsc : Head loss coefficient due to sudden contraction
(Refer to the right table)

A_3 / A_2'	fsc
0.0	0.50
0.1	0.48
0.2	0.45
0.3	0.41
0.4	0.36
0.5	0.29
0.6	0.21
0.7	0.13
0.8	0.07
0.9	0.01
1.0	0.00

a) Before contraction

- Width per channel	b_3'	=	3.567	m	= b_3 / N
- Water depth	h_3'	=	5.201	m	= h_3
- Flow area	A_3'	=	18.550	m	= h_3*'b3
- Wetted perimeter	P_3'	=	13.969	m	= h_3 *2+b3
- Hydraulic radius	R_3'	=	1.328	m	= A_3' / P_3'
- Velocity	V_3'	=	0.889	m/s	= Q_1 / V_3'
- Velocity head	$V_3'^2 / 2g$	=	0.040	m	
	EL3'e		43.244		

b) After contraction (Beginning section of conduit)

- Base elevation	EL_4	=	38.003	m	= EL_3
- Width of channel	b_4	=	3.300	m	
- Water depth	h_4	=	3.600	m	
- Flow area	A_4	=	11.880	m	= $b_4 h_4$
- Wetted perimeter	P_4	=	13.800	m	= 2 x ($b_4 + h_4$)
- Hydraulic radius	R_4	=	0.861	m	= A_4 / P_4
- Velocity	V_4	=	1.389	m/s	= Q_1 / A_4
- Velocity head	$V_4^2 / 2g$	=	0.098	m	

c) Calculation of hsc

	A_4 / A_3'	=	0.64		
	fsc	=	0.18		(Refer to the table above)
	hsc	=	0.018	m	=
	EL4e	=	43.226	m	= EL3'e-hsc4

(3) Length and bed slope of conduit

- Length	L_5	=	85.675	m
- Bed slope	I_5	=	0.0010	

(4) Head loss due to friction

$$hf_5 = (n\, Vm_5 / Rm_5^{2/3})^2 L_5$$

where,
hf_5 : Head loss due to friction
Vm_5 : Mean velocity $Vm_5 = (V_4 + V_5) / 2$
Rm_5 : Mean hydraulic radius $Rm_5 = (R_4 + R_5) / 2$

a) Beginning section of conduit

- Base elevation	EL_4	=	38.003	m	
- Width of channel	b_4	=	3.300	m	
- Water depth	h_4	=	3.600	m	
- Flow area	A_4	=	11.880	m	
- Wetted perimeter	P_4	=	13.800	m	
- Hydraulic radius	R_4	=	0.861	m	
- Velocity	V_4	=	1.389	m/s	
- Velocity head	$V_4^2 / 2g$	=	0.098	m	3.698

b) End sections of conduit (UNIFLOW)

- Base elevation	EL_5	=	37.918	m	0.085
- Width	b_5	=	3.300	m	= b_4
- Water depth	h_5	=	3.600	m	= h_4
- Flow area	A_5	=	11.880	m	= $b_5 h_5$
- Wetted perimeter	P_5	=	13.800	m	= $2b_5 + 2h_5$
- Hydraulic radius	R_5	=	0.861	m	= A_5 / P_5
- Velocity	V_5	=	1.389	m/s	= Q_1 / A_5
- Velocity head	$V_5^2 / 2g$	=	0.098	m	

c) Calculation of hf_5

	Vm_5	=	1.389	m/s	
	Rm_5	=	0.861	m	3.698
	hf_5	=	0.040	m	= OK
	EL5e	=	43.186	m	= EL4e-hf5

5. Beginning Section of Outlet Flume

(1) Drainage Canal Max 2011WL+49.5cFood WL = 42.619 m

(2) Change of water level due to closed conduit
WL_{6e} = WL of Open at Drainage Canal at D.R.15.8

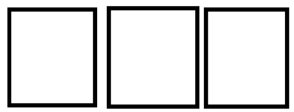
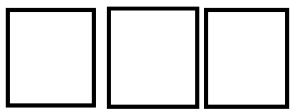
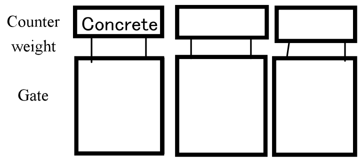
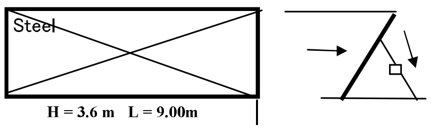
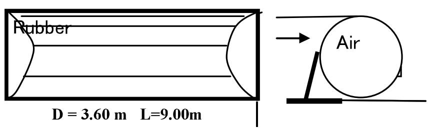
= EL_{5e} - $V_5^2 / 2g$ = 43.186 m - 0.098 m = 43.088 m = EL_{6e}

> 42.619 m

OK

C.3.1.3 Comparative Table of Gate Type for Drainage Outlet Structure at Phlai Chumphon Main Canal

Gate dimensions : 3.00 m. width x 3.60 m Height - 3 units (Total width = 9.00 m, Height = 3.60m (WL. 42.60 - EL.39.00)

Kind of Date	Sectional Figure	Merit	Demerit	Cost / Steel weight	Evaluation
Steel Sluice Gate **3.00m width x 3.60m height-3 units**		Easy in manufacturing	Rifting force is beggar than roller gate Manually operation is hard works	Steel weight 6.50 ton x 3 units =19.5 ton	
Steel Roller Gate **3.00m width x 3.60m height-3 units**		Smoothly lifting than the sluice gate	Complex structure than the slide gate	Steel weight 7.8 ton x 3 units = 23.4 ton Total cost 5,000,000 Bahts Gate 3 units 3,150,000 Bahts Log 1 unit 1,350,000 Bahts	Recommended
Steel Sector Gate **3.00m width x 3.60m height-3 units**		Smoothly lifting than the roller gate	Complex structure than the roller gate Operation & maintenance shall be hard than the roller gate.	Steel weight 7.8 ton x 3 units = 23.4 ton with concrete counter weight	
Steel Tender Gate **Steel Turning Gate** **H = 3.6 m L = 9.01m**		Automatic operation in flood without operator	For the operation of hydraulic system, electrical power attached is need. Very difficulty than the sluise/roller gate in the installation and asimbling Costly than the other gate type. Cost shall be expencibe than the others	30 ton x 1 = 30 ton / unit with oil supply units (Steel Turning Gate)	Rejected due to high cost and complex
Rubber Dam Gate **D = 3.60 m L=9.00m**		Automatic operation in flood without operator	For the operation of air supply system, electrical power attached is need. Structure life time shall be shorter than the other gate type due to material (= rubber) Not so strong than the other steel gates		Rejected due to high cost and complex

Remarks:

C.3.1.4 Structural Calculation of Conduit (3-barrel box culvert, original dimensions and modified loadings)

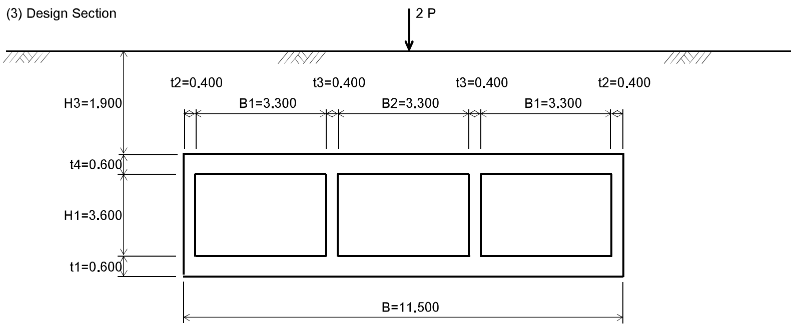
(1) Design Condition

Unit weight of reinforced concrete	γc :	2.40	(t/m3)
Unit weight of soil (saturated)	γsat :	2.15	(t/m3)
Unit weight of soil (wet)	γs :	1.90	(t/m3)
Unit weight of soil (under water)	γ ' :	1.15	(t/m3)
Unit weight of water	γw :	1.00	
Internal friction angle	Φ :	30	(degree)
seismic coefficient	k :	0.00	
Coefficient of earth pressure (Normal)	ka :	0.3333	
Live load (Truck loading)	P :	8.40	(t/wheel)

(2) Loading Condition

Case 1 : Normal Condition (Groundwater: not considered, Internal water depth Empty)

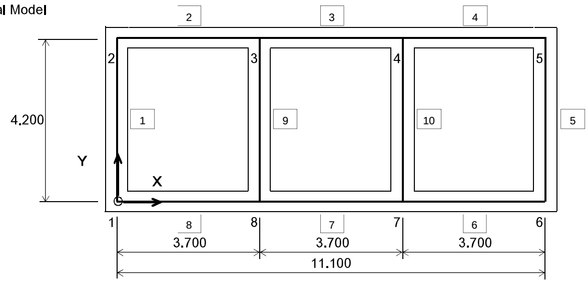
(3) Design Section



Design Dimensions

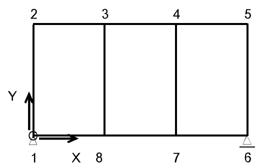
Whole width of culvert	B =	11.500	(m)	
Unit length of culvert	L =	1.000	(m)	
Internal width of channel	B1 =	3.300	(m)	
	B2 =	3.300	(m)	
Internal height of side wall	H1 =	3.600	(m)	
Internal water depth (Empty)	H2 =	0.000	(m)	
Soil depth above top slab	H3 =	1.900	(m)	
Ground water depth from top slab	H4 =	-	(m)	(not considered)
Thickness of bottom slab	t1 =	0.600	(m)	
Thickness of side wall	t2 =	0.400	(m)	
Thickness of partition wal	t3 =	0.400	(m)	
Thickness of top slab	t4 =	0.600	(m)	

(4) Structural Model



(a) Coordinate

Joint No	X (m)	Y (m)	Support		
			X	Y	θ
1	0.000	0.000	1	1	
2	0.000	4.200			
3	3.700	4.200			
4	7.400	4.200			
5	11.100	4.200			
6	11.100	0.000		1	
7	7.400	0.000			
8	3.700	0.000			



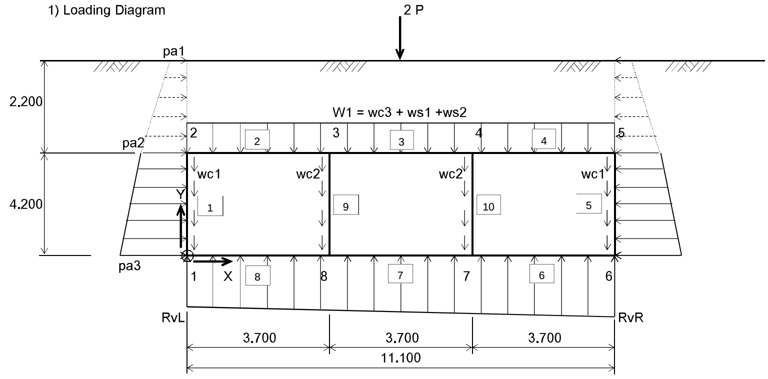
(b) Dimension, Moment of Inertia and Dead Load

Member	Joint	Dimension	Area	I	Dead Load	Weight	Cover of Re-bar	
	I - J	h(m) x b(m)	(m2)	(m4)	(t/m)	(t)	upper/left	lower/right
1	1 - 2	0.40 x 1.00	0.400	0.00533	0.96	4.03	5.0	5.0
2	2 - 3	0.60 x 1.00	0.600	0.01800	1.44	5.33	5.0	5.0
3	3 - 4	0.60 x 1.00	0.600	0.01800	1.44	5.33	5.0	5.0
4	4 - 5	0.60 x 1.00	0.600	0.01800	1.44	5.33	5.0	5.0
5	5 - 6	0.40 x 1.00	0.400	0.00533	0.96	4.03	5.0	5.0
6	6 - 7	0.60 x 1.00	0.600	0.01800	1.44	5.33	5.0	5.0
7	7 - 8	0.60 x 1.00	0.600	0.01800	1.44	5.33	5.0	5.0
8	8 - 9	0.60 x 1.00	0.600	0.01800	1.44	5.33	5.0	5.0

9	9 - 10	0.40 x 1.00	0.400	0.00533	0.96	4.03	5.0	5.0
10	10 - 0	0.40 x 1.00	0.400	0.00533	0.96	4.03	5.0	5.0

(5) Case 1 : Normal Condition (Groundwater: not considered, Internal water depth: empty)

1) Loading Diagram



wc1 = 0.960 (t/m)	W1 = wc3+ws1+ws2	RvL = 7.483 (t/m)
wc2 = 0.960 (t/m)	= 6.030 (t/m)	RvR = 7.483 (t/m)
wc3 = 1.440 (t/m)	Pa2 = 1.393 (t/m)	
wc4 = 1.440 (t/m)	Pa3 = 4.053 (t/m)	
ws1 = 0.980 (t/m)		
ws2 = 3.610 (t/m)		

2) Calculation of Loading

i) Self-weight of Concrete

Member 1,5	wc1 =	0.96 (t/m)
Member 9,10	wc2 =	0.96 (t/m)
Member 2,3,4	wc3 =	1.44 (t/m)
Member 6,7,8	wc4 =	1.44 (t/m)

ii) Active Earth Pressure

	ka	γs	h	q	pa
pa1 = ka(γs x h1 + q)	0.3333	1.90	0.000	0.000	0.000 (t/m)
pa2 = pa1 + ka(γs x h2)	0.3333	1.90	2.200		1.393 (t/m)
pa3 = pa2 + ka(γs x h3)	0.3333	1.90	4.200		4.053 (t/m)

iii) Live load ws1 = 2 P / 17.14 ws1 = 0.980 (t/m)

iv) Weight of Soil ws2 = H3×γs ws2 = 3.610 (t/m)

v) Foundation Reaction

Total width Bo = 11.10 (m)

Total weight of live + soil Ws = (ws1+ws2) x Bo Ws = 50.95 (t)

Calculation of Moment (Origin : node 1)

