10.2.3 Landslide－Wright-Taft, Km 846 (Region VIII)

(1) Background of the Existing Countermeasure

The road section at Wright-Taft Road, Km 846 was damaged for a length of 80 m by a landslide which occurred in the lower embankment slope. Based on the boring investigation and site damage assessment, the following factors were assumed to cause the said damage:

- The lower slope of the road, which has a gentle gradient, was assumed to have been constructed of the surplus soil from the road construction. The road slip seems to have occurred at the border between the fill layer and the previously exiting ground surface, which may suggest that surface treatment of the embankment was not sufficient.
- A 50 m long gabion wall 7.5 m high was constructed on the slope approximately 50 m from road centerline, as shown in Figure 10.25. The said wall is assumed to have been constructed as a retaining structure, however, the fill material behind the wall has washed out, and the wall has partially collapsed.

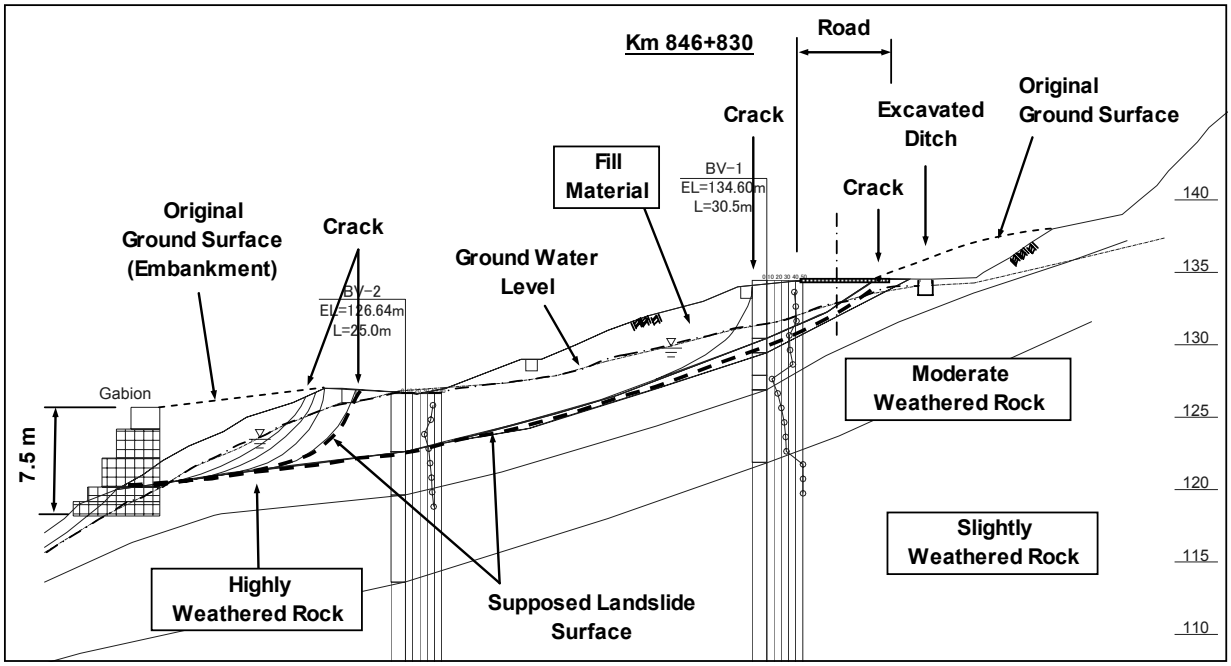


Figure 10.25 Engineering Geological Profile at Wright-Taft Road Km 846

- The existing earth side ditches were not able to drain the large amount of surface water at the said section. Further, the said ditches were already clogged.
- The slope embankment material may be saturated with groundwater. Actually, confined ground-water was observed at the boring survey point on the lower slope.

From the above described site situation, the embankment material at the lower slope may have acted as a landslide block, and this landslide caused road pavement damage. As indicated in Figure 10.25, the damaged section has two landslide blocks; 1) the entire block of the embankment body that affected the road surface, and ii) a small sized block behind the existing gabion wall that caused its collapse.

(2) Structure to be Applied and Design Procedure for Countermeasure

Features of the countermeasure design include a retaining wall structure, horizontal drain holes, and surface drain system. Plan and typical cross section of the countermeasure are shown in Figure 10.26 and Figure 10.27, respectively. Major features of the countermeasure are as follows;

(a) Horizontal Drain Holes

Number of Drain Holes : 4 locations

Elevations of Lateral Holes : +129.8 m (upper drains), +123.0 m (lower drains)

Length of Drain Holes : 25 m x 5 holes (upper drains), 35 m x 5 holes (lower drains)

(b) Retaining Wall Structure

Wall Type : Reinforced Concrete Cantilever Wall

Height of Wall : H = 7.5 m

Total Length : L = 72 m

Foundation Type : Spread Foundation (Embedment Depth = 1.5 m)

(c) Surface Drain System

Concrete U-Ditches : L = 209 m (size 500 x 500), L = 204 m (size 1,000 x 1,000)

Pipe Culverts for Cross Drains : φ1,200 x 11 m x 2 locations

Figure 10.28 shows the design procedure for the above countermeasure. The large landslide block is to be taken care of by horizontal drain holes, and the small block by the retaining wall structure. Hereafter, the method for design analysis of the horizontal drain holes and reinforced concrete cantilever wall to protect against landslide force is undertaken taking into consideration the effect of external forces.

