Appendix-5　Other Relevant Data

5-1 Estimate for Design Flood Discharge

(1) Design flood discharge at the Bemos intake weir

The design flood discharge for the river control and development plan at the Bemos intake weir is decided in accordance with the following analysis.

1) Analysis-1: Based on data of rainfall and catchment area
2) Analysis-2: Based on data of flood trace and river surveying

① Analysis-1: Design flood discharge in accordance with the data of rainfall and catchment area

(a) Catchment area

The catchment areas in Bemos and Comoro rivers where were investigated in detail on "The Study on Community-based integrated watershed management in Laclo and Comoro river basins, JICA" are as follows.

Name of river	catchment areas
Comoro river (river mouth)	212.0 km^2
Bemos river（at the confluence of Comoro river）	43.9 km^2
Bemos river (at the intake facility)	30.3 km^2

(b) Rainfall intensity

The following nine rain gauge stations exist in the Comoro river basin. The data of the Dili rain gauge station, which has the longest observation period and the largest maximum annual rainfall (observation period: 30 years, maximum annual rainfall: 2,821mm), are used for analysis of design flood discharge though the Dare station (observation period: 22 years, maximum annual rainfall: 2,628mm) is the nearest river basin among them.

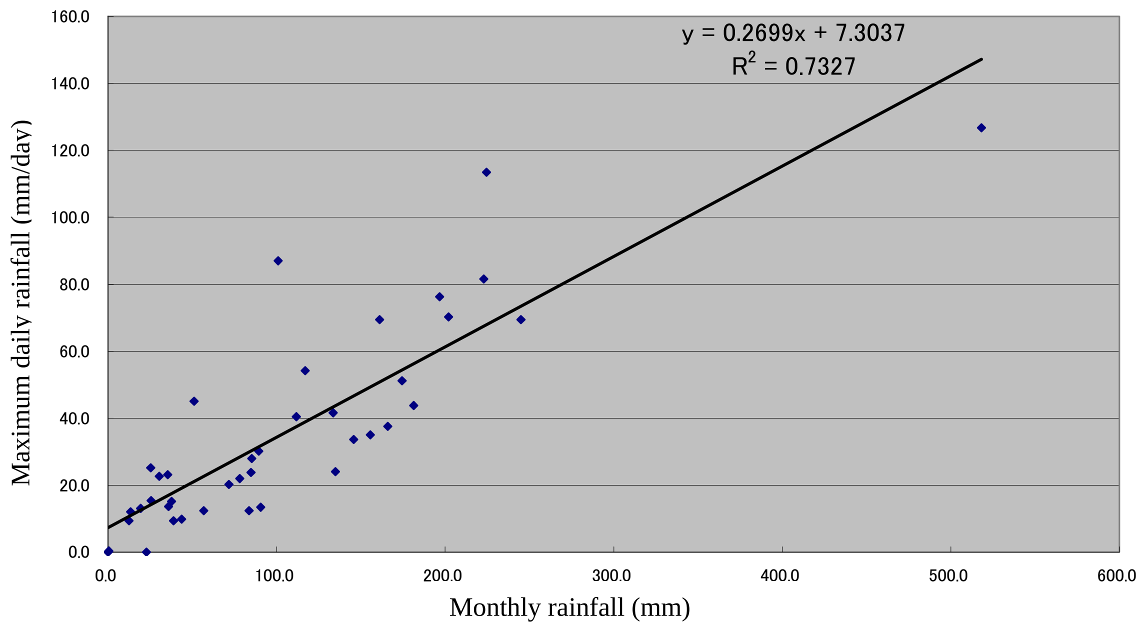
No.	Rain gauge station No.	Station name	Observation years	Observation period		location		altitude	Annual rainfall（mm）	
						Lat.	Long.	(m)	Max.	Min.
1	CT 7	Dare	22	1953-74		08°36′S	125°34′E	498	2,628	869
2	CT 8	Dili	30	1953-75, 1977-84		08°35′S	125°35′E	4, 15	2,821	475
3	CT 9	Ermera	7	1968-74		08°45′S	125°24′E	1,160	-	-
4	CT 11	Fasenda Algarve	23	1952-74		08°40′S	125°21′E	916	2,565	1,200
5	CT 13	Gleno	7	1968-74		08°43′S	125°27′E	770	-	-
6	RT 2	Lahane	2	1970-74		08°35′S	125°35′E	80	-	-
7	RT 4	Remexio	18	1956-64,	1966-74	08°37′S	125°40′E	875	3,879	1,325
8	RT 5	Aileu	17	1955-64,	1966-74	08°44′S	125°34′E	930	3,110	988
9	RT 10	Maubara	18	1956-64,	1966-74	08°37′S	125°12′E	15	1,225	588

In the Dili rain gauge station, the daily rainfall data exists only for 6 years from the year of 2003 to 2008 (however, the year of 2008 only in the rainy season from January to June) though the monthly rainfall data remain for 55 years from the year of 1953 to 2007.

Therefore, when the correlation between the monthly rainfall and the maximum daily rainfall in each month for the rainy season of the year of 2003 to 2008 (November to May) was examined, the equation (maximum daily rainfall in each month = 0.2699 × monthly rainfall + 7.3037) with the high correlation where its coefficient was 0.73 was derived. The maximum daily rainfall in each year from the year of 1953 to 2002 was estimated based on this equation with the maximum monthly rainfall.

Moreover, the probability was processed based on this presumed maximum daily rainfall and the actual maximum daily rainfall from the year of 2003 to 2007. The result of analysis shows that the daily rainfall of 113.4mm in 2005 and 126.7mm in 2004 corresponds to the probability in about 25 years and 77 years respectively.

Correlation between monthly rainfall and maximum daily rainfall



Return period	Probable daily rainfall (mm/day)	remarks
2	72.0	
3	81.3	
5	90.8	
10	101.5	
20	110.9	
Year of 2005	113.4	Correspond to R.P. 24.4 year
30	115.9	
50	121.9	
Year of 2004 (largest-ever)	126.7	Correspond to R.P. 76.6 year
100	129.6	
200	136.9	

Iwai's Method Calculation Results of Probability of Exceedance Location: Dili

Item: flood (rainy season)

unit: mm

both margins 10% Number of data N/10	minimum constant b
5	91.9

Standard deviation Sx	1/a
0.05576	0.07963

	xl Max	xs Min	xg $\log_{10}xg = \Sigma \log_{10}xi$	$xl \cdot xs - xg^2$	2xg−(xl+xs)	bs $xl \cdot xs - xg^2 / 2xg-(xl+xs)$	b mean (bs)
1	126.700	25.927	70.0772	−1625.89	−12.47	130.36	130.4
2	124.170	37.533	70.0772	−250.39	−21.55	11.62	71.0
3	113.400	42.931	70.0772	−42.50	−16.18	2.63	48.2
4	109.056	43.200	70.0772	−199.55	−12.10	16.49	40.3
5	99.070	43.200	70.0772	−630.97	−2.12	298.23	91.9 ◎
6	95.561	49.678	70.0772	−163.54	−5.08	32.16	81.9
7	95.561	51.837	70.0772	42.80	−7.24	−5.91	69.4
8	95.561	53.187	70.0772	171.76	−8.59	−19.99	58.2
9	92.592	53.727	70.0772	63.83	−6.16	−10.36	50.6
10	90.163	54.200	70.0772	−23.98	−4.21	5.70	46.1

return period T (year)	ξ	$1/a \cdot \xi$	平均Y $+1/a \cdot \xi$	x+b	probability by return period x
2	0.0000	0.0000	2.2143	163.8	71.939
3	0.3045	0.0242	2.2386	173.2	81.344
5	0.5951	0.0474	2.2617	182.7	90.823
10	0.9062	0.0722	2.2865	193.4	101.547
20	1.1630	0.0926	2.3069	202.7	110.871
30	1.2967	0.1033	2.3176	207.8	115.903
50	1.4520	0.1156	2.3299	213.8	121.904
100	1.6450	0.1310	2.3453	221.5	129.604
200	1.8215	0.1450	2.3594	228.8	136.888
300	1.9184	0.1528	2.3671	232.9	140.989

effective sample number N= 52		②	③	④	⑤	⑥	⑦	⑧	⑨	⑩
	Total	3,811.20		95.970		115.145	255.130	302,780.7	25,790,323.7	
	Total/N	73.29		1.846		2.214	4.906	5,822.7	495,967.8	

Note: return period of under 2years describes as 1 year

①		②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪				
Order n	YEAR	xi	Fn(%)	$\log_{10}xi$	xi+b	Y= log(xi+b)	Y^2	x^2	x^3	ξ	$period_1$	$period_2$	ξ_1	ξ_2	R.P.Year
1	2004	126.700	98.11	2.10278	218.566	2.33958	5.47365	16,052.890	2,033,901.2	1.5730	76	77	1.5709	1.5745	76.6
2	1998	124.170	96.23	2.09402	216.037	2.33453	5.45002	15,418.288	1,914,495.0	1.5095	61	62	1.5094	1.5141	61.0
3	2005	113.400	94.34	2.05461	205.266	2.31232	5.34681	12,859.560	1,458,274.1	1.2306	24	25	1.2246	1.2380	24.4
4	1992	109.056	92.45	2.03765	200.922	2.30303	5.30394	11,893.211	1,297,026.0	1.1139	17	18	1.1065	1.1263	17.4
5	1956	99.070	90.57	1.99594	190.936	2.28089	5.20245	9,814.805	972,349.8	0.8359	8	9	0.8134	0.8634	8.4
6	1954	95.561	88.68	1.98028	187.427	2.27283	5.16577	9,131.905	872,653.9	0.7347	6	7	0.6858	0.7547	6.7
6	1973	95.561	88.68	1.98028	187.427	2.27283	5.16577	9,131.905	872,653.9	0.7347	6	7	0.6858	0.7547	6.7
6	1995	95.561	88.68	1.98028	187.427	2.27283	5.16577	9,131.905	872,653.9	0.7347	6	7	0.6858	0.7547	6.7
9	1990	92.592	83.02	1.96657	184.458	2.26590	5.13429	8,573.297	793,819.6	0.6477	5	6	0.5951	0.6858	5.6
10	1978	90.163	81.13	1.95503	182.029	2.26014	5.10824	8,129.367	732,968.1	0.5754	4	5	0.4769	0.5951	4.8
11	1996	89.083	79.25	1.94980	180.950	2.25756	5.09657	7,935.852	706,952.7	0.5429	4	5	0.4769	0.5951	4.6
12	1991	88.814	77.36	1.94848	180.680	2.25691	5.09364	7,887.838	700,546.5	0.5348	4	5	0.4769	0.5951	4.5
13	1958	88.274	75.47	1.94583	180.140	2.25561	5.08778	7,792.246	687,850.4	0.5184	4	5	0.4769	0.5951	4.4
14	1999	86.924	73.58	1.93914	178.790	2.25234	5.07305	7,555.817	656,783.3	0.4774	4	5	0.4769	0.5951	4.0
15	1955	86.384	71.70	1.93644	178.251	2.25103	5.06714	7,462.265	644,623.2	0.4609	3	4	0.3045	0.4769	3.9
16	1966	85.035	69.81	1.92960	176.901	2.24773	5.05229	7,230.934	614,881.8	0.4195	3	4	0.3045	0.4769	3.7
17	1961	82.336	67.92	1.91559	174.202	2.24105	5.02232	6,779.200	558,171.6	0.3356	3	4	0.3045	0.4769	3.2
18	2008	81.600	66.04	1.91169	173.466	2.23921	5.01408	6,658.560	543,338.5	0.3126	3	4	0.3045	0.4769	3.0
19	1960	81.256	64.15	1.90986	173.122	2.23835	5.01023	6,602.586	536,501.7	0.3017	2	3	0.0000	0.3045	3.0
20	1988	80.177	62.26	1.90405	172.043	2.23564	4.99807	6,428.303	515,400.1	0.2676	2	3	0.0000	0.3045	2.9

(c) Probability of exceedance in the year applied for river improvement plan

According to the Technical Standard of River and Sediment Control edited by the Ministry of Land, Infrastructure and Transport, it is mentioned that the scale of the river development plan depends on the value of the importance degree of the river and considering current damage status, economical effect, etc. of the past flood

The Bemos River is an ordinary one flowing in mountainous area and both land sides of the river are mountain range. According to the above tables, the importance degree of the Bemos river is evaluated as Grade-D of the ordinary river and its scale of plan is considered probability of exceedance in 50 years. However, the daily rainfall of probability of exceedance in 50 years is 122mm/day calculated from rainfall observation data in Dili and largest-ever daily rainfall in February, 2004 is 126.7mm/day. A basic flood of the project is estimated by the rainfall 126.7mm per day of the largest-ever daily rainfall in February, 2004, in consideration of the difference of both daily rainfalls above is only 4.7mm (about 4%) and having received damage due to the largest-ever flood.

Importance degree of river:	Grade D
Scale of Plan:	the largest-ever flood (probability of exceedance in 50 years)
Design daily rainfall:	126.7mm/day

(d) Design flood discharge at the Bemos intake facility

1) Mean flood velocity

Rziha's formula: $W= 20(h/L)^{0.6}$

Where, W : flood velocity (m/sec)

h : Elevation difference between the upstream end and the downstream end of watercourse (m)

$h = EL.\ 800m – EL.\ 227m = 573m$

L : length of watercourse (m)、L = 8,500m

$$W = 20\times(573/8{,}500)^{0.6} = 3.8m/sec$$

2) Lag time of flood

$T = t_1 + t_2$

Where, T: Lag time of flood（hr）

t_1 : flood inflow time (hr), mountain basin（$2km^2$）: 0.5hr

t_2 : flood runoff time (hr), $t_2 = L / W = 8{,}500 / 3.8m/sec = 0.6hr$

Lag time of flood: T = 0.5 + 0.6 = 1.1hr

3) Rainfall intensity within lag time flood

Mononobe's formula: $r = R_{24}/24 \cdot (24/T)^n$

Where, r : rainfall intensity within lag time flood (mm/hr)

R_{24} : daily rainfall of the largest-ever maximum flood on 6 February 2004 (mm/day)

R_{24} = 126.7mm/day

T : lag time of flood, T = 1.1 hr

n : topography coefficient, n = 0.6

The largest-ever maximum rainfall intensity: $r_{max} = 126.7/24\times(24/1.1)^{0.6} = 33.6$mm/hr

4) Design flood discharge by rainfall analysis

Rational formula: $Q_{f1} = f \cdot r \cdot A / 3.6$

Where, Q_{f1} : design flood discharge by rainfall analysis (m^3/sec)

f : runoff coefficient, f = 0.7 (mountain area)

r_{max} : largest-ever maximum rainfall intensity, r_{max} = 33.6mm/hr

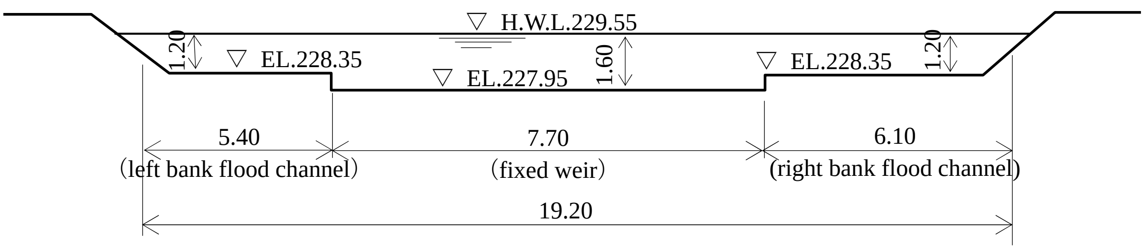
A : catchment area, A = 30.3km^2

Largest-ever maximum design flood discharge: $Q_{maxf1} = 0.7 \times 33.6 \times 30.3 / 3.6 = 198m^3/sec$

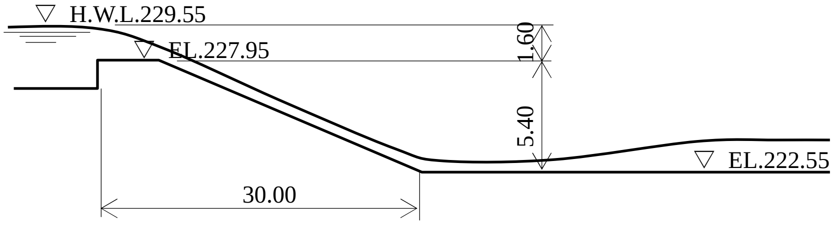
② Analysis-2: Design flood discharge in accordance with the data for flood trace and river surveying

The design flood discharge at the Bemos intake weir is estimated in accordance with the data for flood trace (1.2m in depth in left bank cannel according to the interview for the caretaker) and the river data at the intake weir.