Load		V(t)	H(t)	X(m)	Y(m)	Mx (tm)	My (tm)
Concrete	Member 1	4.03		0.00		0.00	
	Member 2	5.33		1.85		9.86	
	Member 3	5.33		5.55		29.57	
	Member 4	5.33		9.25		49.28	
	Member 5	4.03		11.10		44.76	
	Member 6	5.33		9.25		49.28	
	Member 7	5.33		5.55		29.57	
	Member 8	5.33		1.85		9.86	
	Member 9	4.03		3.70		14.92	
	Member 10	4.03		7.40		29.84	
Live load + soil		50.95		5.55		282.77	
Water		0.00		5.55		0.00	
Earth pressure	upper(L)		8.51		2.80		23.83
	upper(R)		-8.51		2.80		-23.83
Total		99.05	0.00			549.71	0.00
					ΣM =	549.71 (tm)	

Total width Bo = 11.10 (m)

Unit length Lo = 1.00 (m)

Eccentric distance

e = ΣM/ΣV - Bo/2 e = 0.00 (m)

Foundation reaction

Rv = {ΣV/(BoxLo)} x (1 ± 6e/Bo) - wc4 RvL = 7.483 (t/m)

RvR = 7.483 (t/m)

Foundation Reaction at each node

Node	X (m)	Rv (t/m)
1	0.00	7.483
6	3.70	7.483
6	7.40	7.483
6	11.10	7.483

Table A.4F.1a. Calculation Result of Sectional Force

Member	Point	B	H	D	DD	Case 1		
						M	S	N
		(cm)	(cm)	(cm)	(cm)	(t .m)	(t)	(t)
1, 5	upper out-Mmax	100	40	35	5	-2.876	-3.989	-9.473
1, 5	lower out-Mmax	100	40	35	5	-6.231	7.448	-13.505
1, 5	in-Mmax	100	40	35	5	1.471	0.000	-11.370
2, 4	edge out-Mmax	100	60	55	5	-2.876	9.473	-3.989
2, 4	mid.jt. out-Mmax	100	60	55	5	-9.100	-12.838	-3.989
2, 4	in-Mmax	100	60	55	5	4.565	0.000	-3.989
3	out-Mmax	100	60	55	5	-8.273	11.156	-3.652
3	in-Mmax	100	60	55	5	2.045	0.000	-3.652
9, 10	right	100	40	35	5	-0.586	0.336	-28.025
9, 10	left	100	40	35	5	0.827	0.336	-23.993
6, 8	edge out-Mmax	100	60	55	5	-6.231	13.505	-7.448
6, 8	mid.jt. out-Mmax	100	60	55	5	-7.482	-14.182	-7.448
6, 8	in-Mmax	100	60	55	5	5.957	0.000	-7.448
7	out-Mmax	100	60	55	5	-8.068	13.844	-7.784
7	in-Mmax	100	60	55	5	4.738	0.000	-7.784

Sectional Force for Stress Calculation

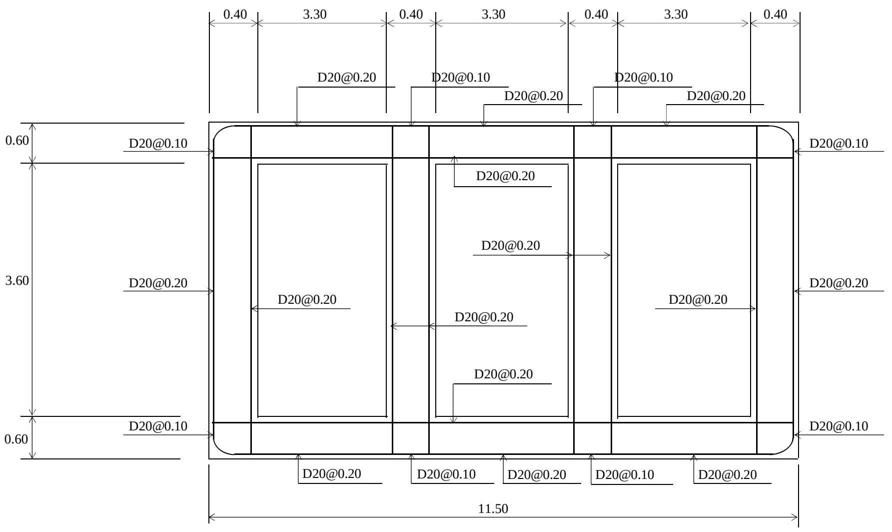
Member		Point	M	S	N	B	H	D	DD
			(t.m)	(t)	(t)	(cm)	(cm)	(cm)	(cm)
Side wall	1,5	upper outside	2.876	3.989	9.473	100	40	35	5
		lower outside	6.231	7.448	13.505	100	40	35	5
		inside	1.471	0.000	11.370	100	40	35	5
Partition wall	9,10	both sides	0.827	0.336	23.993	100	40	35	5
Top slab	2, 3,4	upper edge jt	2.876	9.473	3.989	100	60	55	5
		upper mid. jt	9.100	12.838	3.989	100	60	55	5
		lower	4.565	0.000	3.989	100	60	55	5
Bottom slab	6,7,8	lower edge jt	6.231	13.505	7.448	100	60	55	5
		lower mid. jt	8.068	14.182	7.784	100	60	55	5
		upper	5.957	0.000	7.448	100	60	55	5

Arrangement of Reinforcement Bar

(1) Perpendicular direction to river flow (parallel to weir axis)

Distributing bar: D16@0.20

<Cover>
All member of
framwork : 5cm



STRESS CALCULATION AND BAR ARRANGEMENT

Box Culvert (Original Dimensions and Modified Loadings)

Structure Member			Side wall (upper outside)	Side wall (lower outside)	Side wall (inside)	Partition wall (both sides)	Top slab (upper edge jt)	Top slab (upper mid. jt)	Top slab (lower)	Bottom slab (lower edge jt)	Bottom slab (lower mid. jt)	Bottom slab (upper)
Moment	M	(t.m)	2.876	6.231	1.471	0.827	2.876	9.100	4.565	6.231	8.068	5.957
Shear	S	(t)	3.989	7.448	0.000	0.336	9.473	12.838	0.000	13.505	13.844	0.000
Axial Force	N	(t)	9.473	13.505	11.370	23.993	3.989	3.989	3.989	7.448	7.784	7.488
Width	B	(cm)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Height	H	(cm)	40.0	40.0	40.0	40.0	60.0	60.0	60.0	60.0	60.0	60.0
Effective Height	d	(cm)	35.0	35.0	35.0	35.0	55.0	55.0	55.0	55.0	55.0	55.0
Cover for comp.	d'	(cm)	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Re-bar (tension)	As	(cm2)	31.42	31.42	15.71	15.71	31.42	31.42	15.71	31.42	31.42	15.71
Re-bar arrange			D20 @100	D20 @100	D20 @200	D20 @200	D20 @100	D20 @100	D20 @200	D20 @100	D20 @100	D20 @200
Re-bar (comp.)	As'	(cm2)										
Re-bar arrange												
Compression	σ_c	(kg/cm2)	14.02	28.81	8.95	9.10	6.09	17.92	11.47	13.02	16.58	15.19
Tension	σ_s	(kg/cm2)	194.02	500.92	58.75	0.00	142.65	556.66	478.99	323.06	440.61	565.61
Shear	τ	(kg/cm2)	1.45	2.64	0.00	0.11	2.07	2.73	0.00	2.94	2.99	0.00
Allowable stress	σ_{ca}	(kg/cm2)	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0
	σ_{sa}	(kg/cm2)	1,500.0	1,500.0	1,500.0	1,500.0	1,500.0	1,500.0	1,500.0	1,500.0	1,500.0	1,500.0
	τ_a	(kg/cm2)	4.2	4.2	4.2	4.2	4.2	4.2	4.2	4.2	4.2	4.2
Note						Full compression *						

Royal Irrigation Department

Ministry of Agriculture and Cooperatives

Survey-Design
of
The Improvement of Phlai Chumpol O&M Project
Phitsanulok Province

Contract No. Jor.06/2552 (ฝพพ.4)

Design Criteria

Consultants

Panya Consultant Co., Ltd.

Resources Engineering Consultants Co., Ltd.

Frontier Engineering Consultants Co., Ltd.

June 2010

The Improvement of Phlai Chumpol O&M Project
Phitsanulok Province

Design Criteria

Content

Content (Continued)

Page

Content (Continued)

Content (Continued)

Content (Continued)

Page

Table Content

Table No. **Page**

Figures Content

Figures Content (Continued)

Figure No. **Page**

Thai - English translation

CHAPTER 1
INTRODUCTION

1.1 Background

The Phlai Chumpon Operation and Maintenance Project is part of the Phitsanulok Irrigation Project, covering the total irrigation area of 218,000 rai which comprises some parts of districts in Phitsanulok Province: Phrom Phiram, Mueang Phitsanulok, Bang Rakam, and Bang Krathum; and some parts of districts in Phichit Province: Mueang Phichit and Sam Ngam. Its source of water is the Naresuan Dam that conveys water through one main open channel, shared with the Dong Sethi Irrigation and Maintenance Project and the Tha Bua Irrigation and Maintenance Project located downstream, before distributing water to its irrigation area via lateral canals. Owing to geographic constraints of the project area, the lengths of the main open channel and those of the lateral canals are quite long, since most of them are earth canals, so the irrigation in the project area never fulfills its full designed efficiency. Also, the operation and the management have not met the farmers' requirements and problems remain – flood/inundation in rainy season, shortage of water in some areas in dry season, and other problems that affect the project.

The Royal Irrigation Department has hired a group of consulting engineers: the Resources Engineering Consultants Co., Ltd., the Visuth Consultant Co., Ltd., and the Frontier Engineering Consultants Co., Ltd. to do the feasibility study on the Phlai Chumpon Irrigation and Maintenance Project. The said study was completed in August 2007. The study can be taken as guideline for development of the project in terms of structures and irrigation/drainage systems, management and procedures as well as mutual agreements between farmers and relevant government agencies ("the Hardware") or among groups of farmers ("the Software"), and personnel relating to the project e.g. government officials, farmers, groups of water users, and non-government organizations in the area ("the Human-ware"). The aim is to develop the project efficiency according to its specified potential.

The feasibility study of the Phlai Chumpon Operation and Maintenance Project comprises the estimation of water distribution and drainage status of the project at present and in the future (5, 10, and 20 years ahead) with respect to water management of the neighboring projects (Dong Sethi and Tha Bua Projects); setting up an integral agricultural plan; studying the actual volume of water for irrigation based on the existing data and preparing an operation manual; preparing the operation plan and the investment plan to increase control efficiency in conveying water for thorough, fair and economical distribution, and causing minimal effect to the projects downstream; analyzing the institution of water users for agricultural purpose; and arousing community strength through group dynamic.

According to the aforesaid feasibility study it indicates that the following issues to be developed:

- Development of the main open channel (C1) – Construction and reinstatement of open channel beds with concrete lining (wire mesh reinforcement) from km 0+045 to km 80+000 and development of 116 farm turnouts, installation of water drain gates and cranes, construction of nine reinforced concrete bridges across the main open channel, each with 6-m-wide traffic surface and 1.50-m-wide footpath on both sides;
- Development of lateral and sub-lateral canals – widening six concrete-lined canals and develop 11 irrigation structures to cope with the desired water demand, and concrete lining 21 canals to upgrade their efficiency in water distribution;
- Development of eight head regulators of lateral and sub-lateral canals; and
- Solving problem of repetitious flood by installing drainage pumps – construction of 15 pumping stations and three tail regulators.

1.2 Objective

The objective of this project is to upgrade the efficiency of irrigation and drainage systems by developing earth canals into concrete-lined canals; increasing the capacity of the canals and concrete lining them, modifying irrigation structures matching with water demand; developing the drainage system as well as the embankments to solve problems of repetitious flood/inundation for productive and consumptive uses; and upgrading people's quality of life.

1.3 Project location

The Phlai Chumpon Operation and Maintenance Project located at 204, Moo 8, Thathong Sub-district, Mueang Phitsanulok District, Phitsanulok Province, or at coordinate 47 QPU 288557 of map sheet 5042 IV. At km 125 on highway 117 from Nakhon Sawan City, turn to the right and go straight for another 350 m, there the project office. It shares the area with the Irrigation Office 3 of latitude 16° 45′ 55″ and 100° 12′ 31″, about 8 km away from the city hall of Phitsanulok.

The Phlai Chumpon Operation and Maintenance Project, as shown in Fig. 1.3-1, has the irrigation area under its responsibility of 218,000 rai covering some parts of districts in Phitsanulok Province: Phrom Phiram, Mueang Phitsanulok, Bang Rakam, and Bang Krathum; and some parts of districts in Phichit Province: Muaeng Phichit and Sam Ngam. Its source of water is the Naresuan Dam that conveys water through one main open channel and distributes to its irrigation area via lateral canals.

The development scheme covers the following works:

1. Design and development of the main open channel
   - Design and development of an open channel, 80 km
   - Design and development of farm turnouts, 116 sites
   - Design and development of reinforced concrete bridges, 9 sites
   - Design of a spillway, 1 site
   - Design and development of floating-weed prevention system at the inlet of the main open channel C1, 1 site
   - Design and development of siphon, 2 sites

Fig. 1.3-1 Map showing the irrigation area under responsibility of the Phlai Chumpon Operation and Maintenance Project

2. Design and development of lateral canals
   - Design and development of canals, 129 km
   - Design and development of head regulators, 4 sites
   - Design and development of farm turnouts, 164 sites
   - Design and development of road-crossing culverts, 9 sites
   - Design and development of drop structures, 3 sites
   - Design and development of intermediate regulators, 8 sites

3. Design and development of sub-lateral canals
   - Design and development of sub-lateral canals, 93 km
   - Design and development of head regulators, 4 sites
   - Design and development of farm turnouts, 111 sites

4. Design and development of water drainage for flood mitigation
   - Design of pumping stations, 15 sites
   - Design and development of tail regulators, 20 sites
   - Design and development of the existing pumping stations, 3 sites
   - Design and development of draining canals and their ridges as necessary and appropriate pursuant to the engineering feasibility reviews.