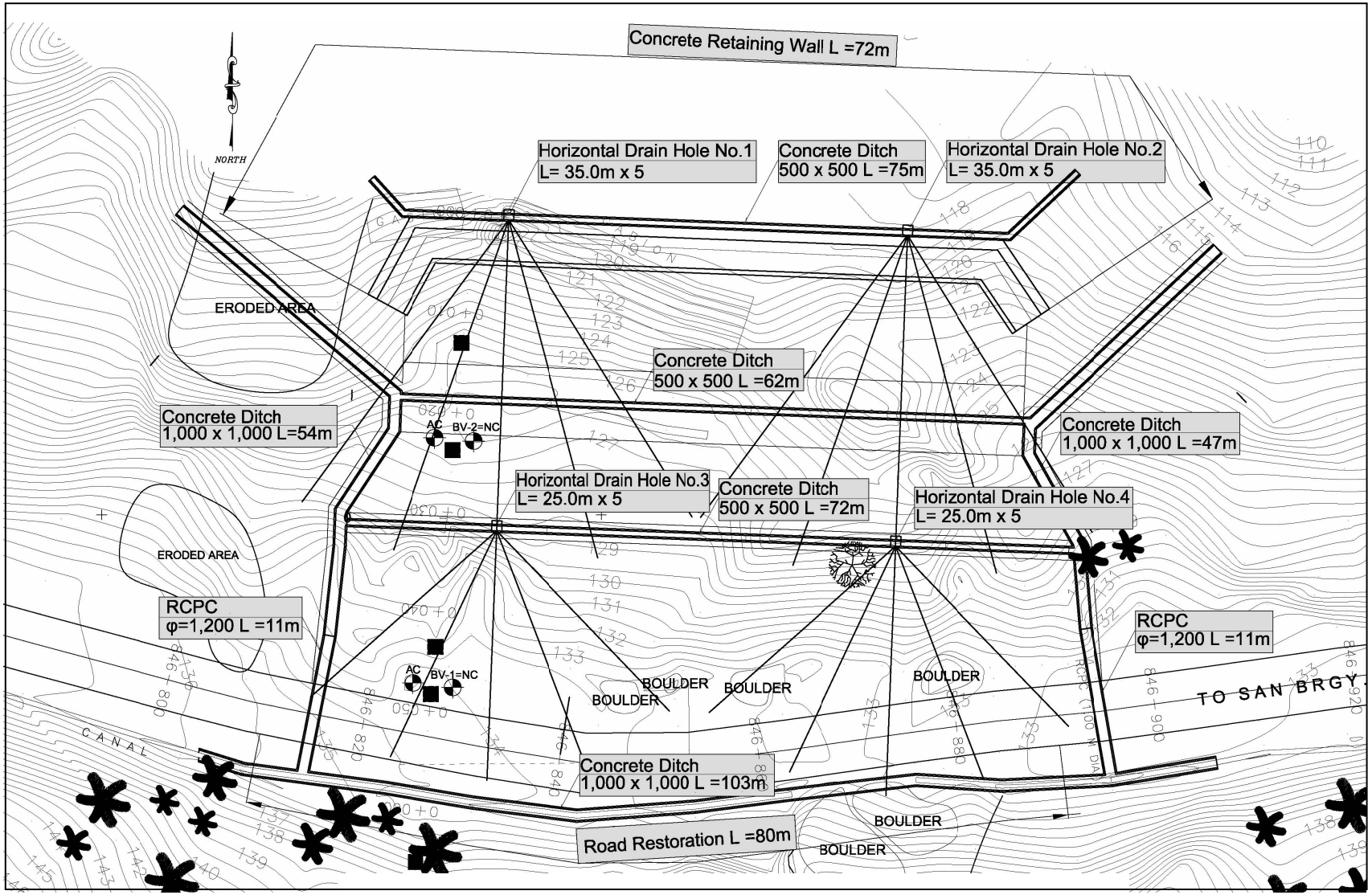


Figure 10.26 Countermeasure Plan for Wright-Taft Road Km 846

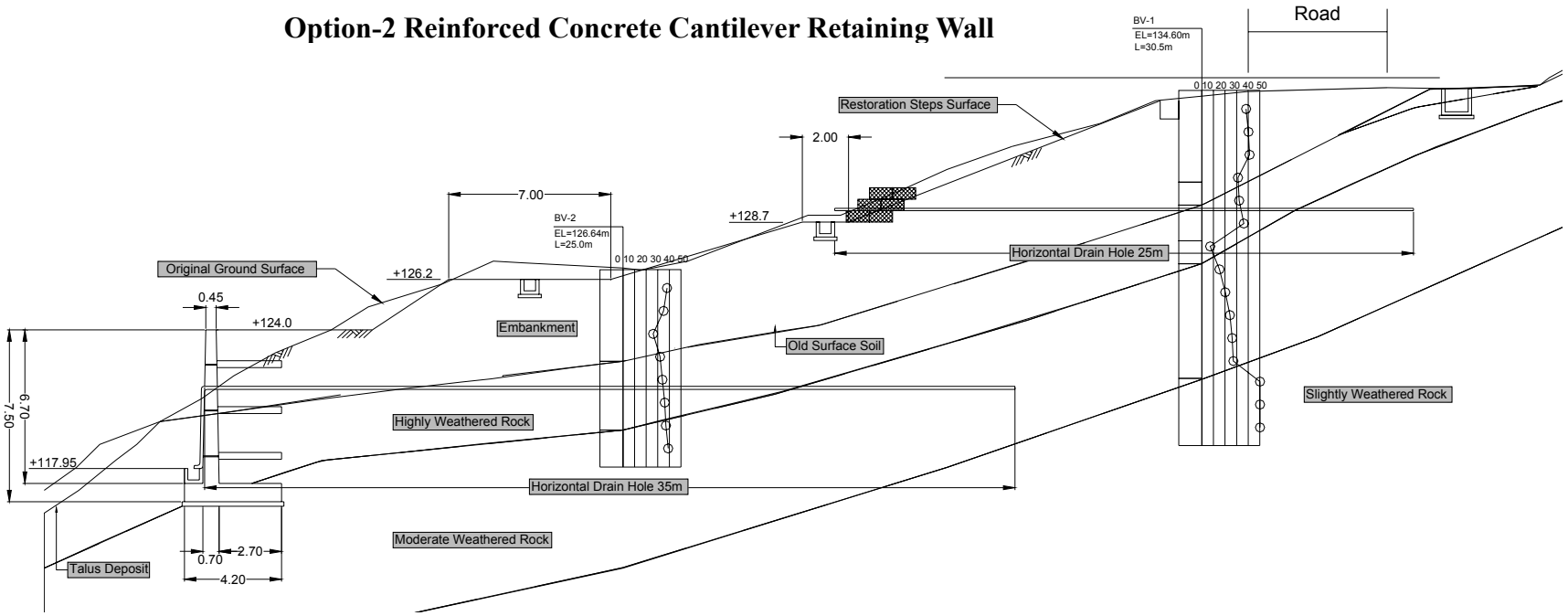
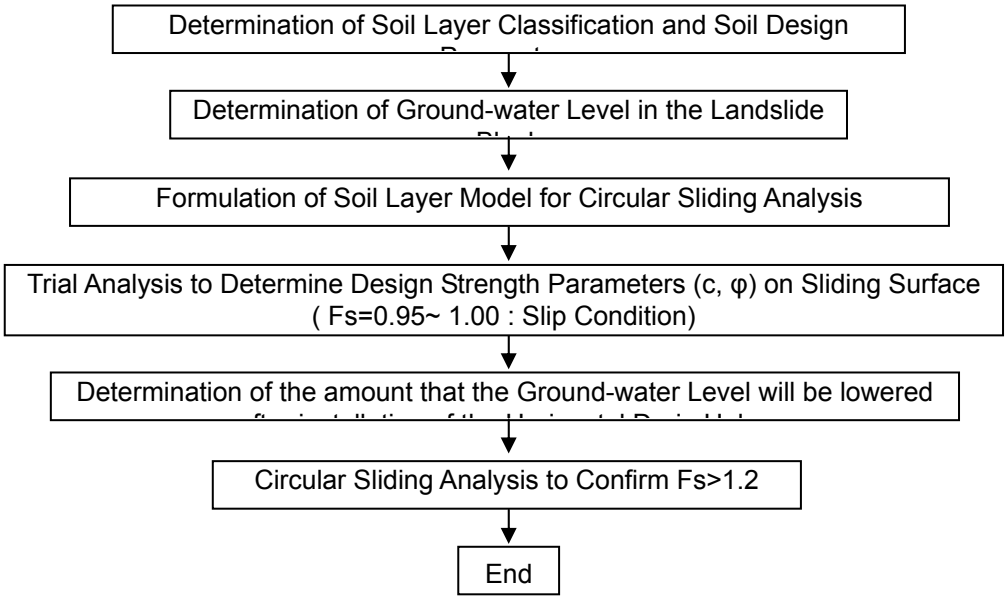
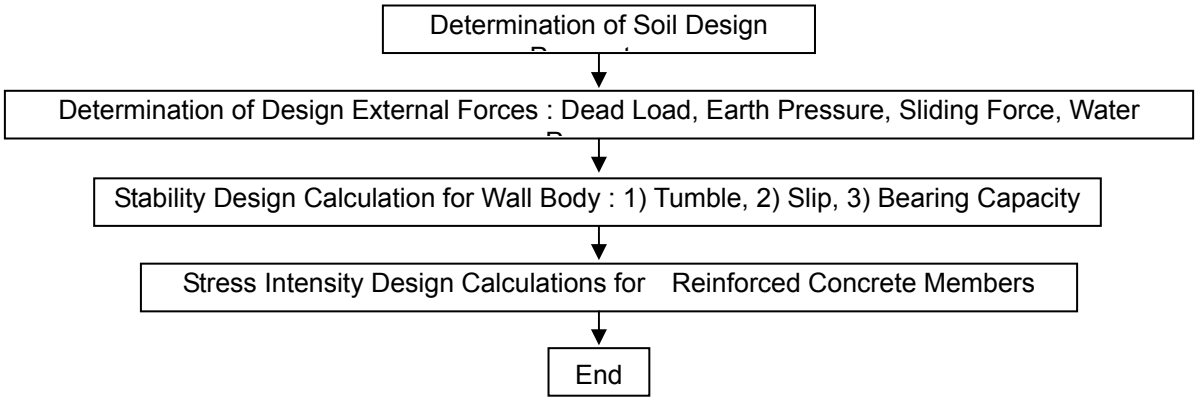


Figure 10.27 Typical Cross Section for Drainage System + Reinforced Concrete Cantilever Wall

[Horizontal Drain Holes]



[Reinforced Concrete Cantilever Retaining Wall]



[Surface Drain System]

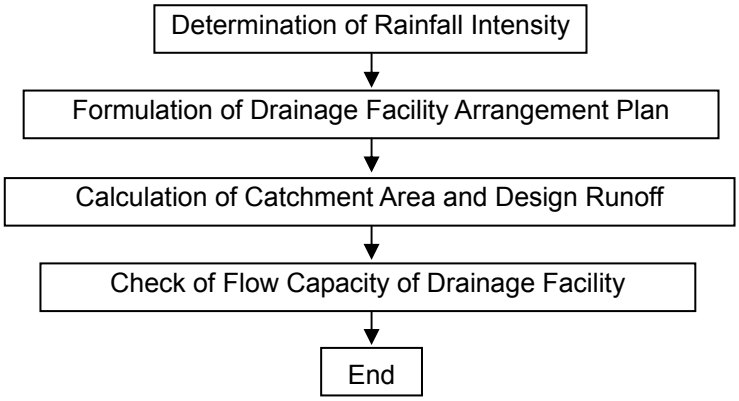


Figure 10.28 Design Procedure for Countermeasure for Wright-Taft, Km 846

(2) Circular Sliding Analysis for Landslide Horizontal Drainage Countermeasure

(a) Soil Layer Modeling Formulation for Circular Sliding Analysis

Circular sliding analysis requires modeling of soil layers and determination of the ground-water line. The analysis ground model presented in Figure 10.29 was formulated based on the results of boring investigations. Also, the soil parameters required for input in the sliding analysis were established from said investigation, as shown in Table 10.9.