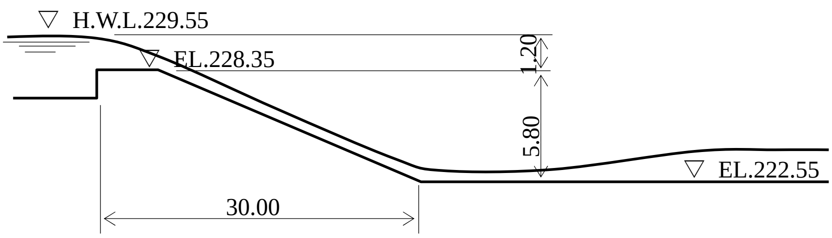
(a) Parameter of intake weir



Front view of intake weir



Section of fixed weir



Section of flood channel

Note: water levels described in the figures depend on the interview for the caretaker

(b) Overflow discharge at intake weir

As the height of 5.4m of the intake weir is larger than overflow depth of the weir, the overflow at the intake weir becomes a free overflow.

Overflow discharge: $Q_{f2} = C \times B \times H^{3/2}$

Where, Q_{f2} : overflow discharge of weir (m^3/sec)

C : coefficient of overflow C = 1.70 (free overflow of broad crested weir)

B : width of overflow（m）

H : overflow head, $H = h + hv$（m）

1) Flood water level by the interview（H.W.L. 229.55m）

According to the interview for the caretaker of the intake facility, the largest-ever maximum flood water level was the depth of 1.20m from the flood channel. Therefore, the largest-ever maximum flood water level is EL. 228.35m + 1.20m = H.W.L. 229.55m. Then, the velocity is 5.03m/sec and the velocity head is 1.29m regarding the calculation results of uniform flow at the upstream of the weir.

a) Overflow discharge of fixed weir: Q_1

$$Q_1 = 1.70 \times 7.70 \times (1.60 + 1.29)^{3/2} = 64.3 m^3/sec$$

b) Overflow discharge of the left bank flood channel: Q_2

$$Q_2 = 1.70 \times (4.80 + 1.20/2) \times (1.20 + 1.29)^{3/2} = 36.1 m^3/sec$$

c) Overflow discharge of the right bank flood channel: Q_3

$$Q_3 = 1.70 \times (5.50 + 1.20/2) \times (1.20 + 1.29)^{3/2} = 40.8 m^3/sec$$

d) Overflow discharge of weir: Q_{f21}

$$Q_{f21} = 64.3 + 36.1 + 40.8 = 141.2 m^3/sec$$

Therefore, based on the interview survey, the overflow discharge at the flood water level of 229.55m is Q_{f21} =141.2m^3/sec.

2) Flood water level depending on the largest-ever maximum flood (Q_{maxf2} = 198m^3/sec)

The largest-ever maximum flood discharge is 198m^3/sec. The water level at the upstream of weir at the discharge of 198m^3/sec is assumed as follows.

Assumed flood water level at the largest-ever maximum flood: H.W.L. 229.95m

Moreover, based on the calculation results of uniform flow at the upstream of the weir, the velocity is 5.59m/sec and the velocity head is 1.59m.

a) Overflow discharge of fixed weir: Q_1

$$Q_1 = 1.70 \times 7.70 \times (2.00 + 1.59)^{3/2} = 89.2\text{m}^3/\text{sec}$$

b) Overflow discharge of the left bank flood channel: Q_2

$$Q_2 = 1.70 \times (4.80 + 1.60/2) \times (1.60 + 1.59)^{3/2} = 54.4\text{m}^3/\text{sec}$$

c) Overflow discharge of the right bank flood channel: Q_3

$$Q_3 = 1.70 \times (5.50 + 1.60/2) \times (1.60 + 1.59)^{3/2} = 61.1\text{m}^3/\text{sec}$$

d) Overflow discharge of weir: Q_{f21}

$$Q_{f2\,1} = 89.2 + 54.4 + 61.1 = 204.7\text{m}^3/\text{sec} \fallingdotseq 198\text{m}^3/\text{sec}$$

Therefore, the flood water level at the largest-ever flood is estimated at 229.95m

③ Summary on the design flood discharge

The analysis results at the time of flood of the Bemos intake weir are as follows.

item	unit	Largest-ever maximum flood	Maximum flood by the interview survey
1. catchment area	km^2	30.3	
2. length of watercourse	m	8,500	
3. elevation difference of watercourse	m	573 (= EL.800m – EL.227m)	
4. mean watercourse slope	-	1/15	
5. lag time of flood	hr	1.1	
6. daily rainfall	mm/day	126.7	Correspond to 90.2
7. rainfall intensity	mm/hr	33.6	Correspond to 23.9
8. design flood discharge	m^3/sec	198	141
9. flood water level at the upstream of weir	m	H.W.L.229.95m	H.W.L.229.55m
10. velocity at the upstream of weir	m/sec	5.59	5.03
11. velocity head at the upstream of weir.	m	1.59	1.29

From the above table, the flood discharge at the maximum flood water level of H.W.L.229.55m (flood channel water depth: 1.2m) by the interview survey is Q_f = 141m^3/sec. Though the discharge is lower than the estimated largest-ever maximum flood discharge of 198m^3/sec, it is assumed that the interview data is based on the memory after several years so that the observation value has shifted at the time of the flood peak and has given a small result.

Therefore, the design flood discharge on the Bemos river control and development plan is decided to the largest-ever maximum discharge in correspondence with the probability of exceedance in 50 years in consideration of the importance of river structures and the influence of the flood struck etc.

- Largest-ever maximum flood discharge: Q_{max} = 198m^3/sec ≒ 200m^3/sec、
- Flood water level at intake weir: H.W.L.229.95m

④ Design discharge of low water channel revetment

The river channel is formed due to the flood that occurs a few times in a year. This discharge is called control discharge. Based on the result of the probability processing of the rainfall, if the probability exceedance in 2 years of daily rainfall (72mm/day) is adopted, the control discharge for the catchment area of 30.3km^2 is estimated as follows.

Control discharge: Qc = 72mm/day × 198m^3/sec / 126.7mm/month = 112.5m^3/sec ≒ 110m^3/sec

Therefore, the height of the low water channel revetment is decided to be able to flow the control discharge of 110m^3/sec.

(2) Design specific flood discharge in the tributary

① Lag time of flood

$T = t_1 + t_2$

Where, T: lag time of flood（hr）

$t_{1:}$: flood inflow time (hr), mountain basin（2km^2）: 0.5hr

t_2 : flood runoff time (hr), t_2 = 0.0hr

Lag time of flood: T = 0.5 + 0.0 = 0.5hr

② Rainfall intensity within lag time flood

Mononobe's formula: $r = R_{24}/24 \times (24/T)^n$

Where, r : rainfall intensity within lag time flood (mm/hr)

R_{24} : daily rainfall of the largest-ever maximum flood on 6 February 2004 （mm/day）

T : lag time of flood, T = 0.5hr

n : topography coefficient, n = 0.6

The largest-ever maximum rainfall intensity in the tributary: $r = 126.7/24 \times (24/0.5)^{0.6}$ = 53.9mm/hr

③ Design specific flood discharge in the tributary

Rational formula: $q_{bf} = f \times r / 3.6$

Where, q_{bf} : design specific flood discharge in the tributary (m^3/sec/km^2)

f : runoff coefficient, $f = 0.7$ (mountain area)

r_{max} : largest-ever maximum rainfall intensity, $r_{max} = 53.9$mm/hr

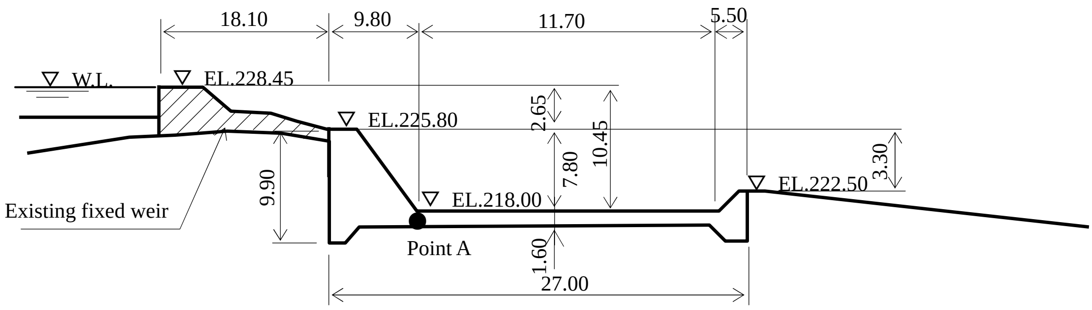
Design specific flood discharge in the tributary: $q_{bf} = 0.7 \times 53.9 / 3.6 = 10.5\text{m}^3/\text{sec/km}^2$

④ Specific control discharge in the tributary

Rainfall intensity within lag time
of control discharge in the tributary: $r = 72.0/24 \times (24/0.5)^{0.6} = 30.6$mm/hr

Specific control discharge in the tributary: $q_{bc} = 0.7 \times 30.6 \times 1.00 / 3.6 = 5.95\text{m}^3/\text{sec/km}^2$

5-2 Design of Intake Weir



Section of intake weir planned

(1) Downstream apron of Intake Weir

① Downstream apron length

As the downstream riverbed is in danger of scouring from overflow on the weir, a downstream apron of the intake weir is designed to protect the downstream riverbed from scouring. Based on "Headworks Design Standard of Ministry of Agriculture, Forestry and Fisheries, Japan", the length of the apron is planned as follows.

Downstream apron length is given by Bligh’s formula.

$$l_1 = 0.6 \times C\sqrt{D_1} \qquad = 0.6 \times 6 \times \sqrt{10.45} = 11.64\text{m} \leqq 11.70\text{m}$$

where, l_1 : Downstream apron length(m)

D_1 : height from above the downstream apron edge to fixed weir crest (m)

D_1 = EL 228.45m – EL 218.00 m = 10.45m

C : Bligh’s coefficient (gravel), C=6

Therefore, downstream apron length becomes 11.70m.

② Examination of Creep Length

It is necessary to secure a creep length along the weir foundation and the rear of the retaining wall for the prevention of piping. The creep length required should adopt the bigger numerical value calculated by the Bligh’s method and the Lane’s one (Refer to the headworks design standard).

There is no danger of piping for the existing weir because it sticks firmly to the bedrock. The new fixed weir added to the downstream requires examining the prevention of the piping because it is to be constructed on the gravel layer.

Therefore, maximum water level difference between the upstream and the downstream is EL. 225.80m – EL. 222.50m = 3.30m.

③ Examination of Creep Length

i) Bligh’s method

$S \geqq C \times \triangle H = 6 \times 3.30 = 19.80m \leqq 38.40m$

where, S : Creep length along ground-contact surface of the weir (m)

$S = 9.90 + 0.50 + 1.00 + 27.00 = 38.40m$

C : Bligh's coefficient (gravel), C=6

$\triangle H$: the maximum water level difference between up- and downstream

$\triangle H$ = EL. 225.80m – EL. 222.50m =3.30 m

ii) Lane's method

$L \geqq C' \times \triangle H = 2.5 \times 3.30 = 8.25m \leqq 20.40m$

where, L: weighted creep length (m), $L = \Sigma \ell_v + 1/3 \cdot \Sigma \ell_h$

$L = (9.90 + 0.50 + 1.00) + 1/3 \times 27.00 = 20.40m$

C': Lane's weighted creep coefficient (drift stones including boulders and gravel), C' = 2.5

$\triangle H$: the maximum water level difference between up- and downstream

$\triangle H = 3.30m$

As a result, the downstream apron length of 11.70m satisfies both the equations above, and is inferred to be safe.

④ Thickness of downstream apron

The thickness of the downstream apron is obtained from the following equation concerning the uplifting pressure balance (Refer to the headworks design standard).

$t \geqq 4/3 \times (\triangle H - H_f) / (\gamma - 1)$

where, t : Apron thickness at a point of interest (m)

$\triangle H$: the maximum water level difference between up- and downstream, $\triangle H = 3.30m$

H_f : Head loss of seepage water to the point of interest (m)

γ : Specific gravity of the material of weir and apron, $\gamma = 2.30\ t_f/m^3$

4/3 : Safety factor

- Overall creep length:

  $L = 9.90 + 0.50 + 1.00 + 27.00 = 38.40m$

- Creep length to Point A :

  $L_A = 9.90 + 0.50 + 9.80 = 20.20m$

- Head loss of seepage water to Point A :

  $H_f = L_A/L \times \triangle H = 20.20/38.40 \times 3.30 = 1.74m$

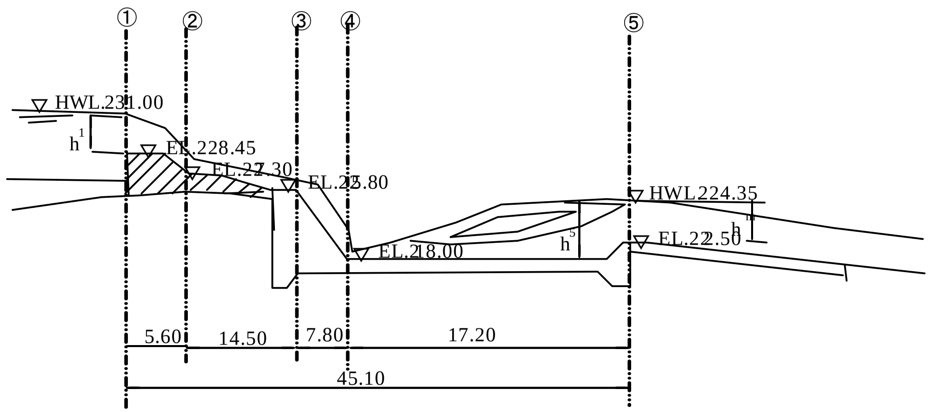
- Apron thickness:

  $t \geqq 4/3 \times (\triangle H - H_f) / (\gamma - 1)$

  $= 4/3 \times (3.30 - 1.74) / (2.30 - 1) = 1.59m \leqq 1.60m$

⑤ The altitude of the apron bottom of energy dissipator pond type

For the supercritical flow at the toe of the weir takes place to energy dissipation as a hydraulic jump, the altitude (EL.218.00m) of the apron bottom of energy dissipator pond type is examined by the trial calculation so that the downstream water level (H.W.L.224.35m) of the weir should be higher than the conjugate water level (W.L.224.11m) to the supercritical flow as follows.