5. Design of an office building, 1 site

1.4 Scope of work

According to employment contract no. Jor. 06/2552 (FPP.4), the consulting engineer shall survey and design the following works:

1. Review the hydrological aspect and groundwater potential of the project;
2. Review the water demand of the project;
3. Review the engineering feasibility and the design concept:
   - Study the alternatives to specify construction and development approaches of the main open channel C1;
   - Study the drainage and dikes of the project on:
     (1) Sufficiency of drainage canals and structures, including heights of dikes;
     (2) Types and feasibility of tail regulators and pumping stations.
   - Study the alternatives to design additional water disposal structures in order that the main open channel C1 can act as bypass waterway of the Nan River near to Phitsanulok City; and

- Study and review the sizes of canals C28 PR to canals C58.7R and their fixtures corresponding to project water management and water diversion to the Yom River.

4. Review the data and do additional engineering surveys;
5. Design the development of irrigation and drainage systems as well as their fixtures;
6. Design the development of access ways for maintaining the irrigation systems and their fixtures;
7. Calculating the quantity of construction work, preparing the bill of quantity, and estimating the cost of construction;
8. Specifying the technical requirements;
9. Duplicating the approved drawings by photocopying and book binding;
10. Organizing public activities; and
11. Organizing a summary seminar of the detailed design procedures to concerning RID officers.

1.5 Project duration

The development scheme is limited to complete within 780 days, from 15 May 2009 to 3 July 2011.

Thai - English translation

CHAPTER 2
IRRIGATION DEVELOPMENT DESIGN CRITERIA

1.1 Irrigation aspect

The Phlai Chumpon Operation and Maintenance Project is a large irrigation project by means of gravity, whereas the Nan River is its capital source of water and the Naresuan Dam supplies water to it via a gate at the intake of the main open channel (C1) which situates on the right bank of the river, that controls the flow of water into it. The main open channel then distributes water to three projects: the Phlai Chumpon Operation and Maintenance Project, the Dong Sethi Operation and Maintenance Project, and the Tha Bua Operation and Maintenance Project. The 80-km-long upper portion of the open channel belongs to the Phlai Chumpon Operation and Maintenance Project together with its 19 lateral canals (on the right side of the open channel) with total length 129 km plus 19 sub-lateral canals with the total length of 93 km. The project irrigation area is about 218,000 rai covering some parts of districts in Phitsanulok Province: Phrom Phiram, Mueang Phitsanulok, Bang Rakam, and Bang Krathum; and some parts of districts in Phichit Province: Mueang Phichit and Sam Ngam.

The Phlai Chumpon Operation and Maintenance Project divide its organization into three divisions (Water Distribution and Maintenance Divisions 1, 2 and 3). These divisions control the operation and the maintenance of irrigation systems in their areas of responsibility, coordinate with district offices and farmers in order to solve problems and contradictions in water distribution; and instruct them of the correct uses of water, including the repair and the maintenance of water distribution and drainage systems.

<u>Area under responsibility of Water Distribution & Maintenance Division 1</u>

The total irrigation area under responsibility of Water Distribution & Maintenance Section 1 is 80,962 rai where 12 canals with 259 fixtures exist.

Item	Canal	Length (km)	No. of fixtures
1	PR (C1)	19.100	47
2	PR-5.5-3R (C2)	9.600	30
3	PR-5.3R-4.2R (C3)	5.020	24
4	PR-5.3R-4.2R-0.4L (C4)	5.125	13
5	PR-8.7R (C5)	12.334	36
6	PR-8.7R-5.2R (C6)	2.750	10
7	PR-8.7R-5.5L (C7)	3.381	9
8	PR-8.7R-8.5L (C8)	4.651	10
9	PR-10.6R (C9)	6.180	23
10	PR-17.0R (C10)	9.069	32
11	PR-17.0R-3.6R (C11)	5.080	15
12	PR-17.0R-3.6L (C12)	3.440	10

Area under responsibility of Water Distribution & Maintenance Division 2
The total irrigation area under responsibility of Water Distribution & Mai ntenance Section 2 is 70,869 rai where 13 canals with 324 fixtures exist.

Item	Canal	Length (km)	No. of fixtures
1	PR (C1)	30.400	90
2	PR-22.2R (C13)	6.297	13
3	PR-23.7R-4.2R (C14)	11.052	45
4	PR-25.0R (C15)	15.750	61
5	PR-36.9R (C16)	3.974	10
6	PR-40.1R (C17)	4.920	20
7	PR-40.1R-2.8R (C18)	5.438	13
8	PR-40.5R (C19)	3.720	14
9	PR-40.5R-2.6L (C20)	4.780	16
10	PR-40.5R-2.6R-2.6L (C21)	6.260	18
11	PR-40.5R-3.6L (C11)	2.312	5
12	PR-44.5R-3.6L (C23)	4.068	13
13	PR-44.5R-1.1L (C24)	2.400	6

Area under responsibility of Water Distribution & Maintenance Division 3
The total irrigation area under responsibility of Water Distribution & Mai ntenance Section 2 is 66,122 rai where 16 canals with 296 fixtures exist.

Item	Canal	Length (km)	No. of fixtures
1	PR (C1)	30.520	71
2	PR-50.0R (C25)	3.828	9
3	PR-50.0R-1.3R (C26)	2.870	8
4	PR-58.2R (C27)	6.424	17
5	PR-58.7R (C28)	3.665	10
6	PR-58.7R -1.1R (C29)	6.220	16
7	PR-58.7R -1.1R -3.3L (C30)	1.740	6
8	PR-61.6R (C31)	2.930	5
9	PR-64.0R (C32)	1.640	4
10	PR-65.3R (C33)	2.378	8
11	PR-70.5R (C34)	4.370	12
12	PR-72.5R (C35)	18.140	58
13	PR-72.5R -1.2R (C36)	4.020	12
14	PR-72.5R -3.3L (C37)	3.220	9
15	PR-72.5R -3.7R (C38)	18.220	49
16	PR-72.5R -5.1R (C36)	1.720	2

There are some changes in water supply shift scheme 2009/2010 of the Phlai Chumpon Operation and Maintenance Project, for instance, water supply shift is extended from seven to nine days. Such change varies with problematic situations of the current water distribution system and the volume of water to be allocated. If the new water distribution system is applied to the 7-day-shift scheme the size of the developing distribution system would be applicable to either 7-day or 9-day shift.

In case the flow of capital water increases or the incoming water is 60 m^3/sec or more and with the 24-hour supply scheme the control of water conveyance and distribution would be fine.

2.2.1 Water conveyance of the main open channel

Based on the 21-day water shift scheme, the Phlai Chumpon Operation and Maintenance Project will be supplied for seven days, and the Dong Sethi Operation and Maintenance Project as well as the Tha Bua Operation and Maintenance Project will be supplied for 14 days. Thus the main open channel will convey water at all the time. If the existing open channel is developed from earth canal to concrete-lined one without modification of its cross-section, it would be able to convey water in shifts during dry season.

2.2.2 **Water conveyance of lateral and sub-lateral canals**

Given that the 21-day shift basis is applied to 19 lateral canals and 19 sub-lateral canals i.e. seven days for the Phlai Chumpon Operation and Maintenance Project (some of its lateral canals and sub-lateral canals may be supplied less than seven days) and 14 days for the Dong Sethi Operation and Maintenance Project and the Tha Bua Operation and Maintenance Project.

2.3 Water duty and design irrigation flow

As the dry season is the peak demand of irrigation water, so it is the period to be taken into consideration i.e. adopting the average irrigation water demand during dry season in designing the irrigation system to be developed, which is equal to 0.36 liter/second/rai. Such figure is appropriate for the design because it is the average of actual demand and not less than the capital flow in dry season of water richness year. It is flexible if some minor areas have different demands while those of vast areas are not so different from the average. The water distribution shall be done in short shifts for water usage in longer period at average flow.

2.3.1 Design irrigation flow rate of the main open channel

Given that the main open channel conveys water in shifts in operation. So, to determine the design irrigation flow of the main open channel the average demand of irrigation water in dry season (0.36 liter/second/rai) shall be multiplied by the irrigation area under responsibility. The derived design irrigation flow rate shall be compared with the calculated flow with respect to the cross-sectional area of the main open channel after concrete lining. Then choose the higher value as the design flow of the main open channel.

The lower portion of the main open channel of the Phlai Chumpon Operation and Maintenance Project shall be able to convey water to the Dong Sethi Operation and Maintenance Project and the Tha Bua Operation and Maintenance Project once their shift arrives. Thus, the main open channel at its transition zone shall be able to convey water not less than the shift flow or about 60 m^3/sec. However, the existing design flow at the outlet of the main open channel is greater than 60 m^3/sec, so this figure can be taken as the minimum design irrigation flow rate.

2.3.2 Design irrigation flow rates of lateral and sub-lateral canals

The design irrigation flow rates of lateral and sub-lateral canals can be determined with respect to the seven-day main shift and the internal sub-shift distributions, starting with the average demand of irrigation water in dry season (0.36 liter/second/rai) and adjust the total shift flow to about 60 m^3/sec. Choose the design flow of each portion of the canal to calculate the dimensions of the canal as summarized in Table 2.3.2-1. The required design flow affects the dimensions of the canal i.e. it is flexible at actual conveyance due to the demand of each canal is not simultaneous.

2.4 Irrigation area and water distribution to scattered areas

The development design of the Phlai Chumpon Operation and Maintenance Project basically relies on the existing data of its irrigation area to the extent of water distribution to scattered areas. Dimensional revision of each scattered area is based on the updated data: topographical map, scale 1:10,000, aerial photographs and field explorations.

Table 2.3.2-1 Design flow in portion-by-portion manner of the canals in the Phlai Chumpon Operation and Maintenance Project (see pp. 2-7 to 2-13 of the original)

2.5 Design of canals' dimensions and capacities

2.5.1 Dimensions and capacity of the main open channel

The typical drawing of the main open channel's cross-section is shown in Fig. 2.5.1-1, based on its existing dimensions and capacity. It will remain unchanged if it is still applicable. Development design of other parts shall comply with the following criteria:

(1) Concrete lining the existing open channel surface; side slope 1:2;
(2) The width of asphaltic-concrete-paved ridge used as road for maintenance purpose shall be the same or at least 8.00 m. with side slope 4% toward the external area, whereas the width of the ridge on the opposite side shall be 2.00 m. Cultivate grasses on both side slopes of the open channel;
(3) The internal side slope of the ridge equals that of the open channel and cultivate grasses on it;
(4) The external side slope of the ridge is 1:2 and cultivate grasses on it;

(5) The width of the berm on both sides shall be 1.00 m;
(6) Longitudinal slope of the open channel bed is more or less to that of the existing one;
(7) The highest water level in the concrete-lined open channel shall be the same as that of the earth canal;
(8) Apply Manning formula to determine the cross-sectional dimensions of the open channel:

$$Q = 1/n\ AR^{2/3}\ S^{1/2} \qquad (2\text{-}1)$$

where Q is the flow of water in the open channel (m^3/sec)
A is the cross-sectional area of flow (m^2)
n is the Manning coefficient of roughness
= 0.018 (for concrete-lined open channel)
R is the hydraulic radius (m) = A/P
P is the wetted perimeter (m)
S is the slope of the open channel bed at uniform flow

(*Open-Channel Hydraulics*, Richard H. French, 1987, p. 164)

(9) Adopt the ratio of open channel bed (B) to depth of water (D) between 1 and 2;
(10) The minimum freeboard of the concrete-lined open channel varies with the flow as shown in the table below.

Fig. 2.3.2-1 Drawing of a typical cross-sectional area of an open channel (see p. 2-15 of the original)

Flow (m^3/sec)	Freeboard (m)	Freeboard behind the crest (m)
< 1.00	0.15	0.45
1.00 – 2.50	0.20	0.60
2.51 - 5.00	0.25	0.70
5.01 – 7.50	0.30	0.80
7.51 - 10.00	0.35	0.85
10.01 - 12.00	0.35	0.90

(*Design of Small Structures*, USBR, 1978, p. 15)

If the freeboard and the freeboard behind the crest in the table are less than the freeboard of the existing earth canal then adopt the freeboard of the concrete-lined open channel equal to that of the earth canal.

(11) The thickness of concrete lining and length of concrete panel shall comply with the following recommendations:

Flow (m^3/sec)	Concrete panel thickness (cm)	Curb of ridge (cm)	Length of concrete panel (m)
< 2.5	6	15	3
2.5 - 10.0	8	20	3
> 10.0	10	25	3

Remark: 10-cm-thick concrete lining shall be reinforced with wire mesh

(12) In case the open channel bed is 60 cm less than the ground level, weep holes and filters shall be embedded under the concrete panels of the bed and the side slope of the open channel at every 3.00-m interval or less. The weep holes shall be located along the centerline of the concrete panels.

2.5.2 Dimensions and capacity of the lateral/sub-lateral canals

The typical drawing of a lateral/sub-lateral canal's cross-section is shown in Fig. 2.5.1-2. The dimensions and the capacity of the existing lateral/sub-lateral canal shall be taken into consideration as first priority. It will remain unchanged if it is still applicable. Development design of other parts shall comply with the following criteria:

Fig. 2.3.2-1 Drawing of a typical cross-sectional area of a lateral/sub-lateral canal
(see p. 2-17 of the original)

(1) Concrete lining the existing canals with side slope 1:1.5;
(2) The width of former laterite-impacted ridge used as road for maintenance purpose shall be the same or at least 3.50 m. with side slope 4% toward the external area, whereas the width of the ridge on the opposite side shall be 2.00 m. Cultivate grasses on both side slopes of the open channel;
(3) The internal side slope of the ridge equals that of the open channel and cultivate grasses on it;
(4) The external side slope of the ridge is 1:2 and cultivate grasses on it;
(5) The width of the berm on both sides shall be 1.00 m;
(6) Longitudinal slope of the canal bed is more or less to that of the existing one;
(7) The highest water level in the concrete-lined canal shall be more or less to that of the earth canal. Try to enlarge the canal to the opposite side of the road as much as possible in order to avoid any effect to the ridge and the farm turnout beside the road;
(8) Apply Manning formula to determine the cross-sectional dimensions of the open channel:

$$Q = 1/n\ AR^{2/3}\ S^{1/2} \qquad (2\text{-}2)$$

where Q is the flow of water in the open channel (m^3/sec)
A is the cross-sectional area of flow (m^2)
n is the Manning coefficient of roughness
= 0.018 (for concrete-lined open channel)
R is the hydraulic radius (m) = A/P
P is the wetted perimeter (m)
S is the slope of the open channel at uniform flow

(*Open-Channel Hydraulics*, Richard H. French, 1987, p. 164)

(9) Adopt the ratio of open channel bed (B) to depth of water (D) between 1 and 2;

(10) The minimum freeboard of the concrete-lined open channel varies with the flow as shown in the table below.