Table 10.9 Initial Soil Parameters for Circular Sliding Analysis

Soil Layer	Unit Weight (kN/m^3)		Shear Strength	
	Wet	Saturated	c (kN/m^2)	Φ (deg)
1. Slightly Weathered Rock	22	22	> 500	-
2. Moderate Weathered Rock	21	21	> 500	-
3. Highly Weathered Rock	20	20	350	-
4. Original Surface Soil (Landslide Surface)	14	15	190	-
5. Embankment Material	18	19	210	-

(b) Setting of Design Soil Parameters for Sliding Analysis

The landslide surface is generated at the original ground surface, which is the oblique line in Figure 10.29. However, the safety factor for analysis using the soil parameters shown in Table 10.9 results in considerably more than Fs=1.0.

As a result of the trial analysis, the following design parameters for the landslide surface (original ground surface layer) were determined in order to examine the effectiveness of the proposed countermeasure.

[Design Strength Parameters at the Landslide Surface]

c = 5 kN/m2, Φ = 10 (deg.) : Fs_{min} = 0.98 in upper figure of Figure 10.29

(c) Circular Sliding Analysis of the Horizontal Drain Hole Countermeasure

The horizontal drain holes were arranged in double steps at elevation +129.8 m for the upper drains and at elevation +123.0 m for the lower drains, in a total of four locations. By this measure, the ground-water may be assumed to be lowered to the elevation of the horizontal drains. With the lowering of the ground-water due to the horizontal drains, the safety factor may increase to Fs=1.21 from Fs=0.98 as indicated by the stability analysis as presented in Figure 10.29.

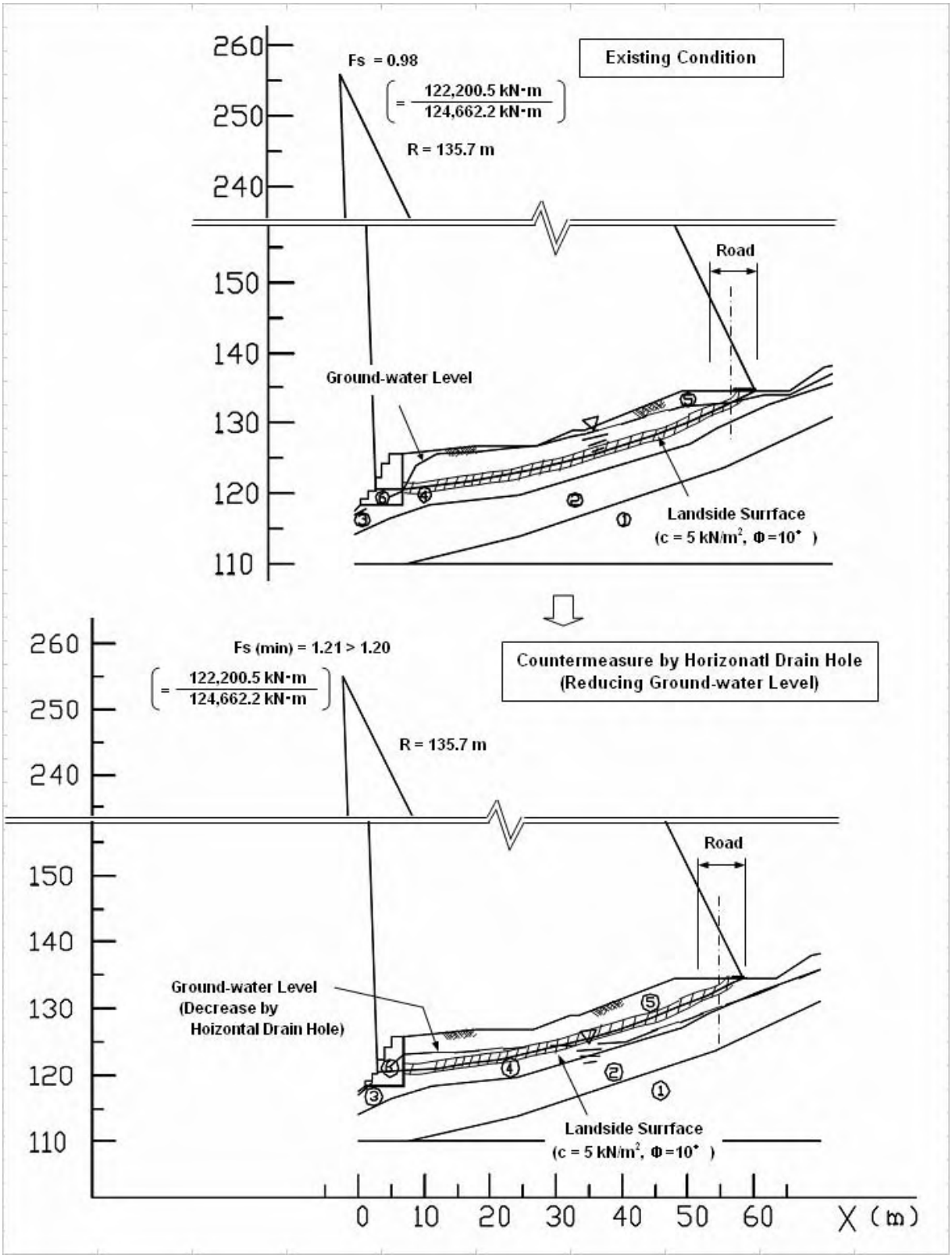


Figure 10.29 Design Shear Strength Parameters at the Landslide Surface by Circular Sliding Analysis and the Effect of lowering the Ground-water