Calculation of non-uniform flow in the downstream of intake weir at the time of the design flood

Location	discharge of unit width q ($m^3/s/m$)	water depth h_i (m)	flow area A_i (m^2)	wetted perimeter Pi (m)	Hydrualic radious Ri (m)	velocity V_i(m/s)	roughness coeeficient ni	hydraulic gradient Ii	velocity head h_{vi} (m)	froude number Fri	sill Zi (m)	water level WL.(m)	energy height Ei (m)	distance of section Li (m)	head loss hi (m)	Ei - hi (m)
①	11.18	2.00	2.00	1.00	2.000	5.59	0.015	0.00279	1.59	1.26	228.45	230.45	232.04	-	0.00	232.04
②	11.18	1.46	1.46	1.00	1.460	7.66	0.015	0.00796	2.99	2.02	227.30	228.76	231.75	5.60	0.31	231.73
③	11.18	1.21	1.21	1.00	1.210	9.24	0.015	0.01490	4.36	2.68	225.80	227.01	231.37	14.50	0.44	231.31
④	11.18	0.79	0.79	1.00	0.790	14.15	0.015	0.06169	10.22	5.09	219.00	219.79	230.01	6.80	1.43	229.93
⑤	11.18	5.30	5.30	1.00	5.301	2.11	0.015	0.00011	0.23	0.29	219.00	224.30	224.53	15.70	0.49	229.52
downstream water level	11.69	1.85	1.85	1.00	1.850	6.32	0.015	0.00396	2.04	1.48	222.50	224.35	226.39	0.00	0.00	224.53

(2) **Downstream riverbed protection of intake weir**

① Length of downstream riverbed protection

The length of downstream riverbed protection of pond type energy dissipator is made up to the downstream edge of hydraulic jump vortex.

Smetana's formula,

$L = 6 \times (h_5 - h_4)$

Where, L : length of hydraulic jump (m)
h_4 : depth of supercritical flow side, $h_4 = 0.95$m
h_5 : sequent depth of subcritical flow side, $h_5 = 6.11$m

$L_{max} = 6 \times (6.11 - 0.95) = 30.94$m
Riverbed protection length: $Lr = L_{max} - La = 30.94 - (11.70 + 5.50) = 13.74$m

Therefore, the riverbed protection length is 15.00m (5 @ 3.00m).

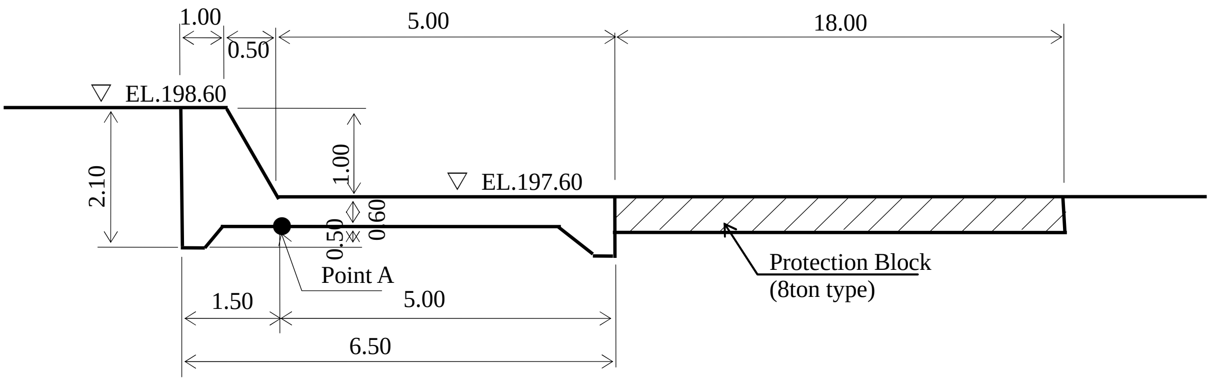
② Type and Weight of Downstream Riverbed Protection

The type and weight of downstream riverbed protection of intake weir are selected from "2-2-2-6 Selection of riverbed protection".

Flow velocity on the downstream riverbed protection is V= 7.20m/sec at the design flood (Q = 200m^3/sec).

Therefore, the weight of the downstream riverbed protection block is 8 tf/piece (3.00m × 3.00 × 1.00m crossing type concrete block and the maximum allowable flow velocity: 7.5m/sec) is adopted.

5-3 Design of Groundsill on the River Crossing No.1



Typical Cross Section of Groundsill

(1) Downstream Apron

① Length of Downstream Apron

As the downstream riverbed is in danger of scouring from overflow on the groundsill, the apron at the groundsill downstream side is designed to protect the downstream riverbed from scouring. The length of the downstream apron is planned by "Headworks Design Standard of Ministry of Agriculture, Forestry and Fisheries, page 207" as follows.

The length of the downstream apron is obtained using the Bligh's formula.

$$l_1 = 0.6 \times C\sqrt{D_1} \quad = 0.6 \times 6 \times \sqrt{1.00} = 3.60\text{m} \leqq 5.00\text{m}$$

where, l_1 : Length of the downstream apron (m)

D_1 : Elevation from above the apron downstream end to the crest of groundsill(m)

D_1 = EL 198.60m – EL 197.60m = 1.00m

C : Bligh's coefficient, (gravel and sand) C=6

Therefore, the length of the downstream apron of groundsill is determined to be 5.00 m.

② Examination of creep length

It is essential to secure a creep length along with ground-contact surface of the weir or back face of bank protection retaining walls for prevention of piping. The creep length to prevent piping is calculated using two methods: Bligh's and Lane's methods. After comparing two values to each other, the larger one is adopted as the minimum length of creep length (refer to Headworks Design Standard of MAFF, page 192).

i) Bligh's method

$$S \geqq C \times \triangle H = 6 \times 1.00 = 6.00\text{m} \leqq 9.10\text{m}$$

Where, S : Creep length along with ground-contact surface of the weir (m)

S = 2.10 + 0.50 + 6.50 = 9.10m

C : Bligh’s coefficient, (sand and gravel) C=6

$\angle H$: the maximum water level difference between up- and downstream sides

$\angle H$ = EL. 198.60m – EL. 197.60m =1.00 m

ii) Lane’s method

$L \geqq C' \times \angle H = 2.5 \times 1.00 = 2.50m \leqq 4.77m$

Where, L: Weighted creep length (m), $L = \Sigma \ell_v + 1/3 \cdot \Sigma \ell_h$

$L = (2.10 + 0.50) + 1/3 \times 6.50 = 4.77m$

C’ : Lane’s weighted coefficient, (Medium size of gravel) C’ = 2.5

$\angle H$: the maximum water level difference between up- and downstream sides

$\angle H = 1.00m$

As a result, the downstream apron length of 5.00 m satisfies both equations above and is inferred to be safe.

③ Thickness of downstream apron

The thickness of the downstream apron is obtained from the following equation concerning the uplifting pressure balance (refer to Headworks Design Standard of MAFF, page 207).

$t \geqq 4/3 \times (\angle H - H_f) / (\gamma - 1)$

where, t : Apron thickness at a point of interest (m)

$\angle H$: the maximum water level difference between up- and downstream sides

$\angle H = 1.00m$

H_f : Head loss of seepage water to the point of interest (m)

γ : Specific gravity of the material of weir and apron

$\gamma = 2.30\ t_f/m^3$

4/3 : Safety factor

- Overall creep length:

  $L = 2.10 + 0.50 + 6.50 = 9.10m$

- Creep length to Point A:

  $L_A = 2.10 + 0.50 + 1.50 = 4.10m$

- Head loss of seepage water to Point A:

  $H_f = L_A/L \times \angle H = 4.10/9.10 \times 1.00 = 0.45m$

- Apron thickness:

  $t \geqq 4/3 \times (\angle H - H_f) / (\gamma - 1)$

  $= 4/3 \times (1.00 - 0.45) / (2.30 - 1) = 0.56m \leqq 0.60m$

Consequently, the apron thickness at Point A is determined to be 0.60 m.

(2) Downstream Riverbed Protection

① Length of downstream riverbed protection

The length of downstream riverbed protection is examined using an empirical formula of the Bligh's one.

$$L_r = L - l_a$$

$$L = 0.67 \times C \times \sqrt{\Delta H \cdot q} \text{ x } f$$

Where, L_r : Length of riverbed protection (m)

L : Total length of protection including length of apron l_a and length of riverbed protection L_r (m)

l_a : Downstream apron length, $l_a = 6.00$m

ΔH : Maximum water level difference (m)

ΔH = W.L. 198.60m − EL. 197.60m = 1.00m

q : flow per unit width of maximum design flood discharge (m^3/sec/m)

q = 6.59m/sec × 2.00m = 13.18 m^3/sec/m

$$L = 0.67 \times 6 \times \sqrt{1.00x13.18} \times 1.5 = 21.89\text{m}$$

Therefore, the length of downstream riverbed protection is determined to be L_r = 21.89 − 5.00 = 16.89m ≒ 18.00m.

② Weight of Downstream Riverbed Protection

Downstream riverbed protection blocks must be stable against water flow. The approximate weight of a riverbed protection block is determined as follows (refer to Headworks Design Standard of MAFF, page 259).

$$W > 3.75 \times A \times V^2 / 2g$$

Where, W : weight of each block (tf/piece)

A : area of collision with flowing water, A = 2.70m × 0.30m = 0.81m^2

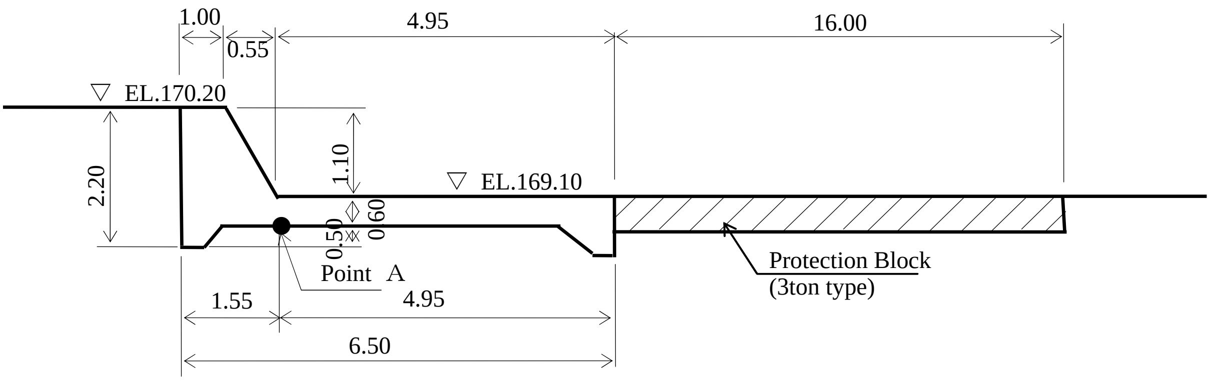
V : velocity at which flowing water collides with block (m/sec), V = 6.59m/sec

g : acceleration of gravity, g = 9.80 m/sec^2

$$W > 3.75 \times 0.81 \times 6.59^2 / (2 \times 9.8) = 6.73 \text{ tf/piece}$$

Therefore, the weight of downstream riverbed protection block is adopted to be 8tf/piece (crossing type concrete block: 3.00m × 3.00 × 1.00m).

5-4 Design of Groundsill on the River Crossing No.2



Typical Cross Section of Groundsill

(1) Downstream Apron

① Length of Downstream Apron

As the downstream riverbed is in danger of scouring from overflow on the groundsill, the apron at the groundsill downstream side is designed to protect the downstream riverbed from scouring. The length of the downstream apron is planned by "Headworks Design Standard of Ministry of Agriculture, Forestry and Fisheries, page 207" as follows.

The length of the downstream apron is obtained using the Bligh's formula.

$$l_1 = 0.6 \times C\sqrt{D_1} \qquad = 0.6 \times 6 \times \sqrt{1.10} = 3.78\text{m} \leqq 4.95\text{m}$$

where, l_1 : Length of the downstream apron (m)

D_1 : Elevation from above the apron downstream end to the crest of groundsill(m)

D_1 = EL 170.20m – EL 169.10m = 1.10m

C : Bligh's coefficient, (gravel and sand) C=6

Therefore, the length of the downstream apron of groundsill is determined to be 4.95 m.

② Examination of creep length

It is essential to secure a creep length along with ground-contact surface of the weir or back face of bank protection retaining walls for prevention of piping. The creep length to prevent piping is calculated using two methods: Bligh's and Lane's methods. After comparing two values to each other, the larger one is adopted as the minimum length of creep length (refer to Headworks Design Standard of MAFF).

i) Bligh's method

$$S \geqq C \times \triangle H = 6 \times 1.10 = 6.60\text{m} \leqq 9.20\text{m}$$

Where, S : Creep length along with ground-contact surface of the weir (m)

S = 2.20 + 0.50 + 6.50 = 9.20m

C : Bligh's coefficient, (sand and gravel) C=6

$\triangle H$: the maximum water level difference between up- and downstream sides

$\triangle H$ = EL. 170.20m – EL. 169.10m =1.10 m

ii) Lane's method

$L \geqq C' \times \triangle H = 2.5 \times 1.10 = 2.75m \leqq 4.77m$

Where, L: Weighted creep length (m), $L = \Sigma \ \ell_v + 1/3 \cdot \Sigma \ \ell_h$

$L = (2.20 + 0.50) + 1/3 \times 6.50 = 4.87m$

C' : Lane's weighted coefficient, (Medium size of gravel) C' = 2.5

$\triangle H$: the maximum water level difference between up- and downstream sides

$\triangle H$ = 1.10m

As a result, the downstream apron length of 4.95 m satisfies both equations above and is inferred to be safe.

③ Thickness of downstream apron

The thickness of the downstream apron is obtained from the following equation concerning the uplifting pressure balance (refer to Headworks Design Standard of MAFF, page 207).

$t \geqq 4/3 \times (\triangle H - H_f) / (\gamma - 1)$

where, t : Apron thickness at a point of interest (m)

$\triangle H$: the maximum water level difference between up- and downstream sides

$\triangle H$ = 1.10m

H_f : Head loss of seepage water to the point of interest (m)

γ : Specific gravity of the material of weir and apron

$\gamma = 2.30\ t_f/m^3$

4/3 : Safety factor

- Overall creep length:

  L = 2.20 + 0.50 + 6.50 = 9.20m

- Creep length to Point A :

  L_A = 2.20 +0.50 + 1.55 = 4.25m

- Head loss of seepage water to Point A:

  $H_f = L_A/L \times \triangle H = 4.25/9.20 \times 1.10 = 0.51m$

- Apron thickness:

  $t \geqq 4/3 \times (\triangle H - H_f) / (\gamma - 1)$

  $= 4/3 \times (1.10 - 0.51) / (2.30 - 1) = 0.59m \leqq 0.60m$

Consequently, the apron thickness at Point A is determined to be 0.60 m.