Flow (m^3/sec)	Freeboard (m)	Freeboard behind the crest (m)
< 1.00	0.15	0.45
1.00 – 2.50	0.20	0.60
2.51 - 5.00	0.25	0.70
5.01 – 7.50	0.30	0.80
7.51 - 10.00	0.35	0.85
10.01 - 12.00	0.35	0.90

(*Design of Small Structures*, USBR, 1978, p. 15)

If the freeboard and the freeboard behind the crest in the table are less than the freeboard of the existing earth canal then adopt the freeboard of the concrete-lined canal equal to that of the earth canal.

(11) The thickness of concrete lining and length of concrete panel shall comply with the following recommendations:

Flow (m^3/sec)	Concrete panel thickness (cm)	Curb of ridge (cm)	Length of concrete panel (m)
< 2.5	6	15	3
2.5 - 10.0	8	20	3
> 10.0	10	25	3

Remark: 10-cm-thick concrete lining shall be reinforced with wire mesh

(12) In case the canal bed is 60 cm less than the ground level, weep holes and filters shall be embedded under concrete panels of the bed and the side slope of the open channel at every 3.00-m interval or less. The weep holes shall be located along the centerline of the concrete panels.

2.6 Design of irrigation structures to be developed

2.6.1 Irrigation structures to be developed

The design for development of irrigation structures of the Phlai Chumpon Operation and Maintenance Project covers the development of the existing structures and the construction of new structures in the neighborhood or replacement of the existing structures if they do not meet the new design criteria. In general, the existing structures, if they still serve well, will be developed only some of its part(s).

List of structure types to be developed or constructed

Item	Type of irrigation structure	Development req'd	design & construction req'd
1	Head regulator	Yes	Yes
2	Check structure	No	Yes
3	Drop structure	No	Yes
4	Road-crossing culvert	No	Yes
5	Siphon	No	Yes
6	Flume	Yes	No
7	Farm turnout	Yes	Yes
8	Tail regulator	No	Yes
9	Drain culvert	Yes	Yes
10	Bridge	Yes	Yes

2.6.2 Head regulator

The head regulator is for controlling the water flow into the open channel at required rates. Fig. 2.6.2 shows the typical dimensions of a head regulator which composed of three main portions:

(1) Inlet transition. There is a panel that controls the flow of water at required rates.
(2) Control section. This may be in form of cylindrical or box conduit, or any shape of opening.
(3) Outlet transition. It is the terminal of the head regulator.

Fig. 2.6.2 Drawing of a typical head regulator (see p. 2-21 in the original)

The design flow rate is the maximum flow rate in the canal plus 10%.

1. The hydraulic design of control section in a head regulator is based on the following formulas:

$$V = C\sqrt{2g\Delta H} \qquad (2\text{-}3)$$

$$Q = AV \qquad (2\text{-}4)$$

where: V is the velocity of water in conduit (m/sec)

Q is the flow rate (m^3/sec)
A is the cross-section area of the conduit (m^2)
C is the discharge coefficient
ΔH is the head difference at upstream and downstream under downstream control (m)
= the difference of upstream water level and the downstream water level at center of opening under control (m)
(*Design of Small Structures*, USBR, 1978, p. 10)

If the inlet transition id of rectangular suppressed weir the formula applied in calculating the flow rate through weir is:

$$Q = CLH^{3/2} \qquad (2\text{-}5)$$

where: Q is the flow rate (m^3/sec)
C is the discharge coefficient
L is the length of conduit where water flows over/passes (m)
H is the depth of water over the crest (m)

2. Depth of submergence at inlet
   - Upstream: at least $(1+K_i)\ \Delta h_v + 0.08$ m
     (*Design of Small Structures*, USBR, 1978, p. 148)
   - Downstream: at least 0.05 m but not more than D/6 (where D is the height or the diameter of pipe) by adopting Ko = 0.7. If the height is more than this, adopt Ko = 1.

where: h_{VP} is the velocity head in pipe
h_{VC} is the velocity head in canal
$\Delta h_v = h_{VP} - h_{VC}$

3. The total head loss of the head regulator can be derived from the formula:

$$H_L = K_i\ \Delta h_v + S_f L_c + K_o\ \Delta h_v \qquad (2\text{-}6)$$

where: H_L is the total head loss (m)
K_i is the head loss coefficient of the inlet transition
S_f is the friction slope = $n^2v^2/ R^{4/3}$
n = 0.016
L_c is the length of pipe
K_o is the head loss coefficient of the outlet transition
Δh_v is the difference in velocity heads at pipe and canal (m)
(*Design of Small Structures*, USBR, 1978, p. 158)

2.6.3 Check structure

There are two types of check structure:

1. Open check structure; and
2. Check structure with pipe inlet.

To do the hydraulic design of the open che ck structure as shown in Fig. 2.6.3-1 the following parameters are recommended (*Design of Small Structures*, USBR, 1978, p. 136).

Fig. 2.6.3-1 Drawing of a typical open-check head regulator
(see p. 2-24 in the original)

1. The velocity of flow through the gate shall not exceed 1.50 m/sec;
2. Head loss through che ck = 0.5 Δh_v where Δh_v is the difference in velocity heads at the gate and canal upstream (at least 0.03 m);
3. To prevent flow over the wing wall during normal conveyance of water, the design flow through gate shall be 1.1 times of the maximum flow in the canal;
4. The allowable flow over the wing wall sh all be equal to 0.25 tim e of the design flow, derived from the hereunder formula:

$$Q = CLH^{3/2} \qquad (2\text{-}7)$$

where: Q is the flow rate over the wing wall
$= 0.25\ Q_{design}\ (m^3/sec)$
C is the discharge coefficient
L is the total length of the wall (m)
H is the depth of water over the wall (m)
(*Design of Small Structures*, USBR, 1978, p. 10)

5. Length of the structure (measured from the gate to the outlet transition) shall be longer than the length of jump at partial flow.

To calculate the design hydraulic feat ures of a check- and-pipe-inlet type, usually in case a road crossing the structure, apply the same principle of the inlet side to that of the open check type by taking the overflow wall instead of the wing wall as shown in Fig. 2.6.3-2. To calculate the flow through pipe, apply the calculation principle of orifice flow as described in siphon and road-crossing culvert sections.

Fig. 2.6.3-2 Drawing of a typical check-and-pipe-inlet head regulator
(see p. 2-25 in the original)

2.6.4 Drop structure

Since conveyance of water follows the topography of the irrigation area and the canal shall be lower and lower to its terminal. To cope with irregular topography the drop structure thus is applied by choosing the most appropriate type:

(1) Vertical drop structure. It is a structure that vertically lowers water level and canal bed as shown in Fig. 2.6.4-1. This type of drop structure is suitable with small canals with laminar free flow and the drop does not exceed 1.50 m. The design flow rate can be derived from the formula below:

$$Q = CLH^{3/2} \qquad (2\text{-}8)$$

where: Q is the flow rate
C is the discharge coefficient
L is the length of the structure where water flows through (m)
H is the depth of water over the crest (m)

Fig. 2.6.4-1 Drawing of a typical vertical drop structure (see p. 2-27 in the original)

A stilling pool shall be constructed to prevent erosion due to drop at the end portion of the structure. The length of the stilling pool can be calculated with the formula below:

$$L = [2.5 + 1.1\,(d_c/h) + 0.7\,(d_c/h)^3]\,\sqrt{dc/h} \qquad (2\text{-}9)$$

where: L is the length of stilling pool (m)
d_c is the critical depth (m)
h is the difference in height of bed levels upstream and downstream (m)
(*Canals and Related Structures*, Chap. 5 Canal Structure)

(2) Rectangular inclined drop structure. It is a drop structure for canal with drop more than 1.50 m but not exceeding 5.00 m and the length of the structure is not so long as shown in Fig. 2.6.4-2.

Fig. 2.6.4-2 Drawing of a typical inclined drop structure (see p. 2-28 in the original)

Calculation of stilling pool length shall comply with the following conditions:

(1) The slope of the inclined bed shall not exceed 1:2 (usually apply 1:2);
(2) The inclined bed and the stilling pool has equal widths;
(3) Length of the stilling pool can be calculated with the formula below:

$$B = \frac{18.476\sqrt{Q}}{Q+9.918} \qquad (2\text{-}10)$$

where: B is the width of the stilling pool (m)
Q is the design flow rate (m^3/sec)

(4) Length of the still pool shall be at least 4 times of the depth after jump;
(5) Set the stilling pool level low i.e. at point of hydraulic jum p the water level in the stilling pool shall not be higher than that of the canal.

2.6.5 Road-crossing culvert
In general, the road-crossing culvert is a straight pipe as shown in Fig. 2.6.5-1 and the flow is under very low pressure or no pressure (free flow). The culvert may be in cylindrical or box shape of one or more parallel rows.

Fig. 2.6.5-1 Drawing of a typical road-crossing culvert (see p. 2-30 in the original)

To design the culvert, the following features shall be complied (*Design of Small Canal Structures*, USBR, 1978, p.28):
1) Assuming it is a full flow and the transition to the canal shall be of broken-back type. In case water compression is required, the compressing structure shall be of check-and-pipe-inlet type.
2) Conveyance loss at the inlet transition: $K_i \Delta h_v$ (m)
3) Maximum slope of culvert bed at the inlet and outlet transitions: 1:6 and 1:4, respectively;
4) Submergence at the inlet transition: at least $1+ K_i \Delta h_v$, but not exceeding 0.08 m;
5) Maximum velocity in the culvert: not exceeding 1.5 m/sec;
6) Friction loss in culvert can be derived from the formula:

$$H_f = LS_f$$

where: L is the length of culvert (m)
S_f is the friction slope = $n^2v^2/ R^{4/3}$

7) Divergence loss at the outlet transition: $K_o \Delta h_v$ (m);
8) Manning coefficient of roughness, n = 0.016;
9) Freeboards:
- Freeboard at cutoff = freeboard of concrete-lined portion at cutoff
- Freeboard at headwall = 1.2 times of freeboard at cutoff
10) Outlet submergence: $\leq$ 1/6 of depth at outlet opening (m);
11) Diameter of culvert: at least 0.60 m;
12) Depth and thickness of cutoff shall comply with the following criteria:

Dept of water (m)	Cutoff wall depth (m)	cutoff wall thickness (m)
0 - 1.00	0.50	0.20
1.01 - 2.00	1.00	0.30
2.01 - 3.00	1.50	0.40
> 3.01	At least 2.00	0.50

2.6.6 Siphon

Siphon is a kind of culver t full flow under pressure. It s intermediate portion inverts underground and exposes to the ground surface as shown in Fig. 2.6.6-1. Thus it is so-called “the inverted siphon.” This type of structure will be constructed when a constraint is met, for examples, an obstruction of stream or drainage system or roa d, based on the following conditions:

(a) If the water in the canal is less than that of the ditch, then the canal shall go under the ditch and build a bridge over it;

(b) If the water in the ditch is less than th at of the canal, then the ditch shall go under the canal;

(c) The constraints of the siphon size, the velocity of water in the pipe , and the maintenance measure are:

- At least 0.60 m if the siphon length is less than 20 m;
- At least 0.80 m if the siphon length is over 20 m;
- The velocity of water in the siphon shall not be 1.50 to 3.00 m/sec; and
- If the siphon is more than 100 m long, a blow-off shall be done in order to dispose the sediments.

Fig. 2.6.6-1 Drawing of a typical siphon (see p. 2-32 in the original)

To design a siphon the followi ng hydraulic features shall be taken into consideration (*Design of Small Canal Structures*, USBR, 1978, p. 29):

(1) Assuming it is the full flow under pressure;

(2) Conveyance loss at the inlet transition:

$$h_i = K_i \, \Delta h_v \quad \text{(m)}$$

where Δh_v is the difference in velocity heads at pipe and canal (m);

(3) Conveyance loss at the check structure, if any:

thus, $$h_{ck} = K_{ch} \, \Delta h_v \text{ (m)}$$

where Δh_v is the difference in velocity he ads at the check opening and the upstream canal section (m);

(4) Conveyance loss at the gate:

$$h_g = K_g \, \Delta h_v \quad \text{(m)}$$

where Δh_v is the difference in velocity heads at the gate opening and the upstream canal section (m);

(5) Friction loss in pipe: $h_f = LS_f$ (m)

where: L is the length of pipe (m)
$S_f = n^2v^2/R^{4/3} = 0.016$
and V is the full velocity in pipe (m/sec)

(6) Divergence loss at the outlet transition: $h_o = K_o \Delta h_v$ (m)
where Δh_v is the difference in velocity heads at pipe and canal (m)
(7) For safety factor, add 10% to the sum of all losses
(8) Maximum velocity in pipe: 1.50 m/sec for short siphon and 3.0 m/sec for the long one
(9) Maximum slope at the inlet and outlet transitions: 1:4 and 1:6, respectively
(10) Maximum slope of pipe: 1:2
(11) Slope at the bottom of pipe: not less than 1:200
(12) Inlet submergence: $(1+K_i)\Delta h_v$, (recommended >0.08 m)
(13) Outlet submergence: ≤ 1/6 of depth of outlet opening (m)
(14) Freeboards shall comply with the criteria applied to culvert

2.6.7 Flume

In case the open channel has to pass into a vast low area or a deep wide natural flood way, or in case the water level in the open channel is relatively low, so it is not suitable in all cases to construct any water conveying structure there because the construction and maintenance cost would be high, as well as the strength of the structure. The best alternative to solve this problem is the elevated flume, as shown in Fig. 2.6.7-1. However, the flume has a disadvantage during flood. So its concrete bed elevation should be at least 0.50 m above the maximum water level in the low land or the flood way.