(3) Design of Reinforced Concrete Cantilever Retaining Wall

(a) External Forces

The following external forces were considered in the stability design of the retaining wall;

- Resisting Force =i) Dead Load (W), ii) Positive Earth Pressure (P_P)
- Active Force = i) Sliding Force (P_S), ii) Ground-water Pressure (P_W)

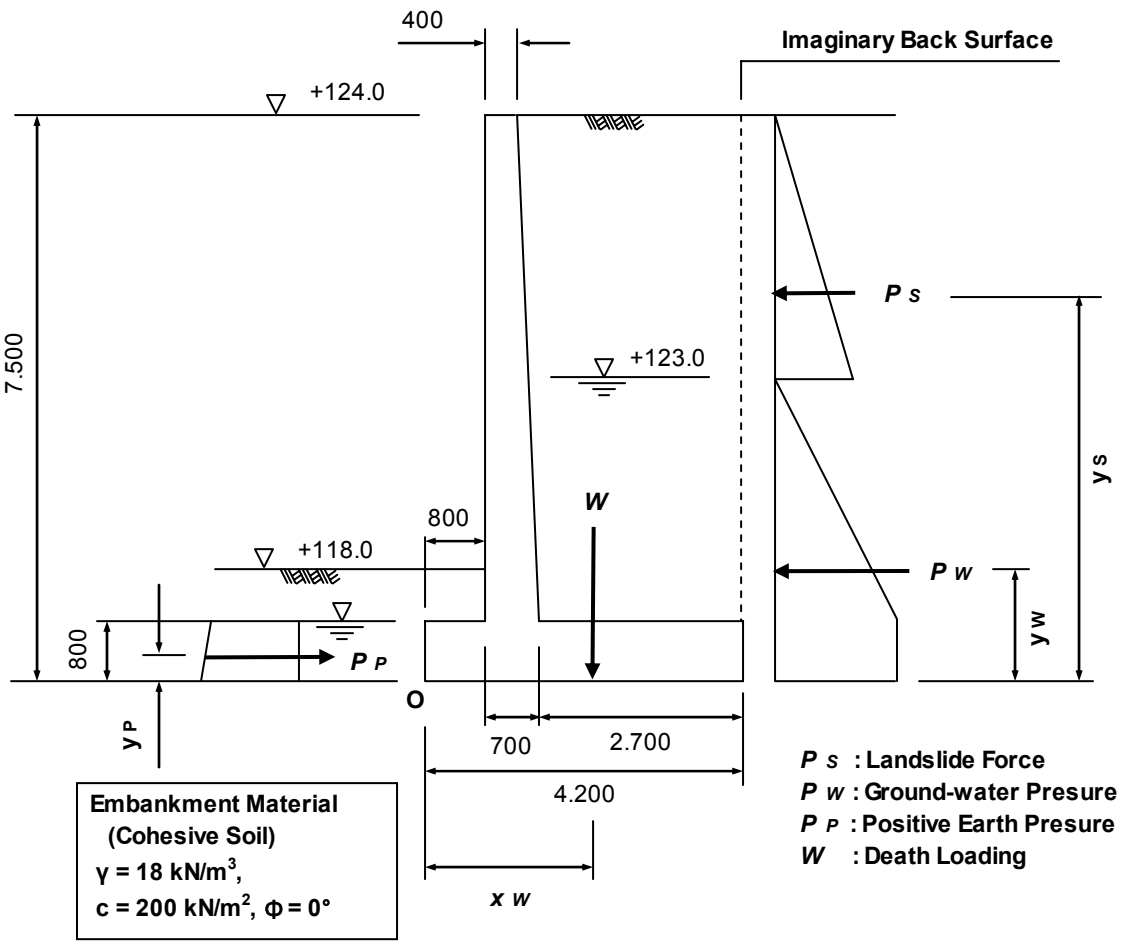


Figure 10.30 External Forces for Stability Calculation of Cantilever Retaining Wall

[Dead Load: W]

In addition to the weight of concrete wall, the soil weight within the imaginary back surface as presented in Figure 10.30 is also included in the resisting force by the following unit weight;

- Unit Weight of Reinforced Concrete : $\gamma_c = 25$ (kN/m^3)
- Unit Weight of Soil : $\gamma_t = 18$ (kN/m^3)

[Positive Earth Pressure : P_P]

The foundation of wall is placed at a 1.5 m depth of embedment in cohesive soil. The intensity of the positive earth pressure of cohesiveness is calculated from the following formula;

$$p_P = \sum(\gamma \cdot h) + w + 2c$$

where, p_P =Intensity of positive earth pressure (kN/m^2)

γ =Unit weight of soil (kN/m^3)

h =Thickness of soil layer to consider for positive earth pressure (m)

w =Loading on ground surface (kN/m^2)

c =Cohesion of soil (kN/m^2)

[Siding Force : P_S]

Sliding force is considered as an earth pressure that acts against the retaining wall. It is assumed to be a deterrent force against sliding failure based on the following formula:

$$F_s > \frac{M_R + P_S \cdot R}{MA}$$

where, F_s =Required safety factor for circular sliding stability (=1.2)

M_R =Resisting moment on sliding surface (kN・m/m)

P_S =Necessary deterrent force against required safety factor (kN/m)

R =Radius of sliding circle (kN/m^2)

M_A =Active moment on sliding surface (kN・m/m)