(3) Downstream Riverbed Protection

① Length of downstream riverbed protection

The length of downstream riverbed protection is examined using an empirical formula of the Bligh's one.

$$L_r = L - l_a$$

$$L = 0.67 \times C \times \sqrt{\Delta H \cdot q} \times f$$

Where, L_r : Length of riverbed protection (m)

L : Total length of protection including length of apron l_a and length of riverbed protection L_r (m)

l_a : Downstream apron length, $l_a = 6.00\text{m}$

$\triangle H$: Maximum water level difference (m)

$\triangle H = \text{W.L. } 170.20\text{m} - \text{EL. } 169.10\text{m} = 1.10\text{m}$

q : flow per unit width of maximum design flood discharge ($\text{m}^3/\text{sec/m}$)

$q = 5.12\text{m/sec} \times 2.10\text{m} = 10.75\text{m}^3/\text{sec/m}$

$$L = 0.67 \times 6 \times \sqrt{1.10 x 10.75} \times 1.5 = 20.74\text{m}$$

Therefore, the length of downstream riverbed protection is determined to be

$$L_r = 20.74 - 4.95 = 15.79\text{m} \fallingdotseq 16.00\text{m}.$$

② Weight of Downstream Riverbed Protection

Downstream riverbed protection blocks must be stable against water flow. The approximate weight of a riverbed protection block is determined as follows (refer to Headworks Design Standard of MAFF).

$$W > 3.75 \times A \times V^2 / 2g$$

Where, W : weight of each block (tf/piece)

A : area of collision with flowing water, $A = 1.70\text{m} \times 0.30\text{m} = 0.51\text{m}^2$

V : velocity at which flowing water collides with block (m/sec), $V = 5.12\text{m/sec}$

g acceleration of gravity, $g = 9.80\text{m/sec}^2$

$$W > 3.75 \times 0.51 \times 5.12^2 / (2 \times 9.8) = 2.51\text{tf/piece}$$

Therefore, the weight of downstream riverbed protection block is adopted to be 3tf/piece (crossing type concrete block: 2.00m × 2.00 × 1.00m).

5-5 Conditions for Structure Design

In Japan, the design method for civil structures has not been firmly established yet, and various design technical standard adopt the past "Allowable stress design method", though recently the structural design method in Japan is gradually shifting to "Limit State Design Method". The structural design method employed under this project is "Allowable stress design method" considering such situation.

In the latest standard specifications for concrete structures, the one that presents "Allowable stress design method" is the Standard Specifications for Concrete Structures-2002 "Structural Performance Verification" Japan Society of Civil Engineers. Therefore, the allowable unit stress and the loads etc. of the materials must conform to the above mentioned specifications.

(1) Allowable Stress of Materials

① Allowable Stress of Reinforced concrete

Allowable stress of reinforced concrete

Allowable stress (N/mm²)		28 days concrete strength (N/mm²)	
		$f'_{ck} = 21.0$	$f'_{ck} = 35.0$
Allowable bending compressive stress		8.0	12.5
Allowable shearing stress	beam	0.42	0.52
	slab	0.85	1.05
Allowable bond stress	rounded steel bar	0.75	0.95
	deformed bar	1.5	1.9
Allowable bearing stress		6.3	10.5
Applicable Structures		Normal reinforced concrete structures	Abrasion resistance reinforced concrete structures

Note: 1. Allowable values of bending compressive, shearing and bond stress are adopted to the average of 18～24N/mm2 and 30～40N/mm2

2. Allowable bearing stress: $\sigma'_{ca} = 0.3 f'_{ck}$

② Allowable Stress of Plain Concrete

Allowable stress of plain concrete

Allowable Stress (N/mm²)	28 days concrete strength (N/mm²)	
	$f'_{ck} = 18.0$	$f'_{ck} = 21.0$
Allowable bending compressive stress	4.5	5.2
Allowable bending tensile stress	0.29	0.29
Allowable bearing stress	5.4	5.9
Applicable Structures	Lean concrete, cap concrete, base concrete	Gravity type retaining wall, concrete pedestal for raw water main

Note: 1. Allowable compressive stress: $\sigma'_{ca} = f'_{ck} / 4 \leqq 5.4\text{N/mm}^2$

2. Allowable bending tensile stress: $\sigma'_{sa} = f'_{ck} / 7 \leqq 0.29\text{N/mm}^2$

3. Allowable bearing stress: $\sigma'_{ca} = 0.3 f'_{ck} \leqq 5.9 N/mm^2$

③ Allowable Tensile Stress of Reinforcing Bar

Allowable tensile stress of reinforced bar

Condition of reinforced concrete	Allowable Tensile Stress (N/mm^2)	
	SD 295	SD 345
In the air, in the ground	176	196
In water	157	176
At the earthquake	264	294

Note: 1. When reinforced concrete always immerses in water, allowable tensile stress adopts the same value that is decided from fatigue strength.

2. Allowable tensile stress at earthquake adopts 1.5 times of the one in the air or in water

(2) Loads

① Dead Loads

The unit weight of each material is as follows.

Unit weight of materials

Type of material	symbol	unit weight (kN/m^3)
Reinforced concrete	γ_c	24.5
Plain concrete	γ_c	23.0
water	γ_w	9.8
wet soil	γ_t	18.0
Saturated soil	γ_{sa}	20.0
Submerged soil	γ_{su}	10.0
Steel materials	γ_s	77.0

② Live Loads

Live loads are as follows.

Live loads acting on structures

Type of live load		Live load	
		(kN)	(kN/m^2)
Traffic loads		250	10.0
Pedestrians loads	General (no heavy vehicles passage)	-	3.0
	Public road (heavy vehicles passage)	-	5.0

③ Seismic Loads

According to the earthquake occurrence record around East Timor including Indonesia (the year of 1938 to 2008), a big earthquake of magnitude 6.0 to 8.5 has occurred seven times. On the other hand, according to the earthquake occurrence record in Japan (in recent100 years, the year of 1891 to 1995), a big earthquake of magnitude 6.9 to 9.0 has been recorded 22 times.

Therefore, the earthquake is considered as same as Japan. According to the earthquake-proof design standard in Japan, it is described as “Even if the influence of the earthquake is not especially considered, it is admitted for the earthquake of the scale to usually endure functionally according to the experience of the past when the design and construction are elaborately done. Therefore, you may omit the stability examination at the earthquake in a general structure of 8m or less in height.”

As the structures of 8m or more in height are not planned in this project, the stability examination for structures at the earthquake is considered unnecessary.

5-6 Alternative Fig. 1-1 to Fig. 4-3

List of Alternative Figures

Alternative Figure No.	Title
FIG. 1-1	Alternative for Structure No.5 Plan of Tributary Crossing (Improvement of Existing Pipeline & Tributary Crossing)
FIG. 1-2	Alternative for Structure No.5 Plan of Tributary Crossing (Improvement of Existing Pipeline & Tributary Crossing)
FIG. 2-1	Alternative for Structure No.10 Plan, Profile & Sections (Rerouting 1/3)
FIG. 2-2	Alternative for Structure No.10 Plan, Profile & Sections (Rerouting 2/3)
FIG. 2-3	Alternative for Structure No.10 Plan, Profile & Sections (Rerouting 3/3)
FIG. 3	Alternative for Structure No. 10-3 Plan, Profile & Section (Improvement of Existing Pipeline by Slope Protection)
FIG. 4-1	Concrete Pavement of Access Road (Alternative-A: Existing Slope)
FIG. 4-2	Concrete Pavement of Access Road (Alternative-B: Cutting Method)
FIG. 4-3	Concrete Pavement of Access Road (Alternative-C: Embankment Method)

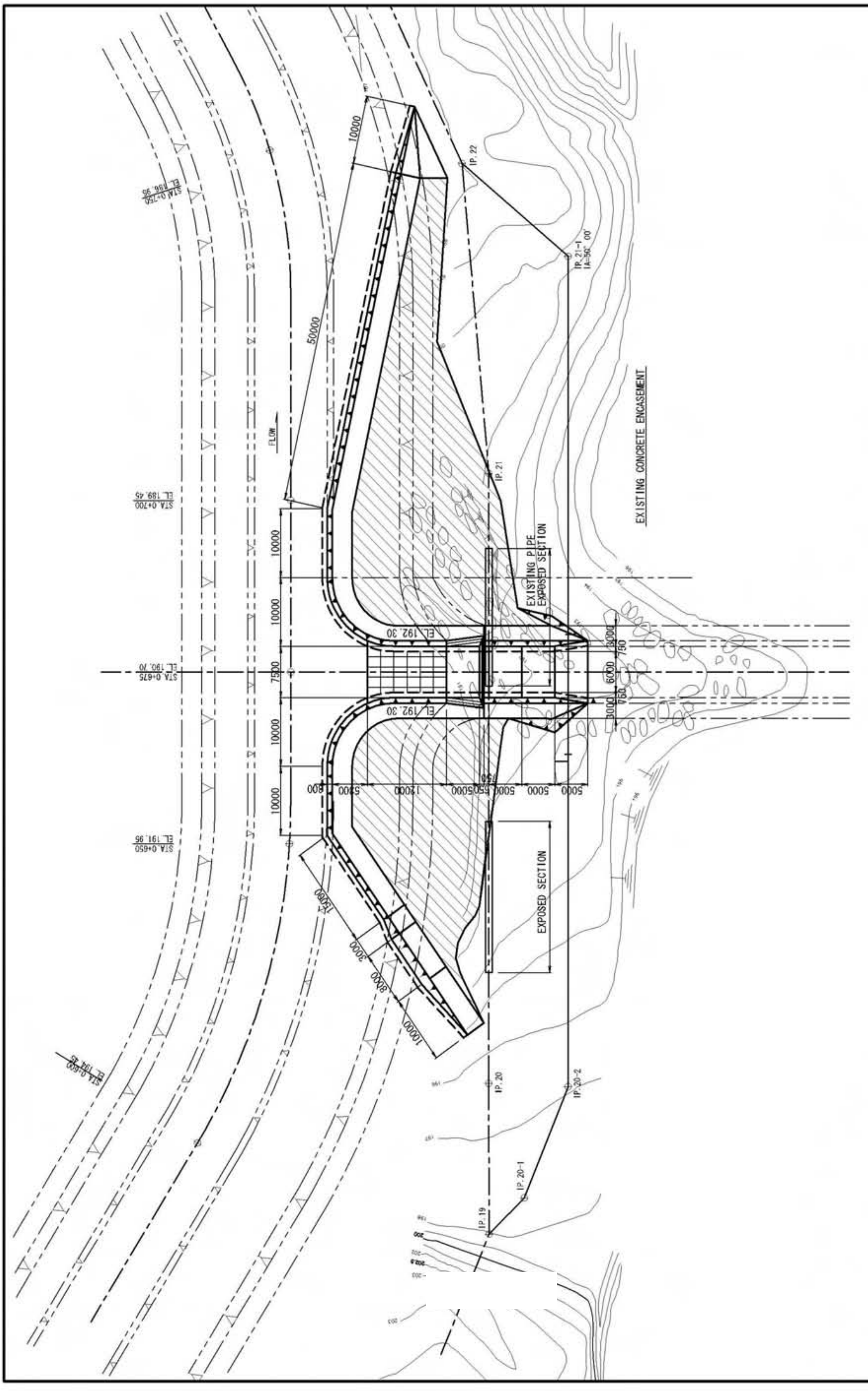


FIG. 1-1 ALTERNATIVE FOR STRUCTURE No. 5 PLAN OF TRIBUTARY CROSSING
(IMPROVEMENT OF EXISTING PIPELINE & TRIBUTARY CROSSING)

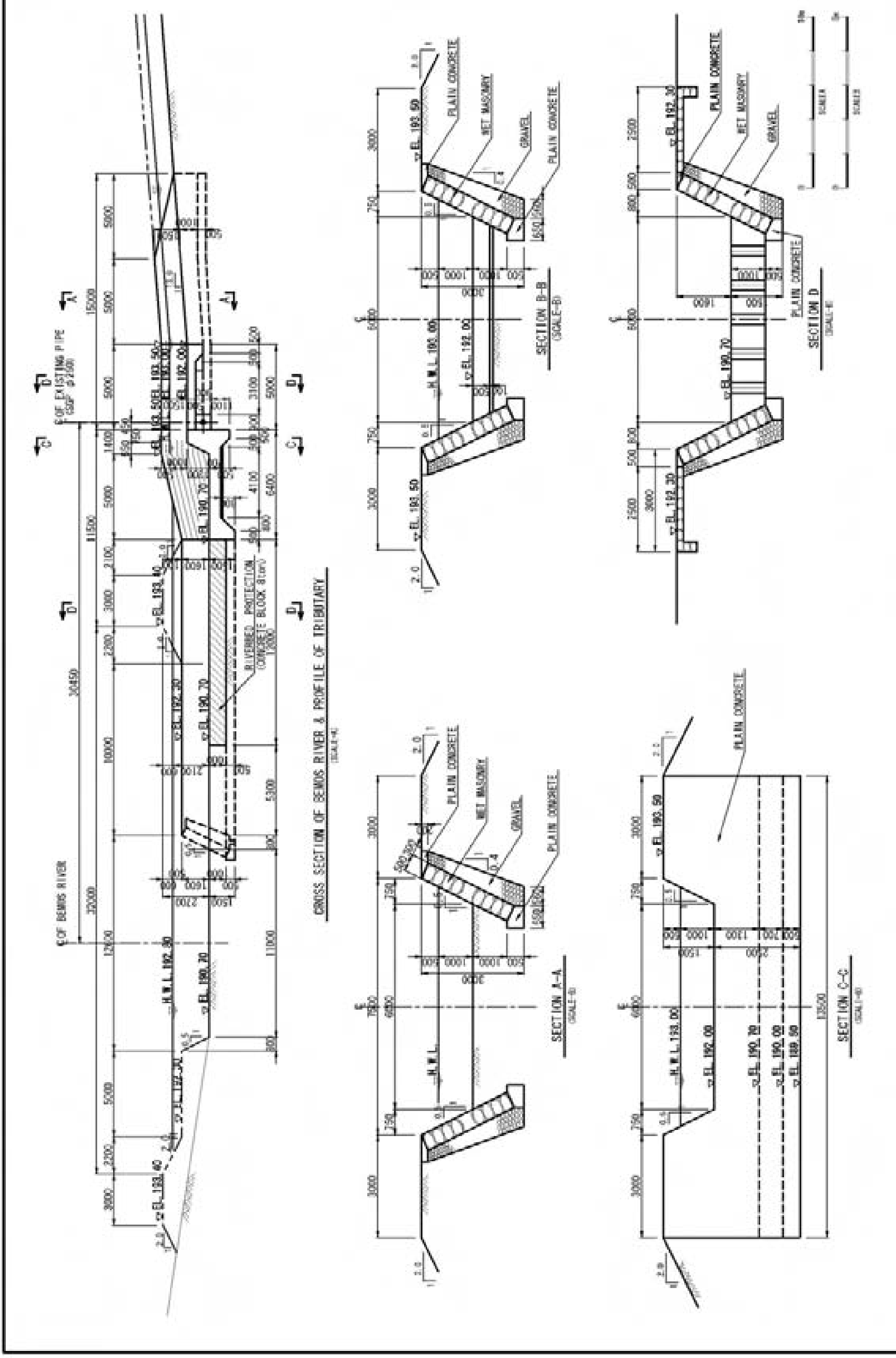


FIG.1-2 ALTERNATIVE FOR STRUCTURE No.5 PLAN OF TRIBUTARY CROSSING (IMPROVEMENT OF EXISTING PIPELINE & TRIBUTARY CROSSING)