Fig. 2.6.7-1 Drawing of a typical elevated flume (see p. 2-35 in the original)

Calculation of hydraulic features of a flume shall comply with the following criteria:

1. Assuming the flow in the flume is subcritical flow
2. Flume width - depth of water in flume ratio (B/D): between 1 and 3
3. Slope of flume bed: 1:500 to 1:10,000
4. Manning coefficient of roughness, n = 0.016
5. Freeboard recommendations:

Flow rate (m^3/sec)	Minimum freeboard (m)
≤ 1.00	0.15
1.01 - 2.50	0.20
2.51 - 5.00	0.25

Remark: Apply the criteria as that of the culvert for freeboard at the cutoff.

6. Conveyance loss at the inlet transition: $h_i = K_i \Delta h_v$ (m)
7. Divergence loss at the outlet transition: $h_o = K_o \Delta h_v$ (m)
8. Friction loss in flume: $h_f = LS_f$ (m)

where: L is the length of flume (m)
S_f is the slope of flume bed = $n^2v^2/R^{4/3}$

9. Total head loss, $h_T = h_i + h_o = K_i\,\Delta h_v + K_o\,\Delta h_v$
   where Δh_v is the difference in velocity heads at canal and flume section (m)

2.6.8 Farm turnout

The farm turnout is a structure conveying adequate water to plantations as per the water demand of plants or preparation for cultivation. It is composed of cylindrical pipeline with a sluice gate at its front and a water measuring structure at its end, as shown in Fig. 2.6.8-1. Normally, the design of a farm turnout takes the calculation of a simple turnout in the same manner as that of the head regulator.

Fig. 2.6.8-1 Drawing of a typical farm turnout (see p. 2-37 in the original)

2.6.9 Tail regulator

The tail regulator, shown in Fig. 2.6.9-1, is the last check structure of the open channel. It irrigates upstream water to farms and drains water from the open channel in case of repair or maintenance. It comprises a sluice gate and cylindrical concrete pipeline that distributes water to the dike leading to a natural stream.
Normally, the design of a farm turnout takes the calculation of a simple turnout in the same manner as that of the head regulator.

Fig. 2.6.9-1 Drawing of a typical tail regulator (see p. 2-38 in the original)

Normally, the design of a tail regulator utilizes the same calculation of a road-crossing culvert with check and pipe inlet structure whereas the design flow is the flow in the open channel from the nearest upstream check structure to the outlet of tail pipe. The diameter of the outlet pipe shall be at least 0.60 m.

2.6.10 Drain culvert

The drain culvert is a pipeline applied to drain natural water underneath an open channel, as shown in Fig. 2.6.10-1 below.

Fig. 2.6.10-1 Drawing of a typical drain culvert (see p. 2-39 in the original)

The hydraulic design criteria are as follows:

1. Estimation of flood flow can be calc ulated with two formulas depending on the size of basin:

For a flood area not exceeding 20 km ^{2}, apply the rational formula with rain - fall intensity of 10-year frequency interval:

$$Q = 0.278\ CIA \qquad (2\text{-}11)$$

where: Q is the maximum flood flow (m^3/sec)
C is the runoff coefficient
I is the rainfall intensity (mm/cm)
A is the watershed area (km^2)
(*Applied Hydrology*, Ven Te Chow, David R. Maidment, Larry W. Mays, 1988, p. 497)

The runoff coefficient, C, depends on the topography and the rainfall intensity in respect of the past 10-year frequency interval:

Topography	C (pursuant to land use aspect)		
	Cultivation area	General pasture	Forest area
Relatively flat (slope 0-2%)	0.36	0.30	0.28
Moderate steep (slope 2-7%)	0.41	0.38	0.36
Steep (slope >7%)	0.44	0.42	0.41

(*Applied Hydrology*, Ven Te Chow, David R. Maidment, Larry W. Mays, 1988, p. 498)

To determine I, adopt the duration equals the time of concentration, T_c, which varies with the length of the dike, L, and the difference in ground elevations at the outlet and at the farthest point of the watershed area, H. Then, apply the formula mentioned in *the California Culverts Practice (1942)*:

$$T_c = (0.87L^3/H)^{0.385} \qquad (2\text{-}12)$$

where: T_c is the time of concentration (cm)
L is the length along the main stream from the outlet to the farthest point of the watershed area (km)
H is the difference in ground elevations at the outlet and at the farthest point of the watershed area (m)
(*Applied Hydrology*, Ven Te Chow, David R. Maidment, Larry W. Mays, 1988, p. 500)

2. The inlet inversion of the culvert shall be on top of or a bit underneath the original ground surface of the dike, or on top of or underneath the dike bed elevation.

3. The slope of the culvert shall be at least 1:200 and the maximum slope shall not be greater than the critical slope. If the steepness is higher than that, just design

the pipeline separately into two parts i.e. the slope of the first part at the inlet terminal is steeper than the critical slope for free flow into the culvert under inlet control while the slope of the second part near to the outlet is rather flat to slope 1:200.

4. The velocity of water in the culvert shall be within the following ranges:
   - $V \leq 3.00$ m/sec if it is equipped with a concrete outlet transition;
   - $V \leq 3.66$ m/sec if it is equipped with an energy dissipater at the outlet;
   - $V \leq 1.00$ m/sec if it is equipped with the inlet and outlet earth transitions
5. Apply the formula below to design the diameter of the culvert:

$$D = 1.13\sqrt{Q/V} \qquad (2\text{-}13)$$

where: D is the pipe diameter (m), at least 1.00 m
Q is the desired flood discharge (m^3/sec)
V is the velocity of the current (m/sec)
(*Design of Small Canal Structures*, USBR, 1978, p. 29)

6. Hydraulic control

a. Inlet control. When the submergence at front of the inlet transition occurs and it is in partial flow while the water level at the outlet transition is low and it does not affect the upstream water level. In addition, the freeboard slope at the inlet transition is less than the critical slope. Thus, apply the orifice equation to derive the water level at the inlet transition:

$$Q = CA\sqrt{2g\Delta H} \qquad (2\text{-}14)$$
$$V = Q/A \qquad (2\text{-}15)$$

where: Q is the flow rate (m^3/sec)
A is the cross-sectional area of the pipe (m^2)
C is the discharge coefficient
ΔH is the difference in upstream water level and diameter elevation of upstream pipe (m)
(*Design of Small Canal Structures*, USBR, 1978, p. 10)

b. Outlet control. If the downstream water level is high and it affects the upstream water level and the flow in pipe. Apply the formula below to determine the losses and what type of flow it is.

- Inlet loss: $h_i = Ki\ \Delta h_v$ (2-16)

Where Δh_v is the difference in velocity head at canal and pipe section (m)

- Bend loss: $h_b = \frac{ØVp^2}{2g}$ (2-17)

- Pipe loss: $h_p = h_f + h_b$ (2-18)

where h_f is the friction loss, derived from Manning formula:

$$h_f = SL$$

where S is the friction slope of pipe = $\dfrac{Vp^2n^2}{R^{4/3}}$

n = 0.016, for concrete pipe

L is the length of pipe (m)

V_p is the velocity of the current in pipe (m/sec)

- Outlet loss: $h_o = K_o\ \Delta h_v$

Remark: The values of K_i, K_o and Ø shown in Chapter 6.
(*Design of Small Canal Structures*, USBR, 1978, p. 210)

2.6.11 Bridge

There are lots of bridge under responsibility of the Phlai Chumpon Operation and Maintenance Project. Some of them are for pedestrians' passageways and some for vehicles'. The design therefore aims at development of the existing bridges and construction of new bridges pursuant to the purpose of use:

Roadway bridge. At any intersection of a canal and a public road (either village to village, provincial road or highway), it is impossible to construct any other structure to let the canal passes through. So it is necessary to construct a bridge there pursuant to the standards specified by the relevant agency or with consent of the road owner. However, the width of a standard roadway bridge shall be at least 6.00 m and it shall be able to support the moving loads of vehicles, size H20-S-16-44, as shown in Fig. 2.6.11-1.

Fig. 2.6.11-1 Drawing of a typical roadway bridge (see p. 2-43 in the original)

2.7 Design of a road for canal maintenance purpose

The type of road for canal maintenance purpose of the Phlai Chumpon Operation and Maintenance Project shall keep along any side of the canal:

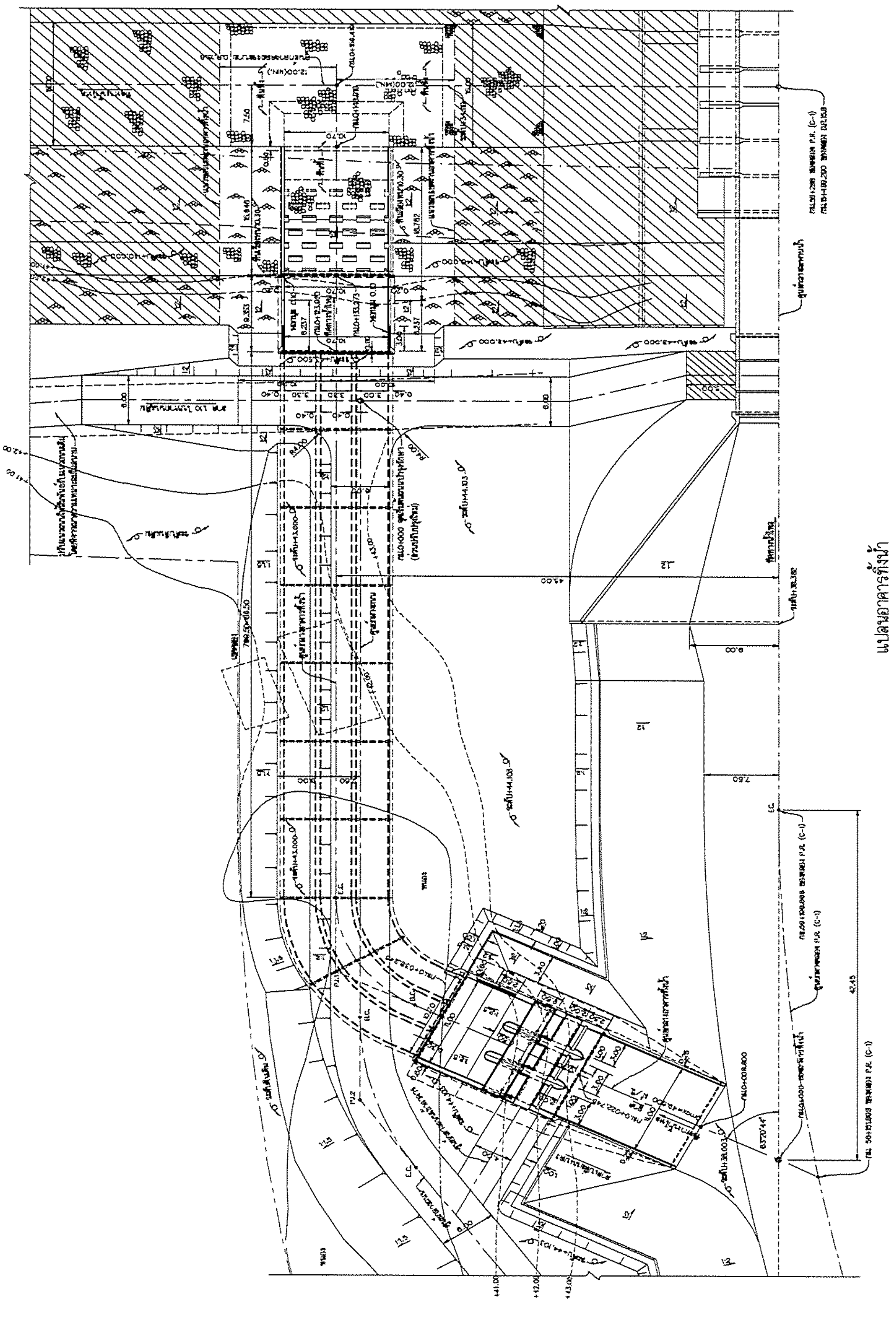
(1) Road on ridge of an open channel. In general, the road keeps along the right side of the canal and its pavement shall be of asphaltic concrete. Some parts of its may be omitted by conjunctively used with asphaltic-concrete-paved public roads which are still in good condition. The development shall be carried out only the road on the ridge is 6-9 m wide i.e. a two-traffic-lane road with 6-m asphaltic concrete pavement and 1.50-m-wide shoulders on both sides.

(2) Road on ridge of a lateral/sub-lateral canal. In general, it is a lateritic soil compacted road. Some parts of its may be omitted in case of conjunctive use with asphaltic-concrete-paved or reinforced concrete-paved public roads developed by local administration organizations. The development will be omitted if the existing road is still in good condition. Only the damaged portion of the existing road will be repaired by keeping the same width, about 3.50 m.