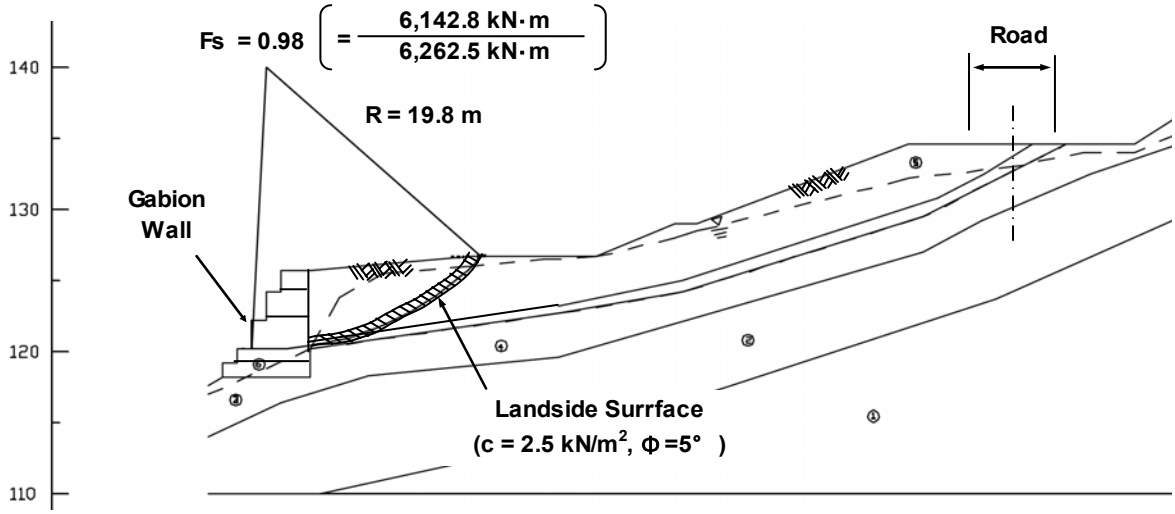


Figure 10.31 Circular Sliding Analysis of Landslide Block on the Back Side of the Wall

As a result of the trial circular sliding analysis for the small landslide block, the shear strength parameters were established as shown in Figure 10.31. Using this condition, the deterrent force against the required safety factor (Fs=1.2) is calculated as follows;

$$P_S = \frac{F_{s\cdot} M_R - M_A}{R} = \frac{1.2\ x\ 6{,}262.5 - 6{,}262.6}{19.8} = 63.2\ \text{(kN/m)}$$

The above force is used as the sliding force, and the active earth pressure under the sliding surface is not considered as an external force, because the soil layer consists of very stiff silty soil, of which, the cohesion is evaluated as c=210 (kN/m^2).

[Water Pressure: P_W]

The design ground water level is to be lowered to the same elevation as the landslide surface by horizontal drain holes and this elevation is assumed in the calculation of water pressure.

[Summary of External Forces]

The above external forces are summarized in Table 10.10.

Table 10.10 Summary of External Forces

Loading	Vertical Force V (kN/m)	Lateral Force H (kN/m)	Distance (m)		Moment (kN・m/m)	
			x	y	V・x	H ・y
Dead Load	526.4	0.0	2.36	-	1,245.3	0.0
Positive Earth Pressure	0.0	(-325.7)	-	0.40	-	-130.3
Landslide Force	0.0	63.2	-	5.17	-	326.7
Water Pressure	0.0	76.8	-	1.30	-	99.8
Σ	526.4	140.0	-	-	1,245.3	296.2

(b) Stability Calculations

The stability calculations with regards to tumble, slip and bearing capacity of the wall body are summarized in Table 10.11.

Table 10.11 Summary of Stability Calculation Results

	d (m) $(=\frac{\sum(V\cdot x)-\sum(H\cdot y)}{V})$	$B/6$ (m)	e (m)	**Judgment**	
Tumble	1.80	0.70	0.30	|e| < B/6	-O.K.-

Slip	V (kN/m)	H (kN/m)	μ	Judgment	
	526.4	140.0	0.5	Fs = 1.88 >1.5	-O.K. -

Bearing Capacity of Ground	N_c	N_q	N_γ	θ (deg)	i_c, i_q	i_γ	
	5.14	1.00	0.00	15	0.69	0.00	
	q_d (kN/m^2)	F	q_a (kN/m^2)	q_1 (kN/m^2)	q_2 (kN/m^2)	Judgment	
	7,263.6	3	2,421.2	179.0	71.6	$q_1, q_2 < q_a$	-O.K.

(c) Intensity of Concrete Stress

The stress intensities of single reinforced concrete are calculated from the following formulas;

$$\sigma_C = \frac{2 \cdot M \cdot 10^3}{k \cdot j \cdot b \cdot d^2}$$

$$\sigma_S = \frac{M \cdot 10^3}{A_S \cdot j \cdot d}$$

$$\tau_c = \frac{S \cdot 10}{b \cdot d}$$

where, σ_c = Bending stress intensity of concrete (N/mm^2)

σ_s = Tensile stress intensity of reinforcing bars (N/mm^2)

τ_c =Shear stress intensity of concrete (N/mm^2)

b =Effective width of concrete member (m)

d =Effective height concrete member (m)

k, j =Parameter to be determined by steel ratio (p)

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p$$

$$j = 1 - \frac{k}{3}$$

$$p = \frac{A_s}{b \quad d}$$

A_s =Reinforcement concept (cm^2)

n =Ratio for Young's modulus of concrete and reinforcing bars (=15)

S =Shear force to act on designing section (kN)

M =Bending moment to act on design section (kN・m)

Examinations of intensity stresses were conducted on the front wall (at Section I-I and Section II-II in Figure 10.32), the toe slab (at Section III-III in Figure 10.32) and the heel slab (at Section IV-IV in Figure 10.32), and then the stress intensities were calculated to be within the required range as summarized in Table 10.12.