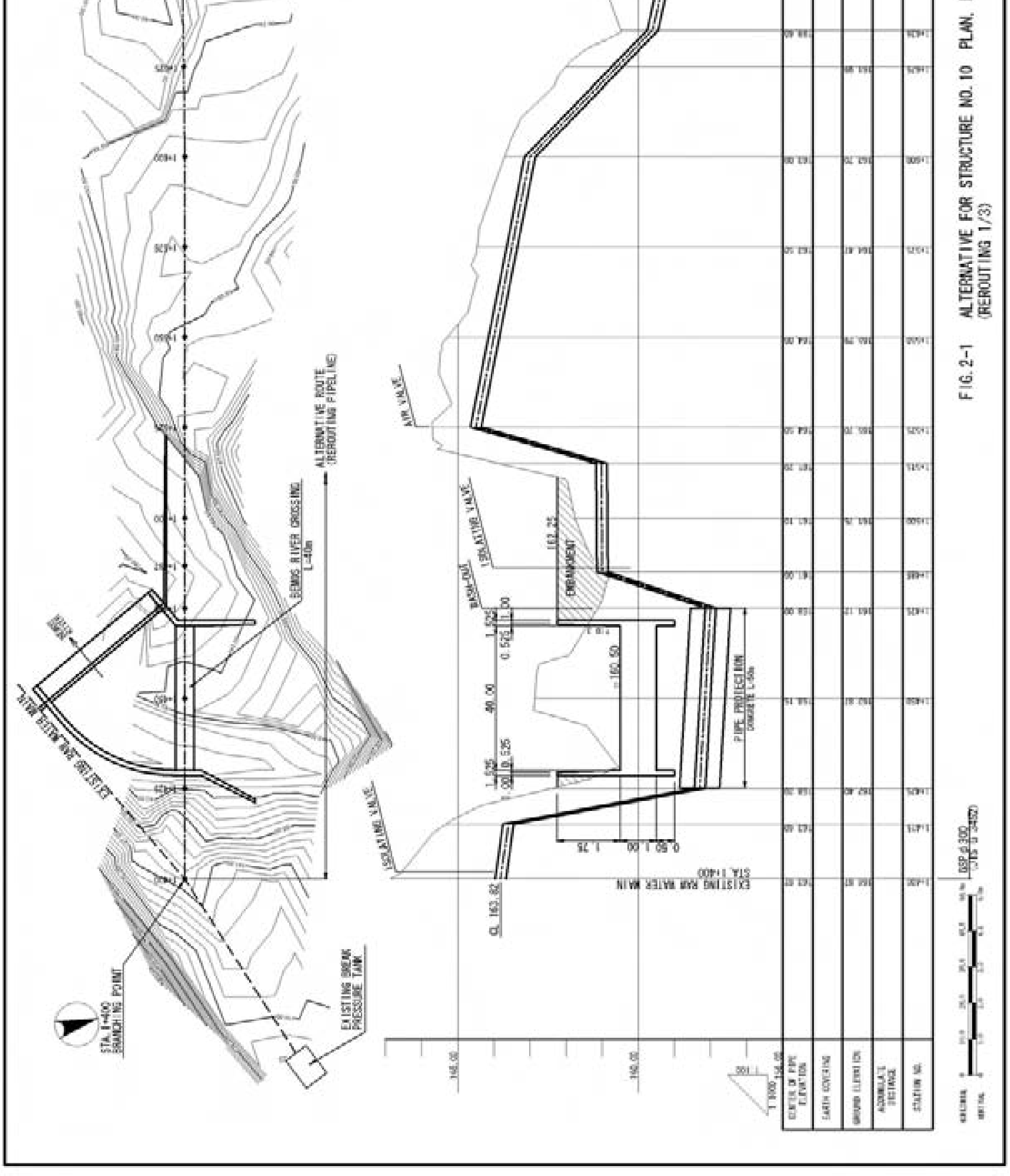


FIG. 2-1 ALTERNATIVE FOR STRUCTURE NO. 10 PLAN, PROFILE & SECTIONS (REROUTING 1/3)

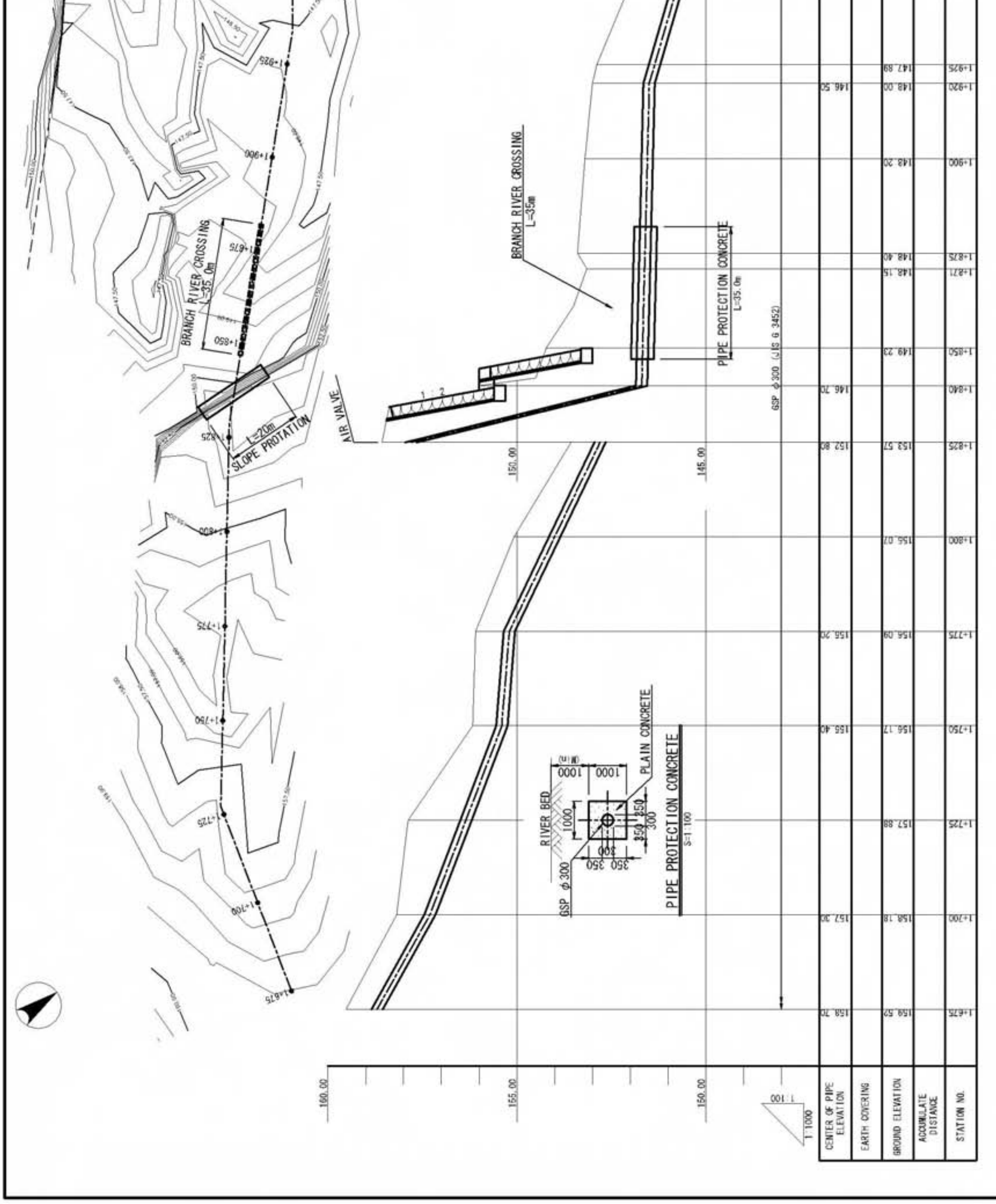


FIG. 2-2 ALTERNATIVE FOR STRUCTURE NO. 10 PLAN, PROFILE & SECTIONS (REROUTING 2/3)

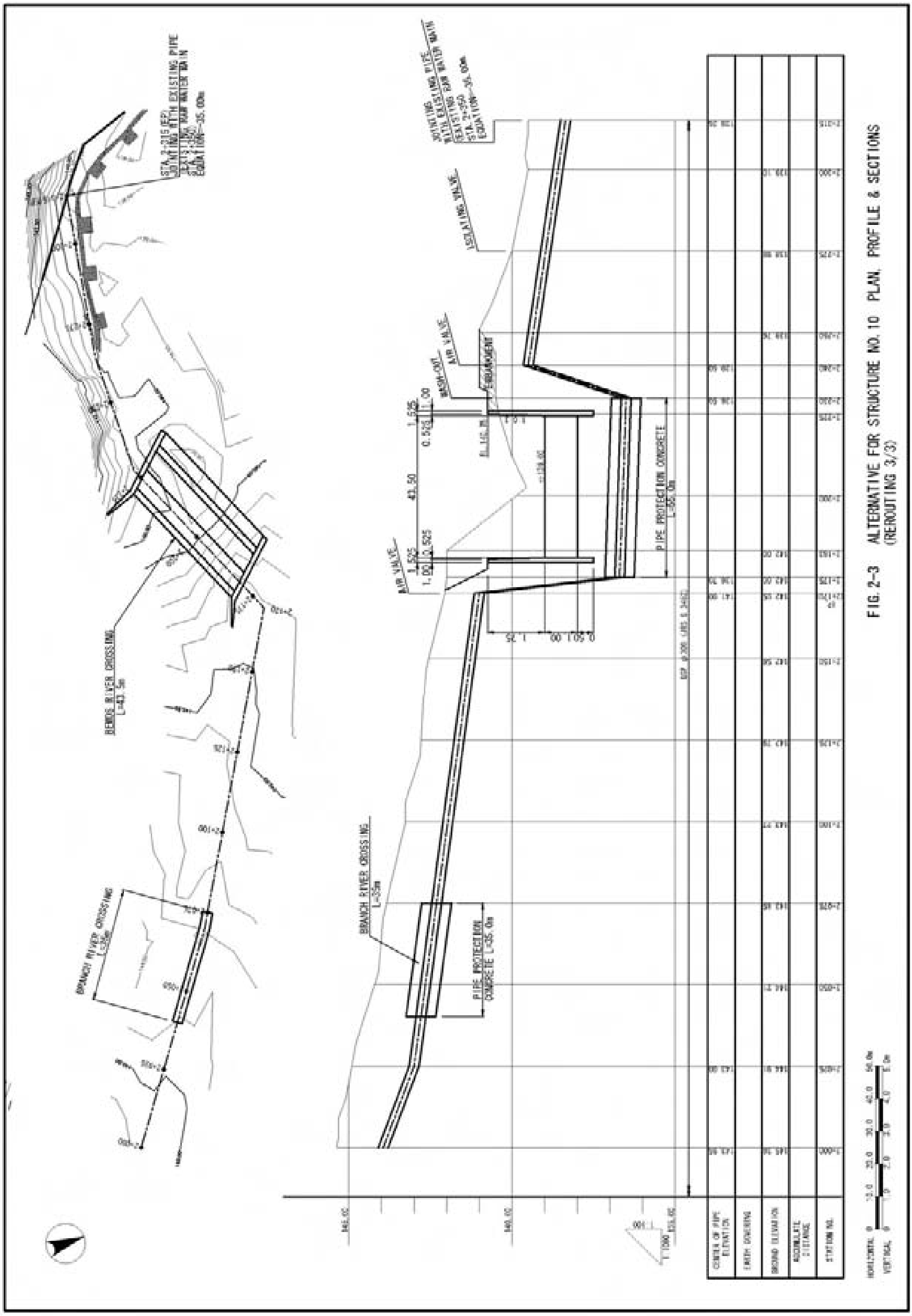
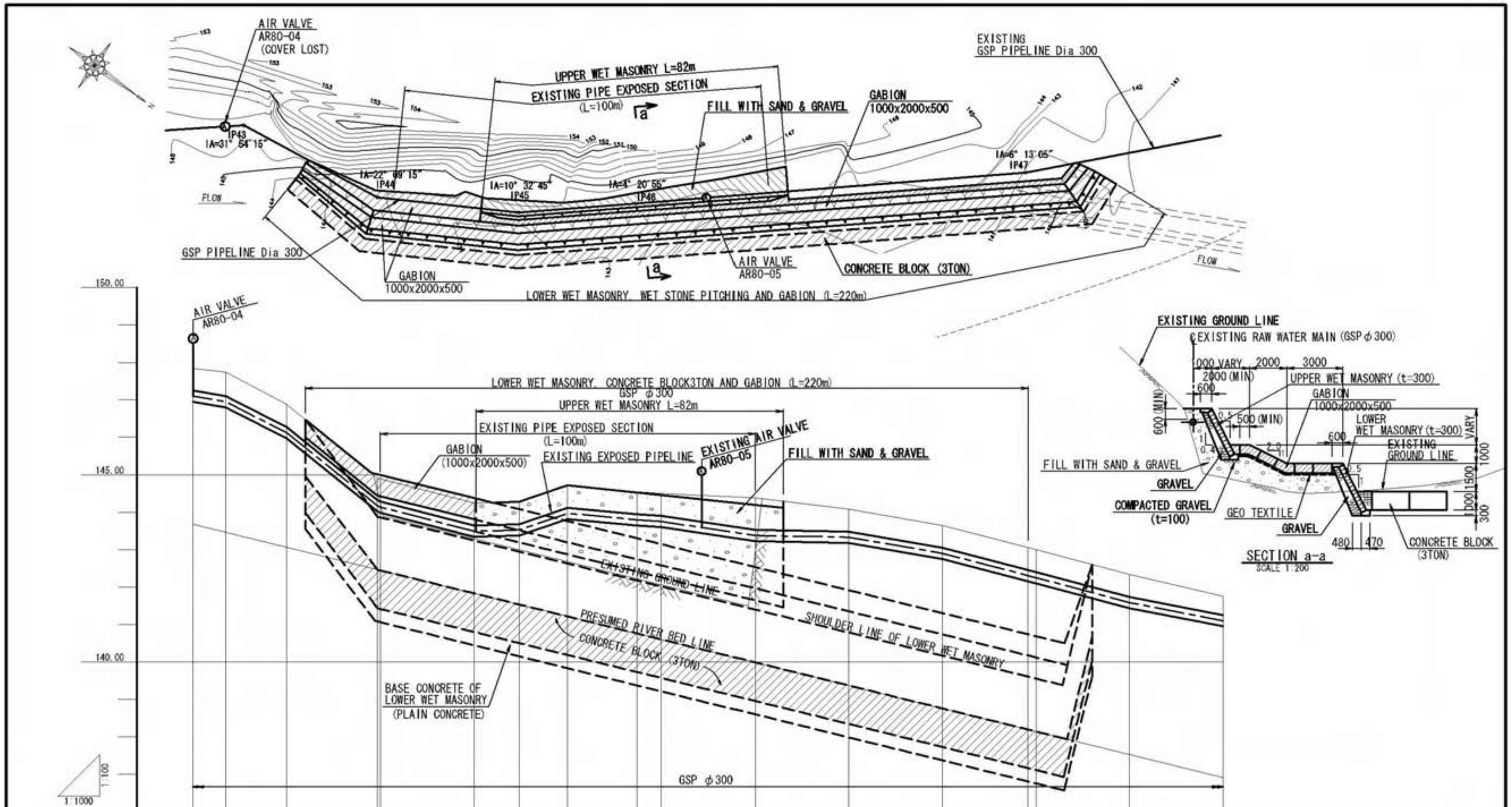