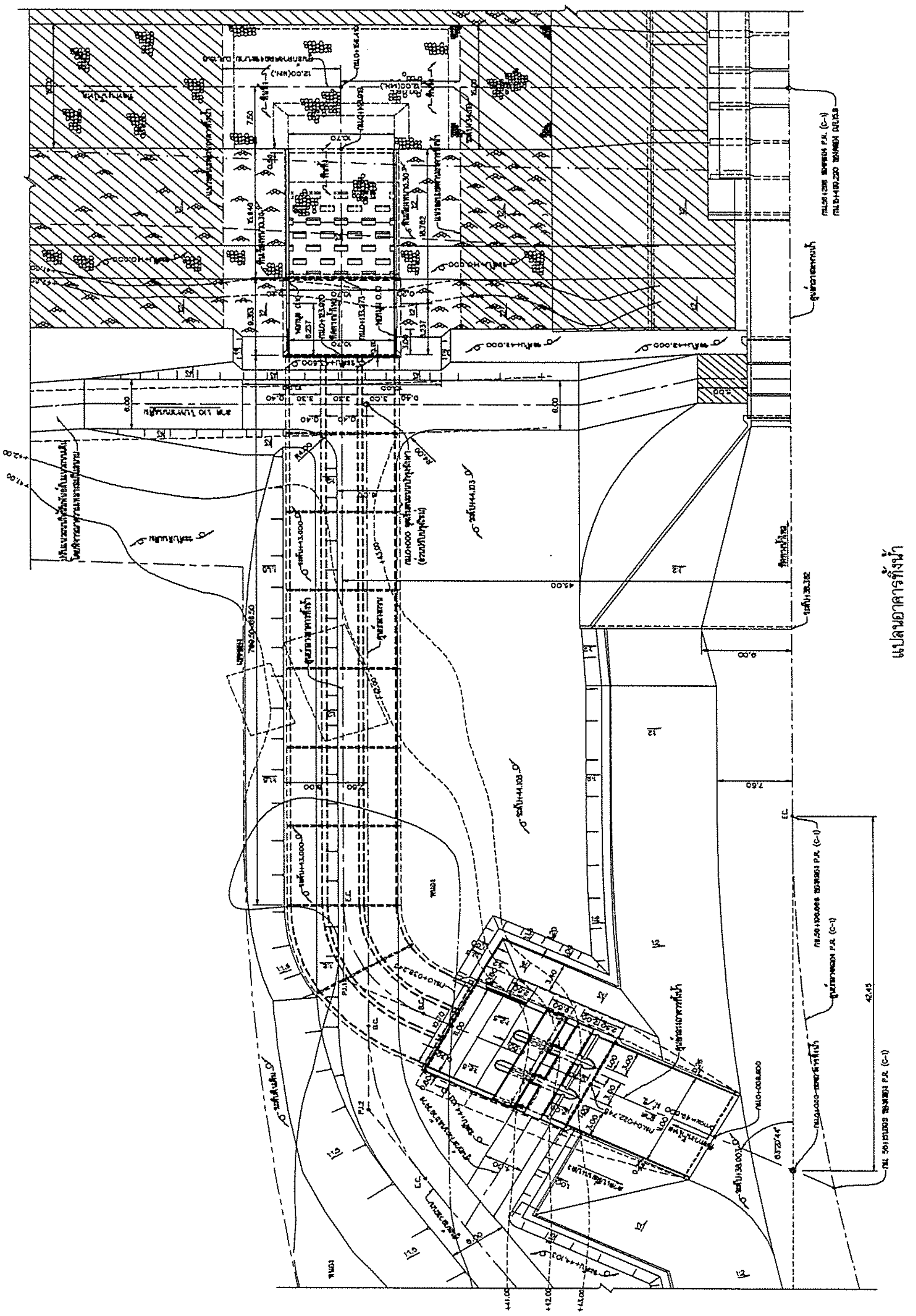
C.3.2.2 Structural Design of Outlet Drainage Structure at km 56+161.898 of C-1 Main Canal

2.1 อาคารทิ้งน้ำจากคลองส่งน้ำ
กม.56+151.898 ของคลองส่งน้ำสายใหญ่

อาคารทิ้งน้ำ กม.56+151.898 ของคลอง P.R. (C-1)



อาคารทิ้งน้ำ กม.56+151.898 ของคลอง P.R. (C-1)



รายการคำนวณด้านชลศาสตร์ INLET TRANSITION

Detail of Irrigation Canal P.R. (C-1) (Upstream)

รายละเอียดคลองส่งน้ำ P.R. (C-1) (ด้านเหนือน้ำ)

Canal Bed Width ความกว้างก้นคลอง, b_u	=	15.00	ม. m.
Water Depth in canal ความลึกของน้ำในคลอง, d_u	=	4.60	ม. m.
Height of Paved Canal ความสูงคลองดาด, H_{cu}	=	5.30	ม. m
Height to top of Dike ความสูงถึงระดับหลังคัน, H_{bu}	=	6.10	ม. m.
Width of Left Canal ความกว้างคลองฝั่งซ้าย, Bm_L	=	1.00	ม. m
Width of Right Canal ความกว้างคลองฝั่งขวา, Bm_R	=	1.00	ม. m.
Width of Canal dike ความกว้างคันคลอง, T_L	=	7.00	ม. m
Width of O&M Road ความกว้างถนนบำรุงรักษา, T_R	=	9.00	ม. m
Maximum water level ระดับน้ำสูงสุด, ร.น.ส.1	=	+42.603	m. (msl.) ม. (ร.ท.ก.)
Canal Bed level ระดับก้นคลอง, ระดับ "1"	=	+38.003	ม. (ร.ท.ก.)
Canal Berm level ระดับชานคลอง, ระดับ "2"	=	+43.303	ม. (ร.ท.ก.)
Canal top dike level ระดับหลังคันคลอง, ระดับ "3"	=	+44.103	ม. (ร.ท.ก.)
Natural Ground Level ระดับดินธรรมชาติ	=	+41.800	ม. (ร.ท.ก.)
Required drainage discharge ปริมาณน้ำที่ต้องการระบาย, Q_{req}	=	45.000	ม.^{3}/วินาที cms.

Detail of Drainage Canal D.R.15.8 (Downstream)

รายละเอียดคลองระบาย D.R.15.8 (ด้านท้ายน้ำ)

Canal Bed Width ความกว้างก้นคลอง, b_d	=	15.00	ม. m.
Water Depth in Canal ความลึกของน้ำในคลอง, d_d	=	3.80	ม.
Maximum Water Level ระดับน้ำสูงสุด, ร.น.ส.2	=	+37.919	m (msl) ม. (ร.ท.ก.)
Canal Bed Level ระดับก้นคลอง, ระดับ "4"	=	+34.119	ม. (ร.ท.ก.)

บริษัท ปัญญาคอนซัลแตนท์ จำกัด บริษัท รีซอสส์ เอนจิเนียริ่ง คอนซัลแตนทส์ จำกัด บริษัท ฟรอนเทียร์ เอ็นจิเนียริ่ง คอนซัลแทนท์ส จำกัด	รายการคำนวณ : อาคารทิ้งน้ำ กม.56+151.898 ของคลอง P.R. (C-1)	วันที่ : มิถุนายน 2554 โดย : นายนิวัฒน์ กุลกาญจนาธร สย.1543

หาขนาดบานระบาย Drainage Gate Size Calculation

Design Discharge ปริมาณน้ำ, Q_{des}		$= 1.1 \times 45.00$	=	49.500	$ม.^3/วินาที$ cms
จาก From	Q	$= 6.264 x C_d x N x B_g x (C_g\text{-}dh) dh^{0.5}$			
Flow Coefficient ส.ป.ส. การไหล, C_d			=	0.60	
Size selection เลือกขนาดช่องเปิด		= 3 - 3.00 x 3.60 ม. m			
Try	dh		=	0.183	ม. m.
จะได้ then	Q		=	49.50	$ม.^3/วินาที$ cms
			=	Q_{des}	OK.
	V	= 6.264x0.60x0.183^0.5	=	1.610	ม./วินาที cms.
Gate Freeboard ระยะเผื่อล้นของบานระบาย, f_g		= 0.10+0.05x3.60	=	0.28	ม. m
Gate Height ความสูงบานระบาย, H_g			=	3.88	ม. m
		ใช้ use	=	4.00	ม. m
Abutment Freeboard ระยะเผื่อล้นของตอม่อ, f_s		= 0.20+0.15x3.60	=	0.74	ม. m
Abutment Height ความสูงตอม่อ		= 3.60+0.74	=	4.34	ม. m
		ใช้ use	=	4.95	ม. m
Thickness of central Abutment ความหนาตอม่อกลาง			=	1.00	ม. m
Width of Building Front ความกว้างอาคารหน้าท่อ, B_s			=	11.00	ม. m

Critical Depth Calculation
หาความลึกวิกฤติ (d_c)

จาก From	d_c		=	$(q^2/g)^{1/3}$	
	q	$= Q_{des}/B_s$	=	4.50	$ม.^3/วิ/ม.$ cms.
จะได้ Then	d_c		=	1.270	ม. m
	n		=	0.010	
	A_c		=	13.970	$ม.^2$ m^2
	P_c		=	13.540	ม. m
	R_c	$= A_c/P_c$	=	1.032	ม. m
	V_c	$= Q_{des}/A_c$	=	3.543	ม./วินาที m/s
	S_c	$= (V_c x n/R_c^{2/3})^2$	=	0.00120	

Size of Pipe Calculation
การหาขนาดท่อ

Pipe flow is designed as free flow in order to release water easily to drainage downstream Canal
เพื่อให้น้ำสามารถระบายไปยังคลองระบายท้ายน้ำได้อย่างสะดวกจึงกำหนดให้การไหลในท่อเป็นแบบ free flow
The freeboard for flowing garbages is included to avoid the flow obstruction.
และมีระยะเผื่อล้นไว้กรณีมีขยะลอยมาจะได้ไม่เกิดการกีดขวางทางน้ำไหล

Pipe slope from building layout จากการวางผังอาคารได้ลาดท่อ, Sb		= 1:1,000	<	S_c	OK.
Use Box pipe for กำหนดให้ใช้ท่อเหลี่ยม จำนวน			=	3	แถว row
Considering for 1 row คิดท่อเพียง 1 แถว		= $Q_{des}/3$	=	16.500	ม.^{3}/วินาที cms.
Assume pipe width สมมติความกว้างท่อ	B_B	= 3.30 ม. m			
Manning Equation สมการของ Manning	Q	= $V_B x A_A$			
	V_B	= $^1/_n x R_B x (1/S_b)^{0.5}$	Equation no. 1 สมการที่......1		
	d_B		=	2.280	ม. m.
	A_B		=	7.524	ม.^{2} m^2
	P_B		=	7.860	ม. m
	R_B	= A_B/P_B	=	0.957	
	n		=	0.014	
Continuity Equation	V_B		=	2.194	ม./วินาที m/s
สมการความต่อเนื่อง	Q	= $V_B x A_B$ ➡ $V_B=Q/A_B$	Equation no. 2 สมการที่....2		
จะได้ Then	V_B		=	2.193	ม./วินาที m/s
	Q		=	16.50	ม.^{3}/วินาที cms
					OK.

Depth Calculation before and after Hydraulic Jump
การหาความลึกก่อนและหลังเกิด Hydraulic Jump

หน้าตัดที่ 1 Section no.1	E_C	= $d_c+V_c^2/2g+Z$			
ระดับพื้นหน้าท่อ Floor level at Pipe front bed			=	+38.003	ม. (ร.ท.ก.) m.(msl.)
Building floor level at Gate ระดับพื้นอาคารบริเวณบานระบาย			=	+39.003	ม. (ร.ท.ก.) m.(msl.)
	Z	= 38.003 - 39.003	=	1.00	ม. m
ดังนั้น Then	E_C	= 1.270+3.543^2/2g+1.00			
		= 2.910 ม. m.			
Section no.2 หน้าตัดที่ 2	E_1	= $d_1+V_1^2/2g+hf_1$			
	L_1	= 2.50xZ	=	2.50	ม. m.
Try	d_1		=	0.684	ม. m.
	n		=	0.010	

A_1	$= B_S x d_1$	=	7.525	ม.^{2} m²
P_1	$= B_S+(2xd_1)$	=	12.368	ม. m
R_1	$= A_1 / P_1$	=	0.608	ม. m
V_1	$= Q_{des}/A_1$	=	6.578	ม./วินาที m/s
S_1	$= (V_1 xn/R_1^{2/3})^2$	=	0.00839	
hf_1	$= S_1 x L_1$	=	0.021	ม. m
จะได้ Then E_1	= 2.911	=	E_c	OK.
Fr_1	$= V_1/(gd_1)^{0.5}$	=	2.54	

Depth after Hydraulic Jump
ความลึกหลังเกิด Hydraulic Jump, $d_2 = d_1/2x((1+8Fr^2)^{1/2}-1)$ = 2.14

< d_B OK.

Pipe freeboard Calculation
หาระยะเผื่อล้นของท่อ

Consider freeboard of Jump
พิจารณาช่วงเกิด Jump ระยะเผื่อล้น, f = $0.10x(V_1+d_2)$ = 0.87 ม. m

Pipe Height ความสูงท่อ, H_B = d_B+f_b = 3.15 ม. m

ใช้ use = 3.60 ม. m

Therefore, box pipe size is
ดังนั้น เลือกใช้ท่อเหลี่ยมที่ได้จากการออกแบบ = 3 - 3.30x3.60 ม. m

Calculation on Flood Wall Length
หาความยาวกำแพงน้ำล้น Ls

Exess water can overflow to downstream of gate without spilling over canal
กำหนดให้ปริมาณน้ำส่วนเกินสามารถไหลข้ามไปด้านท้ายบานระบายได้โดยไม่ทำให้น้ำล้นคลอง

จาก From Q_s	$= 1.84xL_s xH_s^{1.5}$			
H_s	$= H_{cu} - d_u - 0.20$	=	0.50	ม. m
L_s		=	6.50	ม. m
จะได้ Then Q_s		=	4.228	ม.^{3}/วินาที cms.

พิจารณาระดับน้ำที่ปลายท่อ Water level at end of pipe

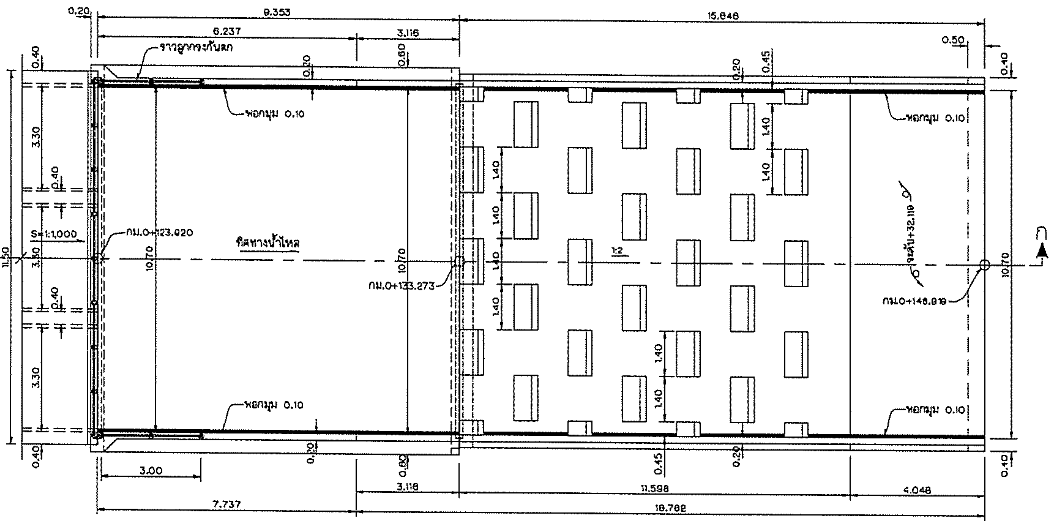
Bed pipe level at outlet point
ระดับพื้นท่อที่จุดออก = +37.918 ม. (ร.ท.ก.) m (msl.)