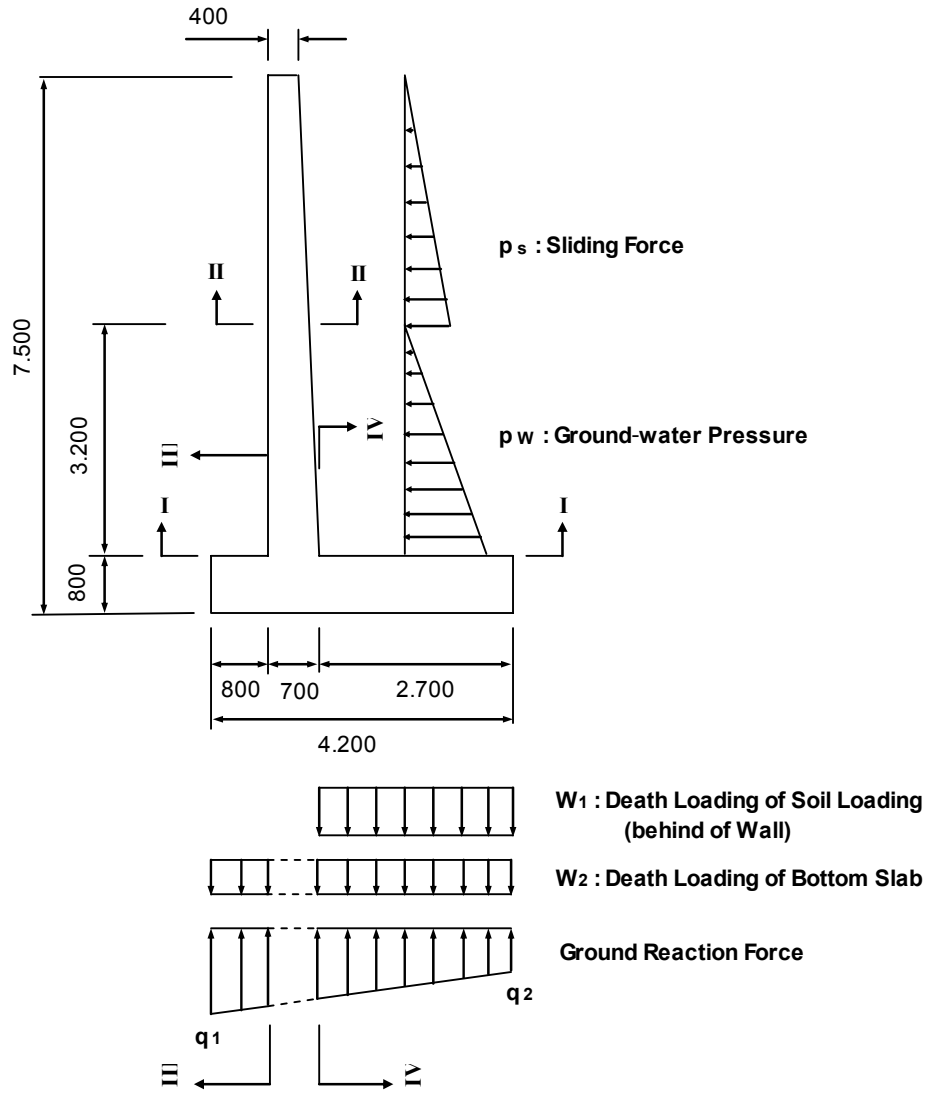


Figure 10.32 External Forces for Concrete Stress Calculations

Table 10.12 Summary of r Stress Intensity of Reinforcement Concrete

a) Front Wall (at Section I-I in Figure 10.32)

Section Force	S (kN/m)	M (kN・m/m)
	115.6	336.2

Re-bar Arrangement	b (cm)	d (cm)	As (cm^2)		p	k	j
	100	63	D25@15.0cm	33.8	0.0054	0.327	0.897

Stress Intensity	Bending Stress (N/mm^2)				Bending Stress (N/mm^2)			
	σc	σca	**Judgment**		σs	σSa	**Judgment**	
	5.78	7.0	$\sigma c<\sigma ca$	-OK-	152	160	$\sigma s<\sigma sa$	-OK-
	Shear Stress (N/mm^2)							
	τc	τca	**Judgment**					
	0.18	0.36	$\tau c<\tau ca$	-OK-				

b) Front Wall (at Section II-II in Figure 10.32)

Section Force	S (kN/m)	M (kN・m/m)
	64.4	75.3

Re-bar Arrangement	b (cm)	d (cm)	As (cm^2)		p	k	j
	100	49	D16@15.0cm	11.2	0.0023	0.230	0.923

Stress Intensity	Bending Stress (N/mm^2)				Bending Stress (N/mm^2)			
	σc	σca	**Judgment**		σs	σSa	**Judgment**	
	2.95	7.0	$\sigma c<\sigma ca$	-OK-	149	160	$\sigma s<\sigma sa$	-OK-
	Shear Stress (N/mm^2)							
	τc	τca	**Judgment**					
	0.13	0.36	$\tau c<\tau ca$	-OK-				

c) Toe Slab (at Section III-III in Figure 10.32

Section Force	S (kN/m)	M (kN・m/m)
	121.4	49.7

Re-bar Arrangement	b (cm)	d (cm)	As (cm^2)		p	k	j
	100	70	D16@25.0cm	7.9	0.0011	0.159	0.947