FIG. 2-3 ALTERNATIVE FOR STRUCTURE NO. 10 PLAN, PROFILE & SECTIONS (REROUTING 3/3)



CENTER OF PIPE ELEVATION	147.10	146.95	146.12	144.17	144.14	143.49	143.53	143.97	143.80	143.75	143.63	143.33	142.90	142.31	142.25	141.59	141.01
EARTH COVERING	0.60	0.64	0.61	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60
GROUND ELEVATION	147.85	147.74	146.88	144.92	144.89	144.24	144.28	144.72	144.55	144.50	144.38	144.08	143.55	143.06	143.00	142.34	141.76
ACCUMULATE DISTANCE	1975.00	1983.70	2000.00	2024.27	2025.00	2050.00	2062.14	2075.00	2093.50	2100.00	2125.00	2150.00	2175.00	2197.94	2200.00	2225.00	2250.00
STATION NO.	1+975	IP43	2+000	IP44	2+025	2+050	IP45	2+075	IP46	2+100	2+125	2+150	2+175	IP47	2+200	2+225	2+250

HORIZONTAL 0 10.0 20.0 30.0 40.0 50.0m
VERTICAL 0 1.0 2.0 3.0 4.0 5.0m

FIG. 3 ALTERNATIVE FOR STRUCTURE No. 10-3 PLAN, PROFILE & SECTION
(IMPROVEMENT OF EXISTING PIPELINE BY SLOPE PROTECTION)

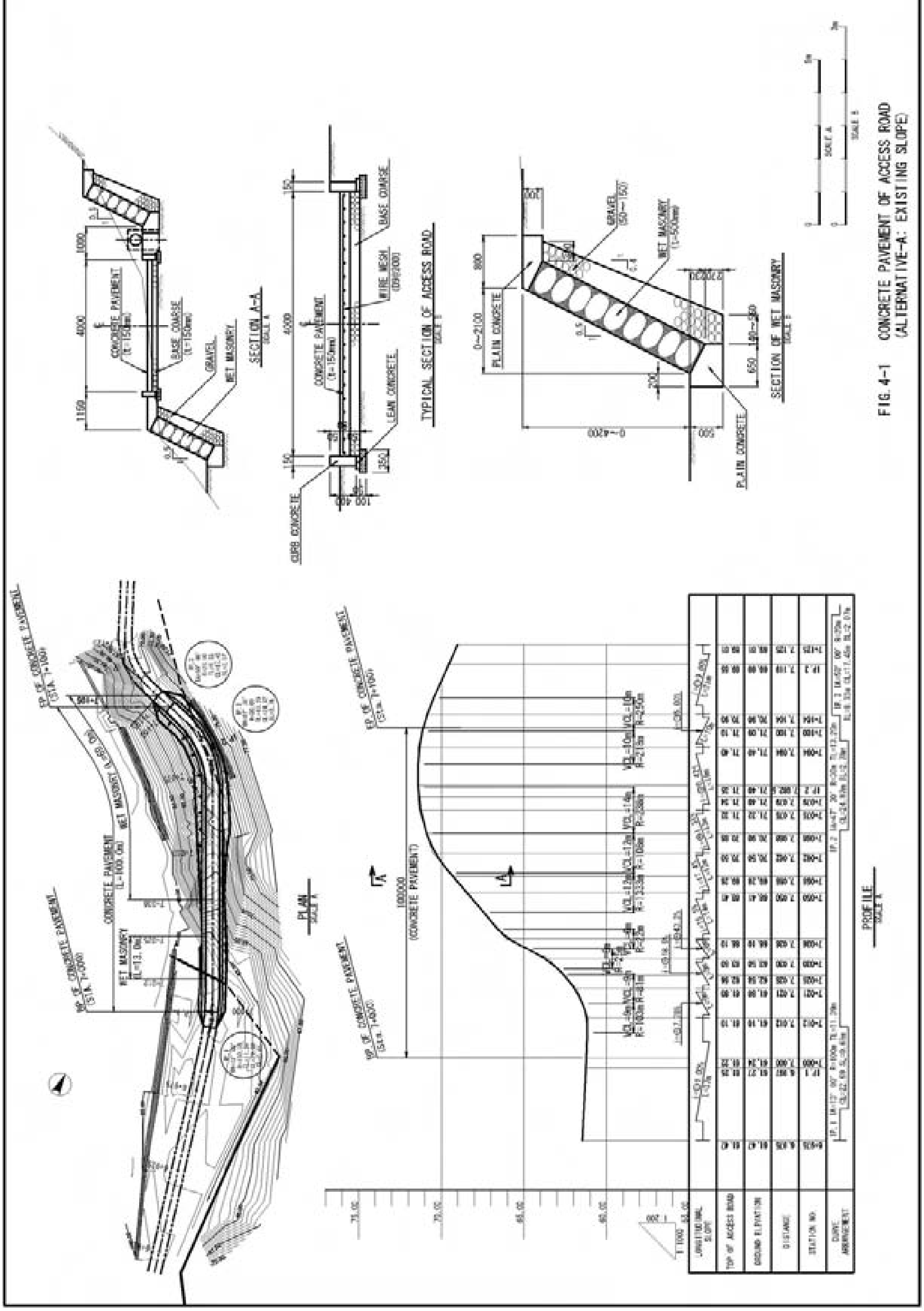


FIG. 4-1 CONCRETE PAVEMENT OF ACCESS ROAD (ALTERNATIVE-A: EXISTING SLOPE)

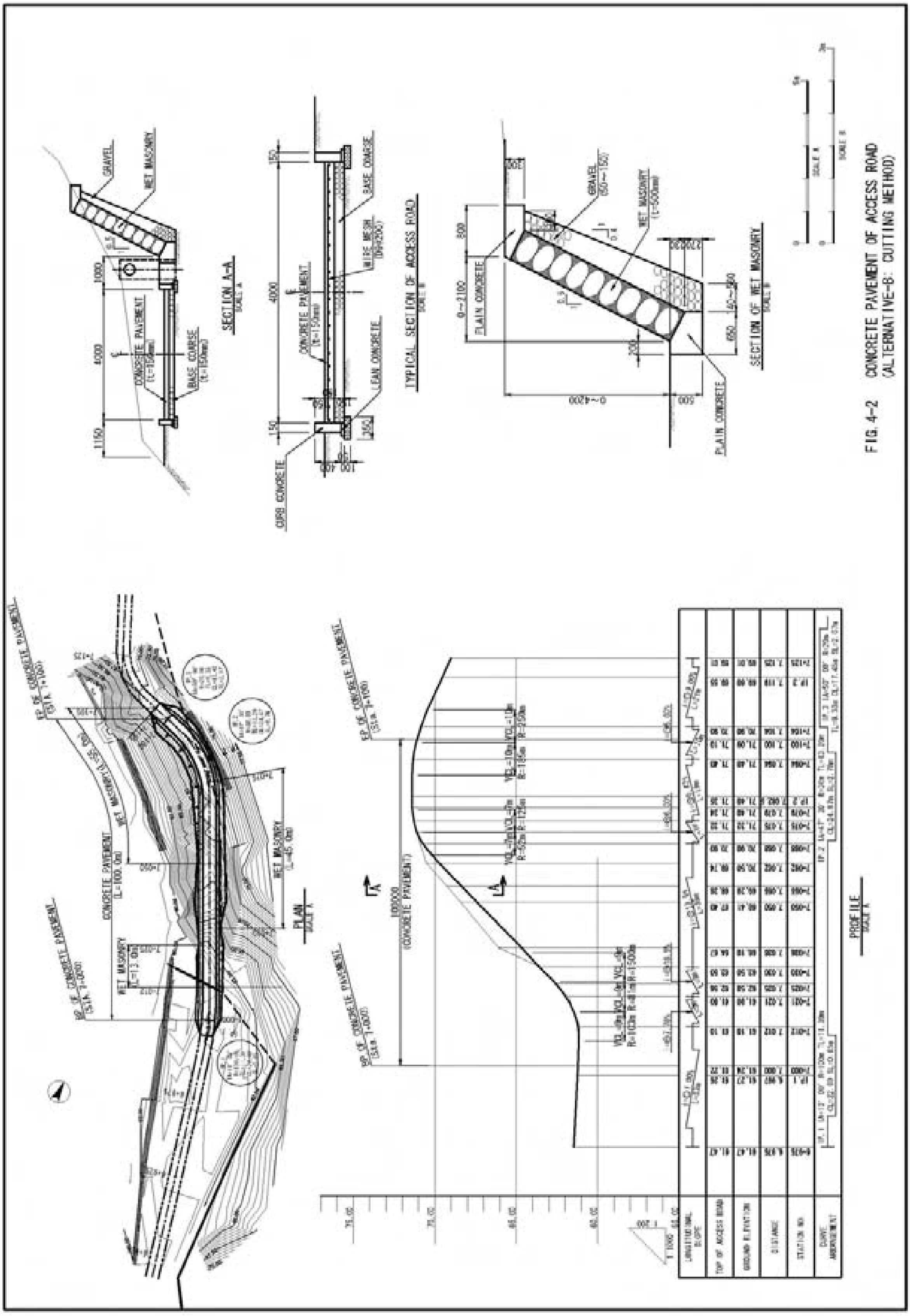


FIG. 4-2 CONCRETE PAVEMENT OF ACCESS ROAD (ALTERNATIVE-B: CUTTING METHOD)

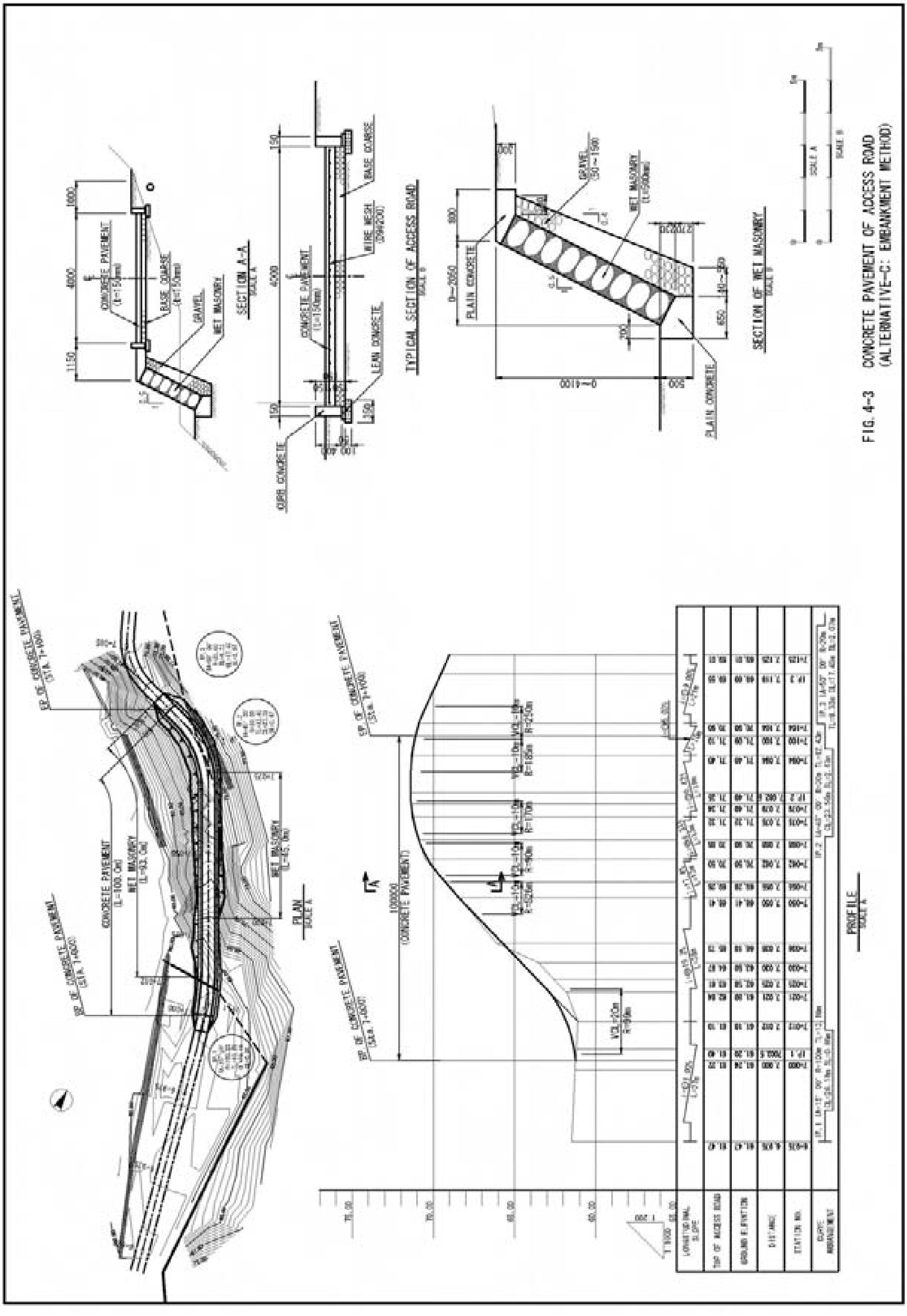
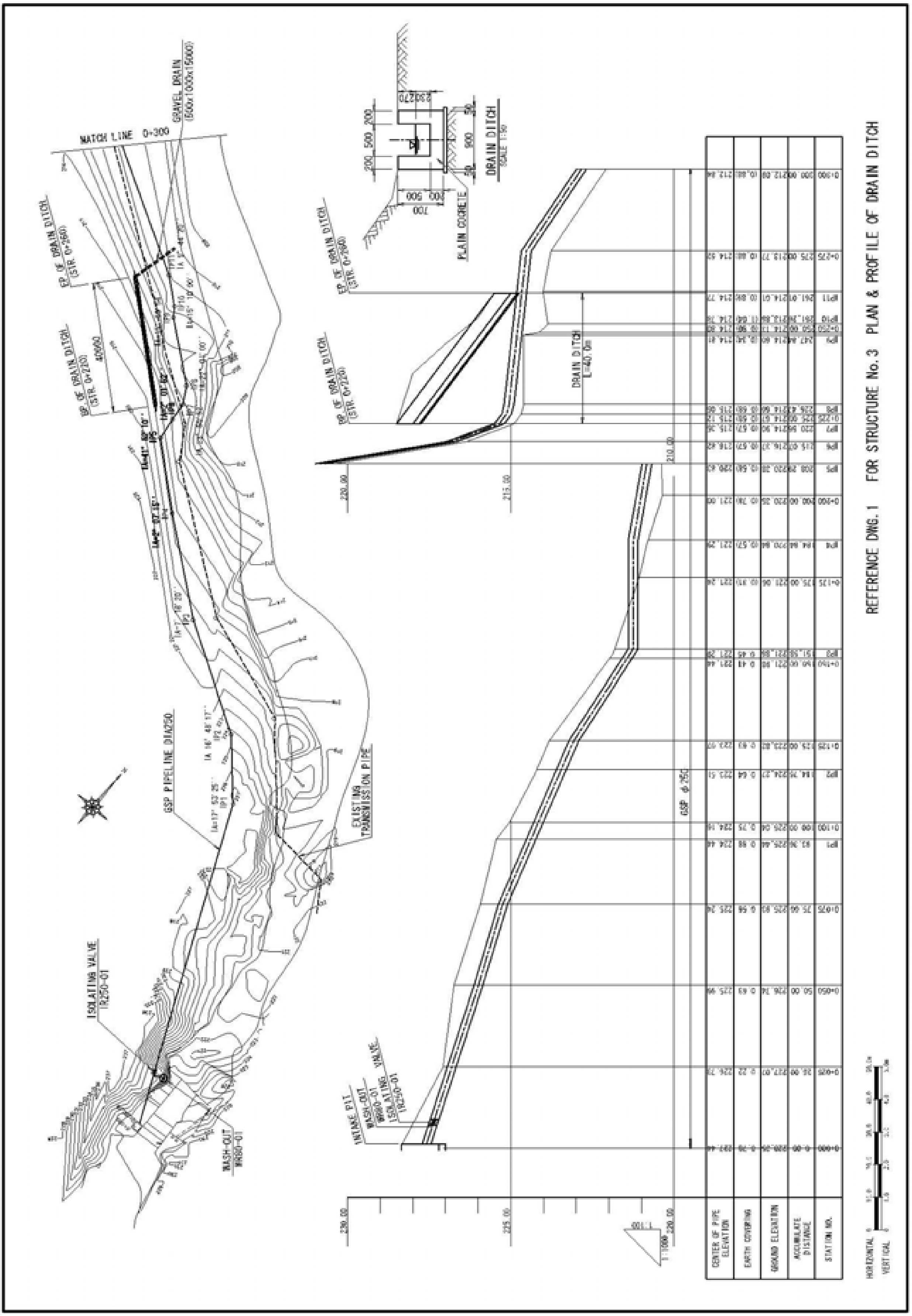