Water level at outlet point
ระดับน้ำที่จุดออก = +40.198 ม. (ร.ท.ก.) m.(msl.)

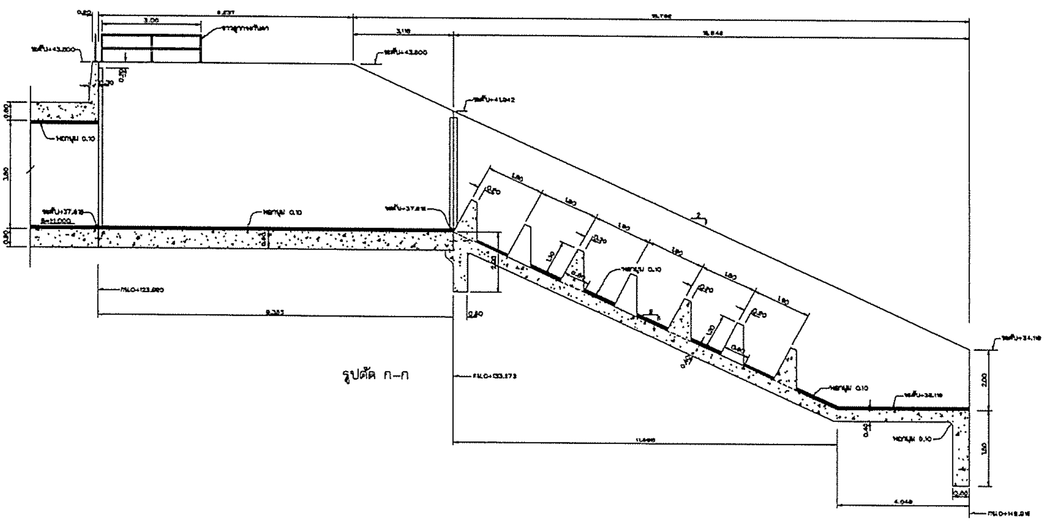
> ร.น.ส.2 +37.919 OK.

โครงการปรับปรุงโครงการส่งน้ำและบำรุงรักษาพลายชุมพล จ.พิษณุโลก		
บริษัท ปัญญาคอนซัลแตนท์ จำกัด บริษัท รีซอสส์ เอนจิเนียริ่ง คอนซัลแตนทส์ จำกัด FEC บริษัท ฟรอนเทียร์ เอ็นจิเนียริ่ง คอนซัลแทนท์ส จำกัด	รายการคำนวณ : อาคารทิ้งน้ำ กม.56+151.898 ของคลอง P.R. (C-1)	วันที่ : มิถุนายน 2554 โดย : นายนิวัฒน์ กุลกาญจนาธร สย.1543

OUTLET TRANSITION



แปลน



รูปตัด ก-ก

รายการคำนวณด้านชลศาสตร์ OUTLET TRANSITION

Level and dimensions calculation for Outlet Transition

การหาระดับและมิติต่าง ๆ ด้าน Outlet Transition

Set slope of flume bed and side wall equal

กำหนด ความลาดของพื้นรางและกำแพงข้าง เท่ากับ 1 : 2

Building width can be calculated from

- หาความกว้างของอาคาร (B_b) โดยประมาณจาก

B_b = Q / q

Design Discharge

เมื่อปริมาณน้ำออกแบบ Q = 49.500 ม.^{3}/ วินาที *cms.*

Table shown relation between Discharge (Q) and allowable discharge per unit width (q)

ตารางแสดงความสัมพันธ์ระหว่างปริมาณน้ำ (Q) กับ ปริมาณน้ำที่ยินยอมให้ใช้ต่อหน่วยความกว้าง (q)

ปริมาณน้ำ Q *Discharge* ม.^{3}/ วินาที *(cms)*			ปริมาณน้ำต่อหน่วยความกว้าง (q) *Discharge per unit width (q)* ม.^{3}/ วินาที *(cms)*		
0	-	1.10	0.45	-	0.90
1.10	-	2.80	0.90	-	1.40
2.80	-	5.30	1.40	-	1.85
5.30	-	13.00	1.85	-	2.80
13.00	-	28.00	2.80	-	4.65
28.00	-	> ขึ้นไป	4.65	-	5.60

From Q, q is

จากปริมาณน้ำ Q จะได้ q = 4.85 ม.^{3}/ วินาที / ม. *cms/m.*

Outlet Transition width (B)

จะได้ความกว้าง Outlet Transition (B) = 10.21 ม. *m*

Width of pipe will be

หาความกว้าง (B) จากขนาดท่อ จะได้ = 10.70 ม. *m*

ใช้ *use* B = 10.70 ม. *m*

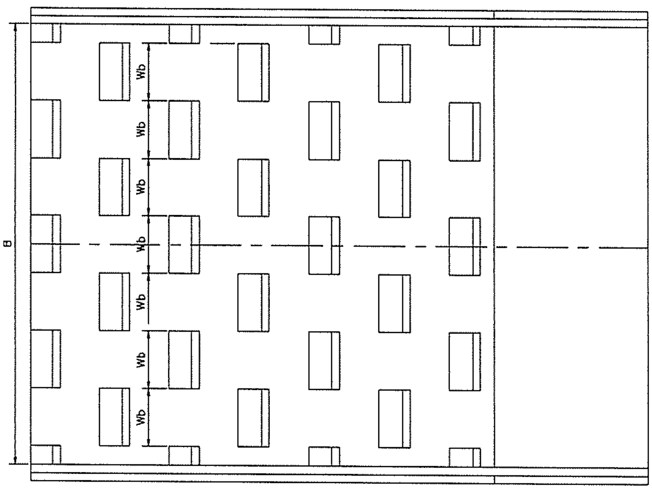
กำหนด ให้ *Set the following figures*

ปริมาณน้ำออกแบบ (Q) *designed discharge*	=	49.50	ม.^{3}/ วินาที *cms.*
ความลึกน้ำในท่อ (d) *Water Depth in pipe*	=	2.28	ม. *m*
ความสูงท่อ (H) *Pipe Heigth*	=	3.60	ม. *m*
ความกว้างท่อ (W) *Pipe width*	=	3.30	ม. *m.*
จำนวนท่อ *number of pipe*	=	3	แถว *row*

Flow area พื้นที่การไหล (A)	=	22.57	ม.^{2} m^2
Hydraulic radius รัศมีชลศาสตร์ (P)	=	23.58	ม. m
n	=	0.014	
slope	1 :	1,000	
R	=	0.957	ม. m
Velocity ความเร็ว (V)	=	2.194	ม. / วินาที m/s
h_v	=	0.245	ม. m

Calculation for size of Baffle with the below row arrangement

- หาขนาดของแท่ง Baffle โดยทำการจัดวางแถวต่าง ๆ ของแท่ง Baffle ดังนี้



Baffle size can be calculated with the relation with d_c

หาขนาดแท่ง Baffle โดยอาศัยความสัมพันธ์กับค่า d_c

d_c	=	$(q^2/g)^{1/3}$	
d_c	=	1.34	ม. m.
Baffle Heigth ความสูงของแท่ง Baffle , h_b	=	$0.90\ d_c$	
use	=	1.20	ม. m.
ใช้ h_b	=	1.20	ม. m.

Baffle width and spacing
ความกว้างของแท่ง Baffle และระยะห่าง W_b

	Min W_b	=	h_b	=	1.20	ม. m
	Max W_b	=	$1.5\ h_b$	=	1.80	ม. m
use ใช้	W_b			=	1.40	ม. m

For width
สำหรับ Partial Block ความกว้าง (W_p)

	Min W_p	=	$1/3\ h_b$	=	0.40	ม. m
	Max W_p	=	$2/3\ h_b$	=	0.80	ม. m
or หรือ	W_p	=	$W_b/2$	=	0.70	ม. m
use ใช้	W_p			=	0.45	ม. m

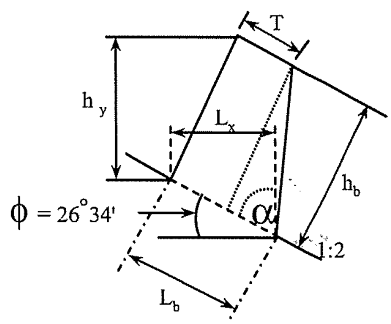
Block Thickness
ใช้ความหนา Block , T = 0.20 ม. m

T shall be 0.20 m – 0.25 m
(T จะต้องอยู่ระหว่าง 0.20 ม. ถึง 0.25 ม.)

Set Length of for straight portion
กำหนด ความยาว Outlet Transition (L_{TD}) ช่วงกำแพงตรง = 6.237 ม. m

Length of from declined wall to Block row no. 1
ความยาว Transition (L1) ช่วงกำแพงเอียงก่อนถึง Block แถวที่ 1 = 3.116 ม. m

Calculation for Block Dimension
- หาขนาดต่าง ๆ ของ Block



Set
กำหนด มุม ϕ = 26 องศา °

34 ลิปดา '

00 ฟิลิปดา "

sin ϕ = 0.447

cos ϕ = 0.894

Length calculation

หาความยาว L_b : มุม $\alpha = 90^\circ - \phi$ = 63.433 องศา

L_b = $T + h_b / \tan\alpha$ = 0.80 ม. *m*

use ใช้ L_b = 0.80 ม. *m*

Length ความยาว L_x = $L_b \cos\phi$ = 0.72 ม. *m*

Heigth ความสูง h_y = $h_b \cos\phi$ = 1.07 ม. *m*

Spacing of Baffle row on slope (S) use bed slope = 2

- หาระยะตามลาด S ระหว่างแถวของแท่ง Baffle (ใช้ค่าความลาดชันพื้นเท่ากับ 2)

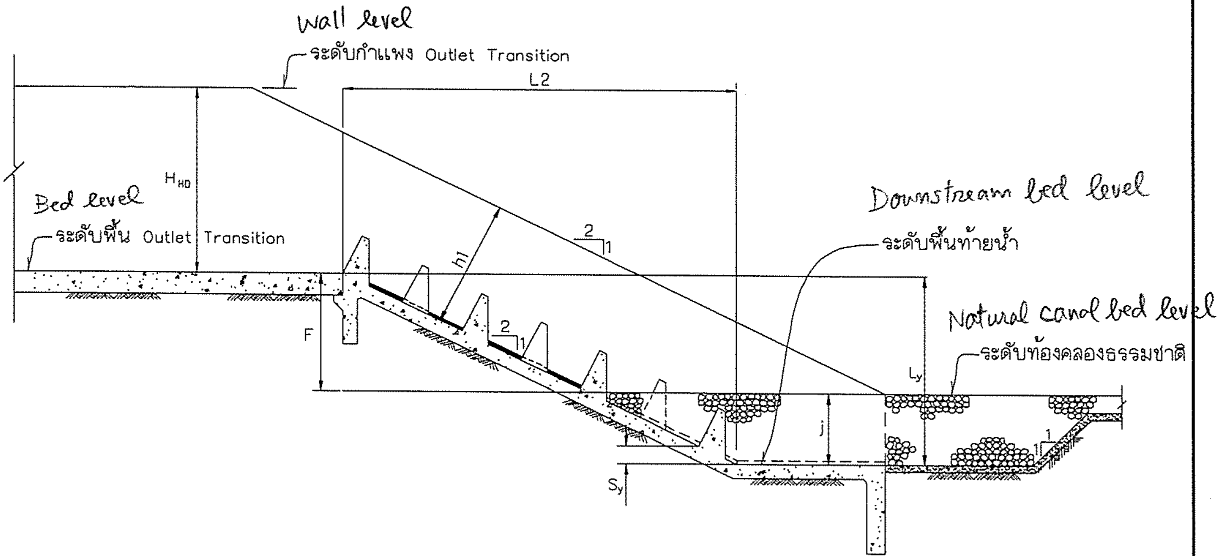
S = $z \times h_b$ = 2 x 1.20 (แต่ไม่เกิน 1.80 ม.) *(not over 1.80 m.)*

= 2.40 ม. *m*

ใช้ *Use* S = 1.80 ม. *m*

Calculation for minimum soil depth (j) at outlet to ensure the last Baffle row will be under the ground after filling soil reach to bed level at downstream

- หาความลึกของดินน้อยที่สุด (j) ทางด้าน Outlet เพื่อให้แน่ใจว่าแท่ง Baffle แถวสุดท้ายจะต้องจมปิดอยู่ในดิน เมื่อทำการกลบดินถึงระดับก้นทางน้ำ



use	S_y	=	$S \sin\phi$	= 0.805	ม. *m*
ใช้	S_y			= 0.969	ม. *m*
	h_y	=	$h_b \cos\phi$	= 1.073	ม. *m*
use	j	=	$S_y + h_y$	= 2.042	ม. *m*
ใช้	j			= 2.000	ม. *m*

Length Calculation

- หาความยาว Apron (L_2)

Natural Canal Bed level ระดับท้องคลองธรรมชาติ		=	+34.119	m.(msl.) ม. (ร.ท.ก.)
Pipe bed level ระดับท้องท่อ		=	+37.918	m.(msl.) ม. (ร.ท.ก.)
Bed level ระดับพื้น Outlet Transition = ระดับท้องท่อ	Pipe Bed level	=	+37.918	m.(msl.) ม. (ร.ท.ก.)
จะได้ Heigth ความสูง F		=	Bed level / Natural canal bed level ระดับพื้น Outlet - ระดับดินท้องคลองธรรมชาติ	
		=	3.799	ม. m
Minimum Heigth ความสูงต่ำสุด , L_y =	$F + j$	=	3.799 + 2.000	
		=	5.799	ม. m
Minimum Baffle row จำนวนแถว Baffle ต่ำสุด =	L_y / S_y	=	5.799 / 0.969	
use number of Baffle row		=	5.98	แถว row
ใช้ จำนวนแถว Baffle		=	7	แถว row

If Ly/Sy ratio is less, the number of row shall be at least 4 and filled by soil at least 1 row or more if necessary at downstream

หากอัตราส่วน L_y / S_y มีค่าน้อย ๆ จะต้องจัดให้มีจำนวนแถวอย่างน้อย 4 แถว โดยทำให้จมลงไปในดินด้านท้ายน้ำ 1 แถว หรือมากกว่านั้นหากจำเป็น

ความยาวพื้นเอียง , L_S =	number of row จำนวนแถว Baffle x S	=	7x1.80	
Length of bed slope		=	12.60	ม. m
ใช้ use L_S		=	12.99	ม. m
L_2 =	$L_S \cos \phi$	=	11.62	ม. m
Length of Outlet at Baffle downstream ความยาว Outlet ช่วงท้าย Baffle		=	4.048	ม. m
ระดับพื้น Outlet ช่วงท้าย Baffle		=	Bed level ระดับพื้น Outlet Transition - L_y	
Bed level of Outlet at Baffle downstream		=	+32.119	m.(msl.) ม. (ร.ท.ก.)