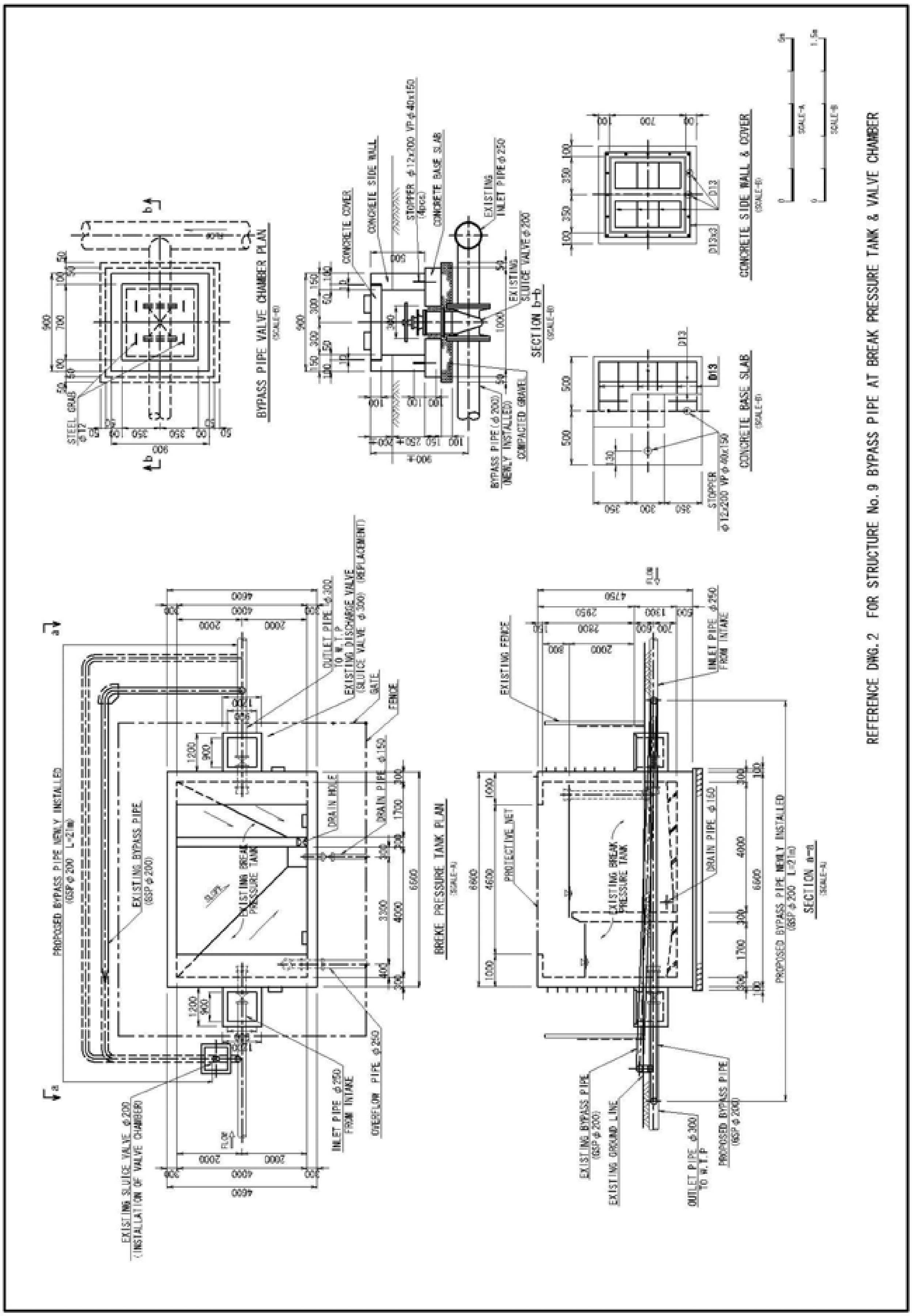
FIG. 4-3 CONCRETE PAVEMENT OF ACCESS ROAD (ALTERNATIVE-C: EMBANKMENT METHOD)

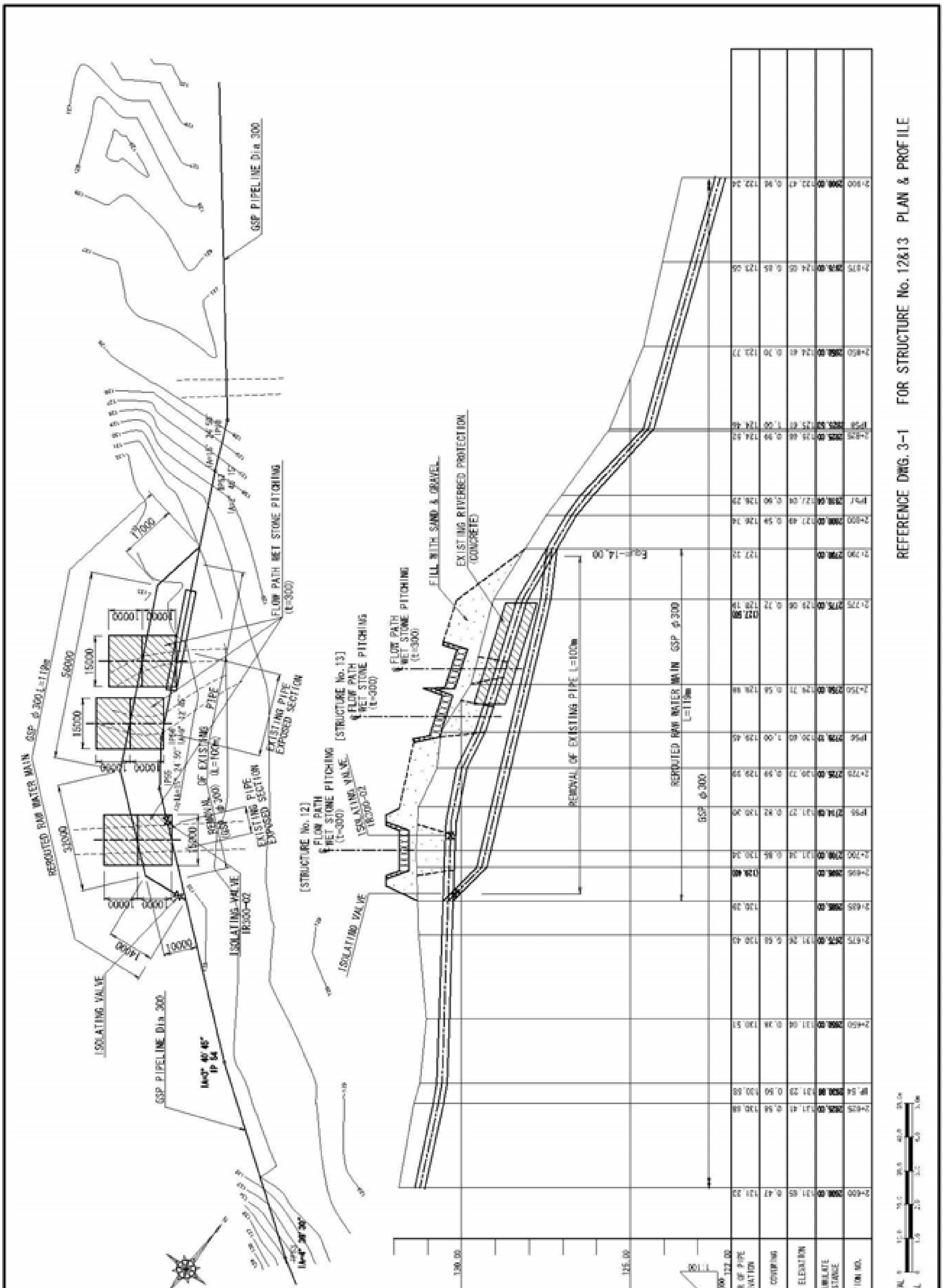
5-7 Reference DWG. 1 to DWG. 5-3

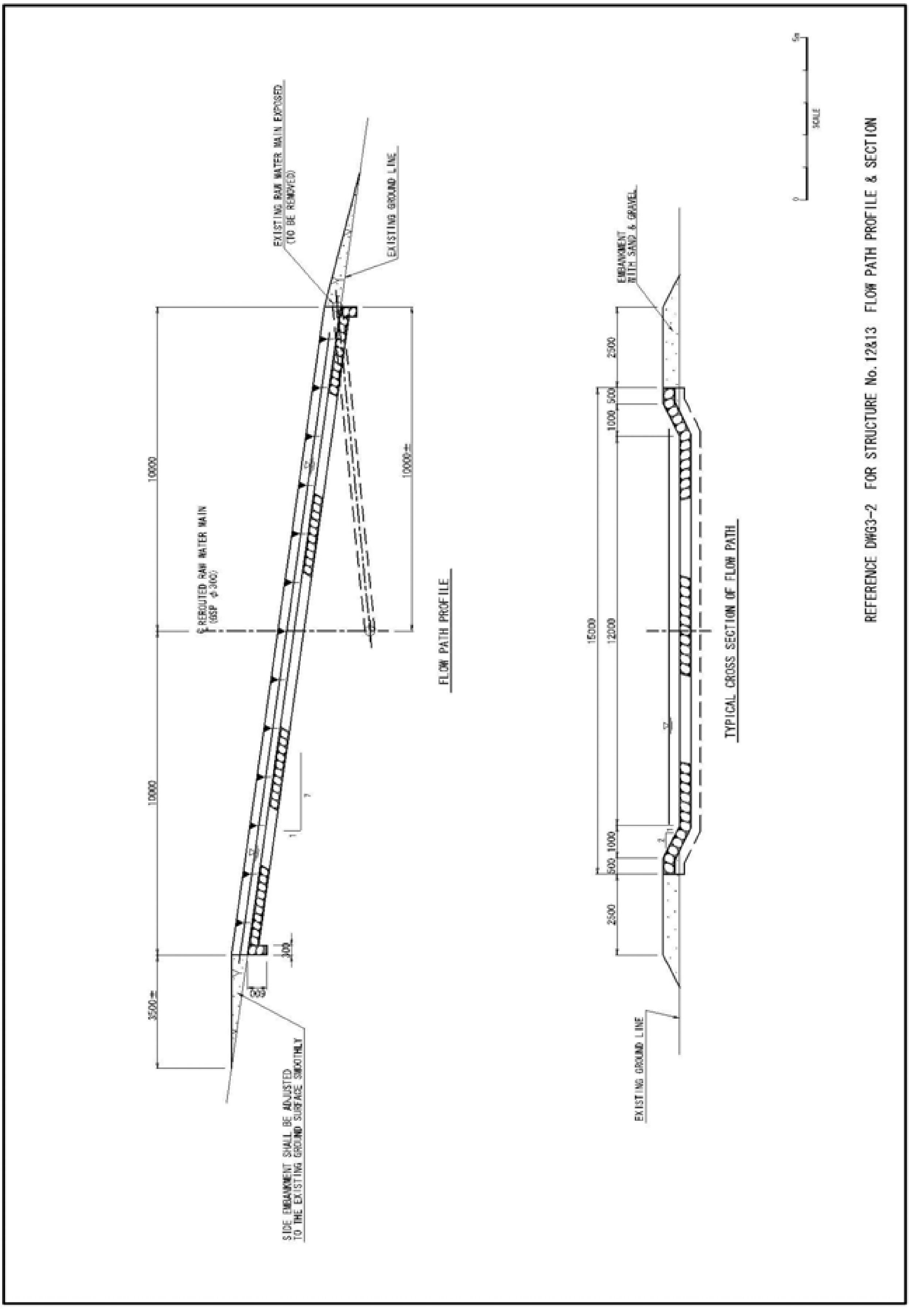
List of Reference DWG.