Calculation for wall height at each section

- หาความสูงกำแพงส่วนต่าง ๆ

Wall crest level at Pipe downstream ระดับสันกำแพงท้ายท่อ	=	+43.500	m.(msl.) ม. (ร.ท.ก.)
Wall Heigth ความสูงกำแพง (H_{HD})	=	Wall crest level at pipe downstream - Outlet bed level ระดับสันกำแพงท้ายท่อ - ระดับพื้น outlet	
	=	5.58	ม. m.

Chute Wall Heigth

ความสูงของกำแพง Chute (h_l)

	h_l =	$3\,h_b$	=	3 x 1.20	
			=	3.60	m ม.
Use ใช้	Bed thickness ความหนาพื้น Outlet Transiton		=	0.60	m ม.
	Bed thickness at slope portion ความหนาพื้น ช่วงพื้นเอียง		=	0.40	m ม.
	Cut off depth at end of flume ความลึก Cut off ที่ปลายรางเท		=	1.50	m ม.
	Cut off thickness ความหนา Cut off		=	0.50	m ม.

โครงการปรับปรุงโครงการส่งน้ำและบำรุงรักษาพลายชุมพล จ.พิษณุโลก		
บริษัท ปัญญาคอนซัลแตนท์ จำกัด บริษัท รีซอสส์ เอนจิเนียริ่ง คอนซัลแตนทส์ จำกัด บริษัท ฟรอนเทียร์ เอ็นจิเนียริ่ง คอนซัลแทนท์ส จำกัด	รายการคำนวณ : อาคารทิ้งน้ำ กม.56+151.898 ของคลอง P.R. (C-1)	วันที่ : มิถุนายน 2554 โดย : นายวิเชียร ทวินันท์ สย.6124

Structural Design Criteria

เกณฑ์กำหนดการออกแบบด้านโครงสร้าง

Ultimate Strength of concrete กำลังอัดประลัยของคอนกรีต , fc'	=			210	*kg./cm²* กก./ซม.^{2}
Allowable strength of Reinforcement Bar หน่วยแรงที่ยอมให้ในเหล็กเสริม , fs	=			1,500	*kg/cm² (deformed bar)* กก./ซม.^{2} (เหล็กข้ออ้อย)
	=			1,200	*kg/cm² (round bar)* กก./ซม.^{2} (เหล็กกลม)
Compressive stress in Concrete หน่วยแรงอัดในคอนกรีต , fc	=	0.45 fc'			
Elastic modulus of Concrete	=			94.50	*kg/cm²* กก./ซม.^{2}
โมดูลัสแห่งความยืดหยุ่นของคอนกรีต ,E_c	=	$w^{1.5}$ 4,270 $fc'^{1/2}$		(w = 2.33 ตัน/ม.^{3})	*ton/m³*
	=			220,075	*kg/cm²* กก./ซม.^{2}
Elastic modulus of Concrete โมดูลัสแห่งความยืดหยุ่นของคอนกรีต ,E_s	=			2,040,000	*kg/cm²* กก./ซม.^{2}
Modulus ratio อัตราส่วนโมดูลัส , n	=	Es / Ec	=	10	

Reinforce Concrete Properties in case of round bar

- คุณสมบัติของคอนกรีตเสริมเหล็กกรณีเหล็กกลม

k	=	1/[1+fs/(nfc)]	=	0.441	
j	=	1-k/3	=	0.853	
R	=	0.5 fc j k	=	17.759	*kg/cm²* กก./ซม.^{2}

Reinforce Concrete Properties in case of deformed Bar

- คุณสมบัติของคอนกรีตเสริมเหล็กกรณีเหล็กข้ออ้อย

k	=	1/[1+fs/(nfc)]	=	0.387	
j	=	1-k/3	=	0.871	
R	=	0.5 fc j k	=	15.909	*kg/cm²* กก./ซม.^{2}
Allowable shear stress of Concrete หน่วยแรงเฉือนที่ยอมให้ของคอนกรีต , v_c	=	0.29 $fc'^{1/2}$	=	4.20	" กก./ซม.^{2}
Allowable shear strength of Steel หน่วยแรงเฉือนที่รับได้โดยเหล็กเสริม , v	=	1.32 $fc'^{1/2}$	=	19.13	" กก./ซม.^{2}
Bonding Stress of deformed bar หน่วยแรงยึดหน่วงสำหรับเหล็กข้ออ้อย , u	=	2.29 $fc'^{1/2}$ /D	<=	25	" กก./ซม.^{2}
Bonding Stress of Round bar หน่วยแรงยึดหน่วงสำหรับเหล็กกลม , u	=	1.145 $fc'^{1/2}$ /D	<=	11	" กก./ซม.^{2}
Unit Weight of Reinforced Concrete หน่วยน้ำหนักของคอนกรีตเสริมเหล็ก	=			2,400	กก./ม.^{3} *kg/m³*
Unit Weight of water หน่วยน้ำหนักของน้ำ , γ_w	=			1,000	กก./ม.^{3} "
Unit Weight of dry soil หน่วยน้ำหนักของดินแห้ง , γ_{dry}	=			1,600	กก./ม.^{3} "
Unit Weight of compacted soil หน่วยน้ำหนักของดินถมบดอัดแน่น , γ_d	=			1,900	กก./ม.^{3} "
Unit Weight of saturated soil หน่วยน้ำหนักของดินอิ่มตัวด้วยน้ำ , γ_{sat}	=			2,150	กก./ม.^{3} "
Soil Angle มุมทรงตัวของดิน , ϕ	=			30	องศา °
K_a	=	$(1-\sin\phi)/(1+\sin\phi)$	=	0.333	

โครงการปรับปรุงโครงการส่งน้ำและบำรุงรักษาพลายชุมพล จ.พิษณุโลก		
บริษัท ปัญญาคอนซัลแตนท์ จำกัด บริษัท รีซอสส์ เอนจิเนียริ่ง คอนซัลแตนทส์ จำกัด บริษัท ฟรอนเทียร์ เอ็นจิเนียริ่ง คอนซัลแทนท์ส จำกัด	รายการคำนวณ : อาคารทิ้งน้ำ กม.56+151.898 ของคลอง P.R. (C-1)	วันที่ : มิถุนายน 2554 โดย : นายวิเชียร ทวินันท์ สย.6124

Structural Calculation

รายการคำนวณด้านโครงสร้าง INLET TRANSITION

Top width of Wall
ความหนาที่ขอบบนกำแพง (t_1) = 0.25 ม. *m*

Bottom width of Wall
ความหนาที่โคนกำแพง (t_2) = 0.80 ม. *m*

Slab thickness
ความหนาพื้น (t_3) = 0.80 ม. *m*

Building Width
ความกว้างของอาคาร = 11.00 ม. *m*

Height of floor to top
ความสูงของอาคารถึงพื้น (H) = 5.10 ม. *m*

Height of building to base slab
ความสูงของอาคารถึงระดับฐานของพื้น (H_T) = 5.10 + 0.80 = 5.90 ม. *m*

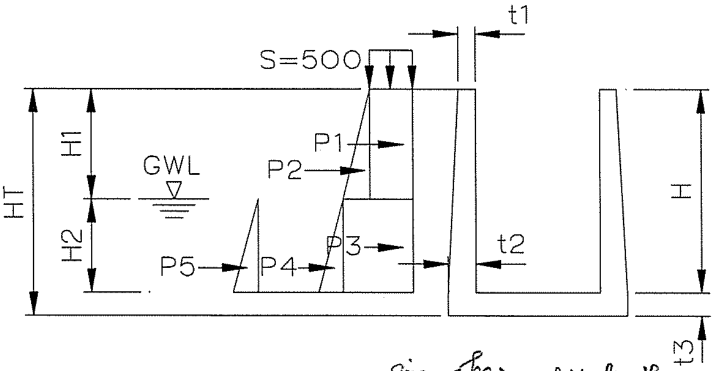
Height of Groundwater Level
ความสูงของระดับน้ำใต้ดิน = HT/2 = 5.90/2 = 2.950 ม. *m*

Height of soil over groundwater level
ความสูงของดินเหนือระดับน้ำใต้ดิน (H_1) = 2.950 ม. *m*

Height of soil below groundwater level
ความสูงของดินที่ต่ำกว่าระดับน้ำใต้ดิน (H_2) = 2.150 ม. *m*

Soil Angle
มุมทรงตัวของดิน (θ) = 30 องศา °

$K_a = (1-\sin\theta)/(1+\sin\theta) = 0.333$



Soil Pressure แรงดันดิน		*Size (kg)* ขนาด(กก.)	*Arm length (m)* ระยะแขน(ม.)	*Moment (kg-m)* โมเมนต์(กก.-ม.)
P_1	= 0.333x500x2.950	491	3.625	1,781
P_2	= 1/2x0.333x1,900x2.950^2	2,753	3.133	8,626
P_3	= 0.333x(500+1,900x2.950)x2.150	4,371	1.075	4,699
P_4	= 1/2x0.333x(2,150-1,000)x2.150^2	885	0.717	634
P_5	= 1/2x1,000x2.150^2	2,311	0.717	1,656
	รวม *Total*	10,811		17,396

$d_M = (M/(Rxb))^{0.5}$ = 33.07 ซม. *cm*

$d_V = V/(v_c xb)$ = 25.73 ซม. *cm*

ใช้ *use* d = 75 ซม. *cm*

A_s = M/($f_s$xjxd) = 17.75 ซม.^{2} *cm²*

ใช้ *use* 16@0.20 A_s = 10.05 ซม.^{2} *cm²* Σo = 25.13 ซม. *cm*

\+ 20@0.20 A_s = 15.71 ซม.^{2} *cm²* Σo = 31.42 ซม. *cm*

รวม *Total* A_s = 25.76 ซม.^{2} *cm²* Σo = 56.55 ซม. *cm*

เหล็กกันร้าว *Temperature Steel* A_s = 0.002x100xt_2/2 = 8.00 ซม.^{2} *cm²*

ใช้ *use* 16@0.20 A_s = 10.05 ซม.^{2} *cm²*

ตรวจสอบหน่วยแรงยึดหน่วง *Bonding Stress check*

u = V/($\Sigma_o$jd) = 2.99 กก./ซม.^{2} *kg/cm²*

u_a = 3.23x$f_c'^{0.5}$/ϕ = 23.40 กก./ซม.^{2} *kg/cm²*

ใช้ *use* u_a = 23.40 กก./ซม.^{2} *kg/cm²* > u O.K.

การลดปริมาณเหล็กเสริมที่ตัวกำแพง *Reinforcement Steel decreasing at Wall*

พิจารณาที่ระยะจากขอบบนของกำแพง *Consider from top of Wall* = 2.80 ม. *m*

ความหนาของกำแพงที่ระยะลดเหล็ก *Wall thickness at steel decreasing point* = 0.55 ม. *m*

ความสูงของดินเหนือระดับน้ำใต้ดิน (H_1) *Height of soil over groundwater level* = 2.800 ม. *m*

ความสูงของดินที่ต่ำกว่าระดับน้ำใต้ดิน (H_2) *Height of soil lower groundwater level* = 0.000 ม. *m*

แรงดันดิน *Soil Pressure*

		ขนาด(กก.) *Size (kg)*	ระยะแขน(ม.) *Arm length (m.)*	โมเมนต์(กก.-ม.) *moment (kg-m)*
P_1	= 0.333x500x2.800	466	1.400	653
P_2	= 1/2x0.333x1,900x2.800^2	2,480	0.933	2,315
P_3	= 0.333x(500+1,900x2.800)x0.000	0	0.000	0
P_4	= 1/2x0.333x(2,150-1,000)x0.000^2	0	0.000	0
P_5	= 1/2x1,000x0.000^2	0	0.000	0
	รวม *Total*	2,946		2,968

ใช้ *use* d = 50.20 ซม. *cm*

A_s = M/($f_s$xjxd) = 4.52 ซม.^{2} *cm²*

ใช้ *use* 16@0.20 A_s = 10.05 ซม.^{2} Σo = 25.13 ซม. *cm*

ตรวจสอบแรงเฉือน *Check shear stress*

V = 2,946 กก. *kg.*

0.5xV_c = 0.5x0.29x210^0.5x100x50.20

= 10,547 กก. *kg.* > V O.K.

ระยะยื่นเหล็กที่จุดลดเหล็ก *Length of bar at steel decreasing point* = ค่ามาก *high value*(12ϕ , d) = 50.20 ซม. *cm*

จะได้ระยะลดเหล็ก *Steel decreasing length* = 2.80-0.50 = 2.30 ม. *m* ใช้ *use* = 2.00 ม. *m*