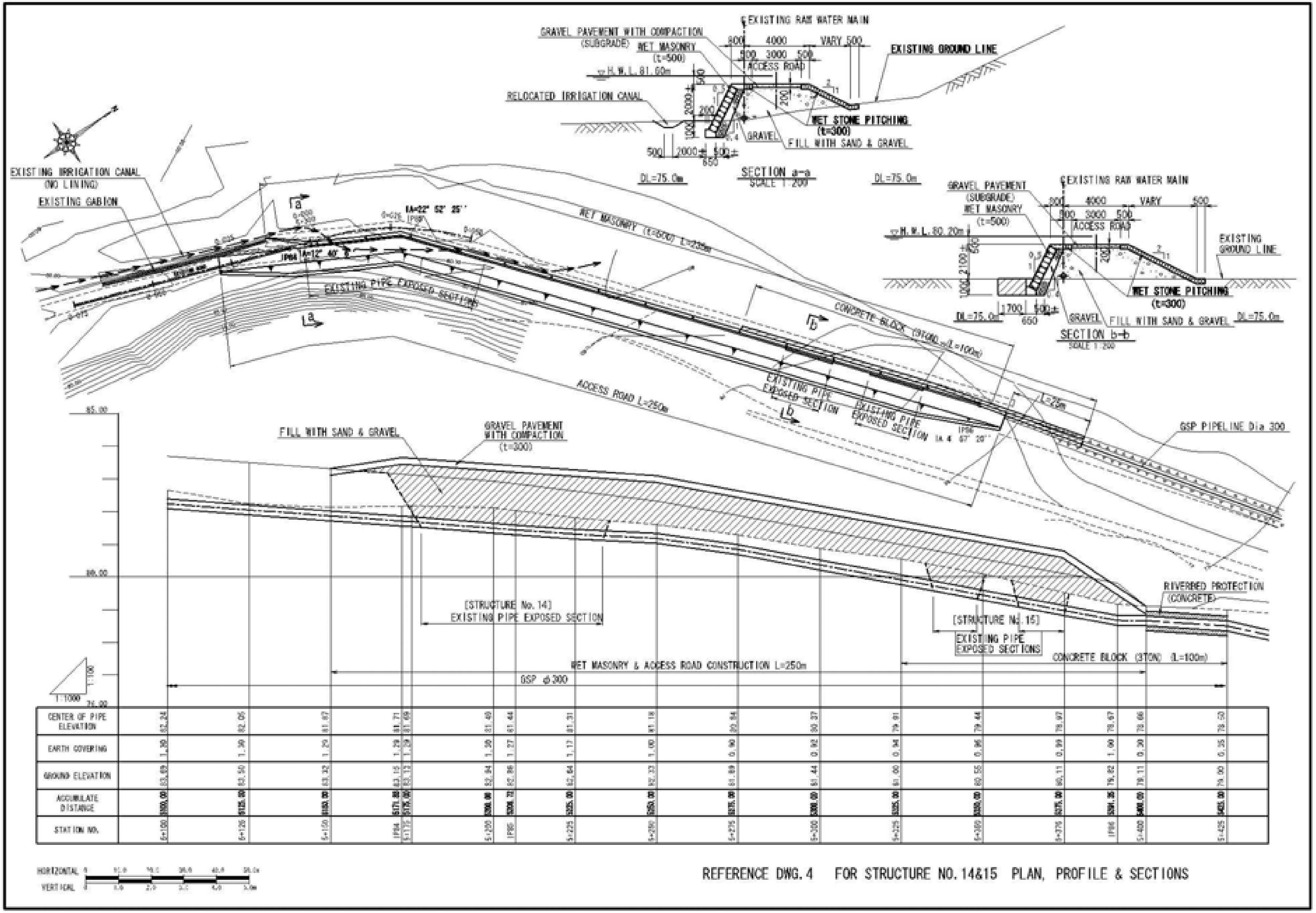
REFERENCE DWG. No.	Title
DWG.. 1	Reference DWG 1 for Structure No.3 Plan & Profile of Drain Ditch
DWG.. 2	Reference DWG 2 for Structure No.9 Bypass Pipe at Break Pressure Tank & Valve Chamber
DWG.. 3-1	Reference DWG 3-1 for Structure No.12 & 13 Plan & Profile
DWG.. 3-2	Reference DWG 3-2 for Structure No.12 & 13 Flow Path Profile & Section
DWG.. 4	Reference DWG 4 for Structure No.14 & 15 Plan, Profile & Sections
DWG.. 5-1	Reference DWG 5-1 for Structure No.16 Plan, Profile & Flow Path of Tributary
DWG.. 5-2	Reference DWG 5-2 for Structure No.16 Plan of Flow Path of Tributary
DWG.. 5-3	Reference DWG 5-3 for Structure No.16 Sections

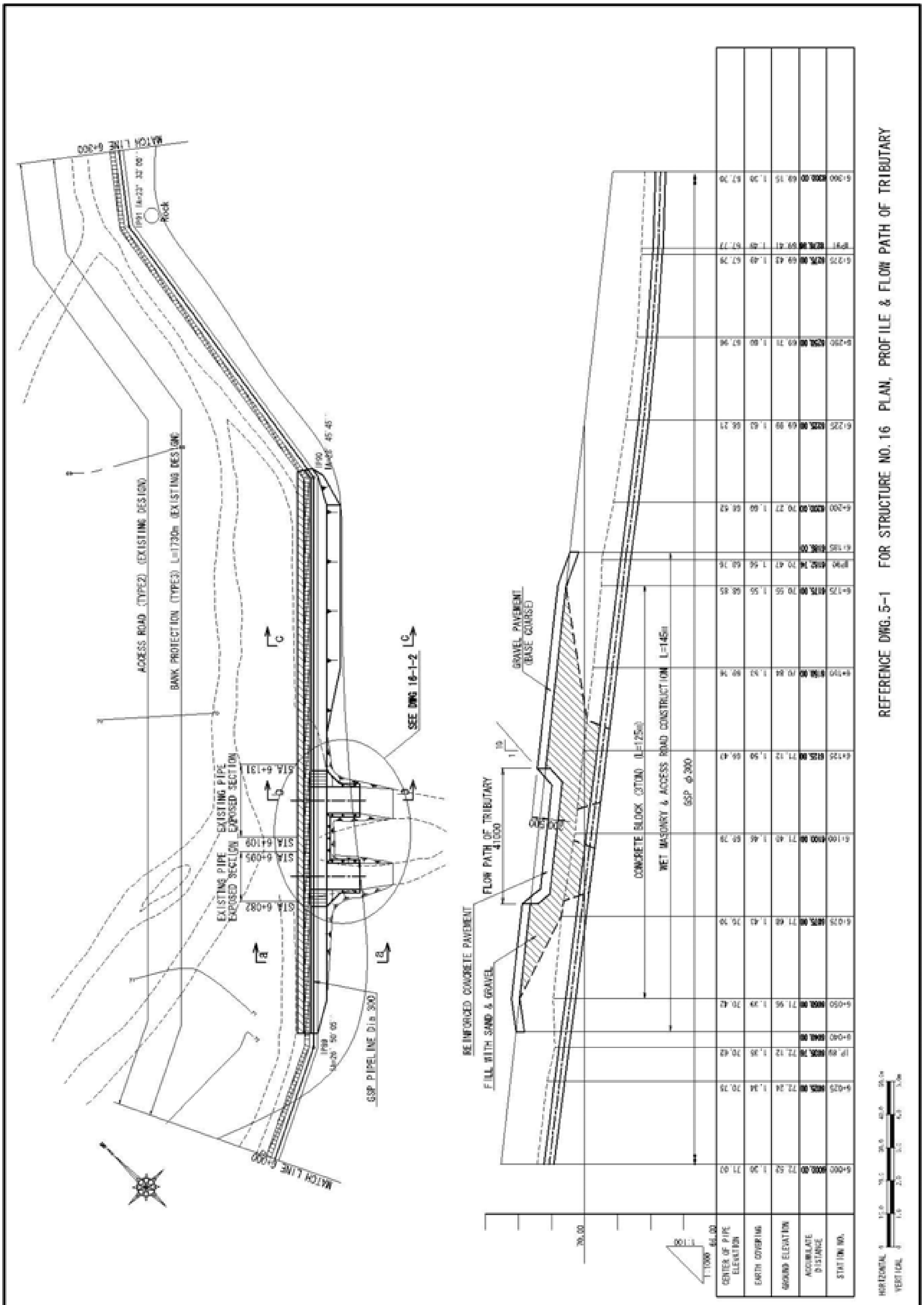

REFERENCE DWG.1 FOR STRUCTURE No. 3 PLAN & PROFILE OF DRAIN DITCH
MATCH LINE 0+300
GRAVEL DRAIN (500x1000x15000)
EP OF DRAIN DITCH (STR. 0+260)
BP OF DRAIN DITCH (STR. 0+220)
GSP PIPELINE DIA250
EXISTING TRANSMISSION PIPE
ISOLATING VALVE IR250-01
WASH-OUT WR80-01
INTAKE PIT
DRAIN DITCH L=40.0m
GSP φ250
DRAIN DITCH
PLAIN CONCRETE
CENTER OF PIPE ELEVATION
EARTH COVERING
GROUND ELEVATION
ACCUMULATE DISTANCE
STATION NO.
HORIZONTAL
VERTICAL


REFERENCE DWG. 2 FOR STRUCTURE No. 9 BYPASS PIPE AT BREAK PRESSURE TANK & VALVE CHAMBER
BREKE PRESSURE TANK PLAN
SECTION a-a
BYPASS PIPE VALVE CHAMBER PLAN
SECTION b-b
CONCRETE BASE SLAB
CONCRETE SIDE WALL & COVER
PROPOSED BYPASS PIPE NEWLY INSTALLED
EXISTING BYPASS PIPE
EXISTING BREAK PRESSURE TANK
OUTLET PIPE φ300 TO W.T.P
EXISTING DISCHARGE VALVE
GATE
FENCE
DRAIN HOLE
DRAIN PIPE φ150
OVERFLOW PIPE φ250
INLET PIPE φ250 FROM INTAKE
EXISTING SLUICE VALVE φ200
EXISTING FENCE
PROTECTIVE NET
EXISTING GROUND LINE
CONCRETE COVER
CONCRETE SIDE WALL
CONCRETE BASE SLAB
COMPACTED GRAVEL
STEEL GRAB

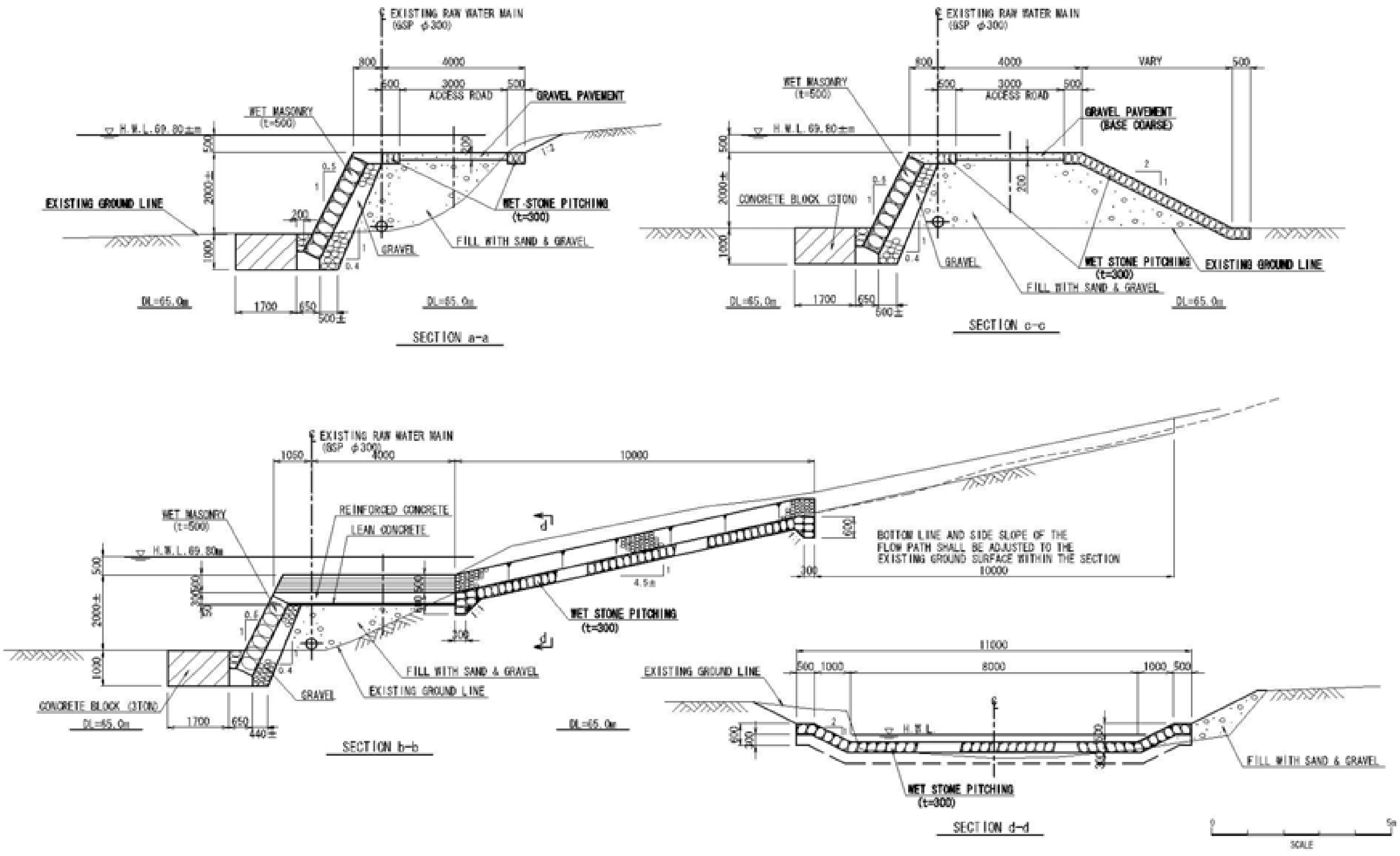

REFERENCE DWG. 3-1 FOR STRUCTURE No. 12&13 PLAN & PROFILE
GSP PIPELINE Dia 300
REROUTED RAW WATER MAIN GSP φ300 L=119m
ISOLATING VALVE
FLOW PATH WET STONE PITCHING (t=300)
EXISTING PIPE EXPOSED SECTION
REMOVAL OF EXISTING PIPE
[STRUCTURE No. 12]
[STRUCTURE No. 13]
FILL WITH SAND & GRAVEL
EXISTING RIVERBED PROTECTION (CONCRETE)
REMOVAL OF EXISTING PIPE L=100m
CENTER OF PIPE ELEVATION
EARTH COVERING
GROUND ELEVATION
ACCUMULATE DISTANCE
STATION NO.
HORIZONTAL
VERTICAL


EXISTING RAW WATER MAIN EXPOSED
(TO BE REMOVED)
EXISTING GROUND LINE
REROUTED RAW WATER MAIN
FLOW PATH PROFILE
SIDE EMBANKMENT SHALL BE ADJUSTED
TO THE EXISTING GROUND SURFACE SMOOTHLY
EMBANKMENT
WITH SAND & GRAVEL
TYPICAL CROSS SECTION OF FLOW PATH
EXISTING GROUND LINE
SCALE
REFERENCE DWG3-2 FOR STRUCTURE No.12&13 FLOW PATH PROFILE & SECTION


EXISTING RAW WATER MAIN
GRAVEL PAVEMENT WITH COMPACTION (SUBGRADE)
WET MASONRY (t=500)
H.W.L.81.60m
ACCESS ROAD
EXISTING GROUND LINE
RELOCATED IRRIGATION CANAL
WET STONE PITCHING (t=300)
GRAVEL
FILL WITH SAND & GRAVEL
DL=75.0m
SECTION a-a
SCALE 1:200
GRAVEL PAVEMENT (SUBGRADE)
WET MASONRY (t=500)
H.W.L.80.20m
SECTION b-b
SCALE 1:200
EXISTING IRRIGATION CANAL (NO LINING)
EXISTING GABION
EXISTING PIPE EXPOSED SECTION
WET MASONRY (t=500) L=235m
CONCRETE BLOCK (3TON) L=100m
ACCESS ROAD L=250m
GSP PIPELINE Dia 300
FILL WITH SAND & GRAVEL
GRAVEL PAVEMENT WITH COMPACTION (t=300)
[STRUCTURE No.14] EXISTING PIPE EXPOSED SECTION
[STRUCTURE No.15] EXISTING PIPE EXPOSED SECTIONS
RIVERBED PROTECTION (CONCRETE)
WET MASONRY & ACCESS ROAD CONSTRUCTION L=250m
CONCRETE BLOCK (3TON) (L=100m)
GSP φ300
CENTER OF PIPE ELEVATION
EARTH COVERING
GROUND ELEVATION
ACCUMULATE DISTANCE
STATION NO.
HORIZONTAL
VERTICAL
REFERENCE DWG.4 FOR STRUCTURE NO.14&15 PLAN, PROFILE & SECTIONS



REFERENCE DWG.5-1 FOR STRUCTURE NO.16 PLAN, PROFILE & FLOW PATH OF TRIBUTARY



REFERENCE DWG.5-3 FOR STRUCTURE No.16 SECTIONS

5-8 Reference Drawing of Structure No. 10-3:Temporary Pipe Support