Stress Intensity	Bending Stress (N/mm^2)				Bending Stress (N/mm^2)			
	σc	σca	**Judgment**		σs	σSa	**Judgment**	
	1.35	7.0	$\sigma c<\sigma ca$	-OK-	94	160	$\sigma s<\sigma sa$	-OK-
	Shear Stress (N/mm^2)							
	τc	τca	**Judgment**					
	0.17	0.36	$\tau c<\tau ca$	-OK-				

d) Heel Slab (at Section IV-IV in Figure 10.32)

Section Force	S (kN/m)	M (kN・m/m)
	115.6	336.2

Re-bar Arrangement	b (cm)	d (cm)	As (cm^2)		p	k	j
	100	70	D25@15.0cm	33.8	0.0048	0.314	0.895

Stress Intensity	Bending Stress (N/mm^2)				Bending Stress (N/mm^2)			
	σc	σca	**Judgment**		σs	σSa	**Judgment**	
	4.89	7.0	$\sigma c<\sigma ca$	-OK-	159	160	$\sigma s<\sigma sa$	-OK-
	Shear Stress (N/mm^2)							
	τc	τca	**Judgment**					
	0.17	0.36	$\tau c<\tau ca$	-OK-				

(3) Design of Surface Drainage System

(a) Calculation Process for Design Run-off

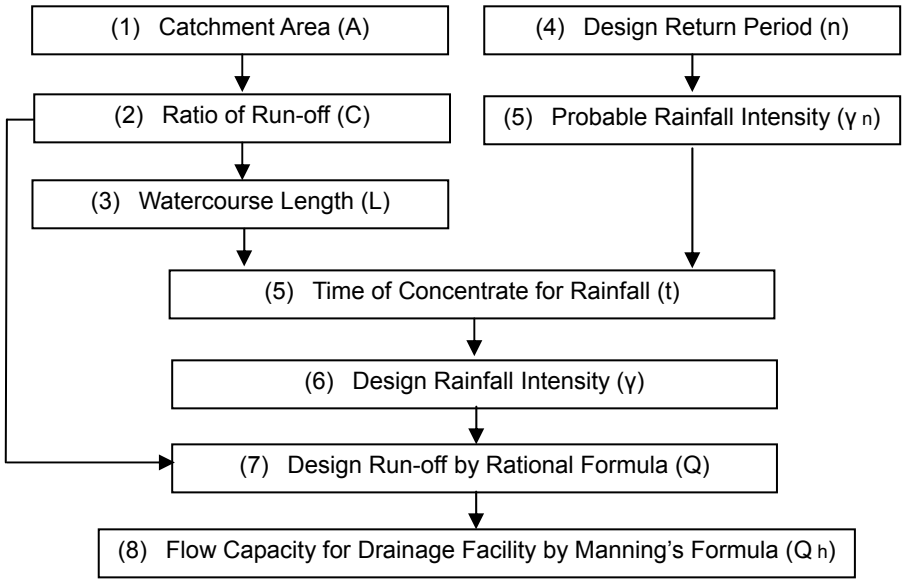


Figure 10.33 Calculation Flow for Design Run-off

The design run-off was calculated from the Rational Formula as presented in Figure 10.33. Each parameter required as input for the Rational Formula was determined as follows:

[Design Return Period : n]

10 years return period was adopted.

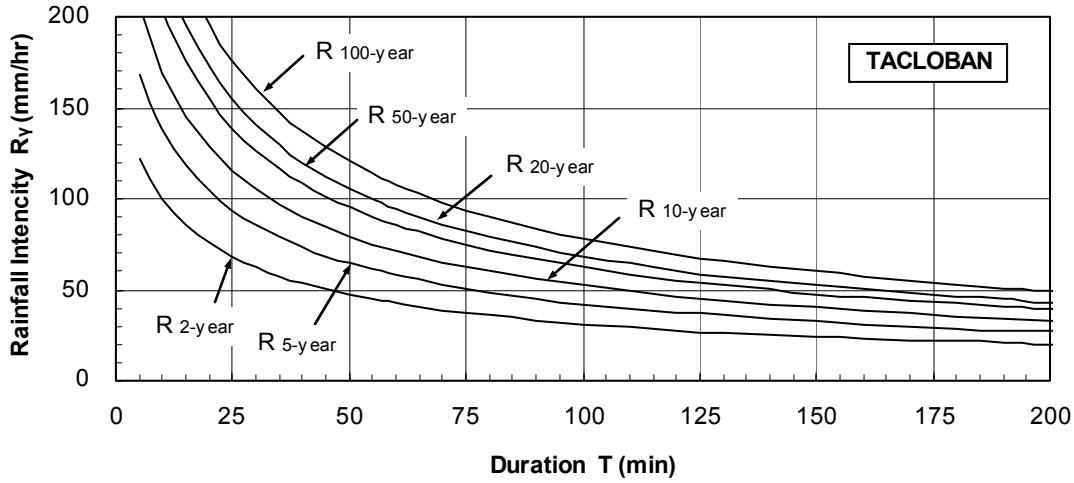
[Probability Rainfall Intensity : γ_n]

"Technical Standards and Guidelines for Planning and Design (Draft) Volume IV: Natural Slope Failure Countermeasures" (March 2002) produced by the Project for the Enhancement of Capabilities in Flood Control and Sabo Engineering of the DPWH" gives rainfall intensity duration frequency curves based on the rainfall data of nationwide synoptic stations. The probability of rainfall intensities at Tacloban Synoptic Station as shown in Figure 10.34 was used to determine the design run-off for a 10 year-return period as follows;

$$\gamma_{10-\text{year}} = \frac{1{,}407.27}{(T + 10.57)^{0.70}}$$

where, T = Duration of Rainfall (min)

It is important to note that the above "Duration of Rainfall (T)" was chosen to be equal to the "Time of Concentrate for Rainfall (t)" in Figure 10.33.



Source: "Technical Standards and Guidelines for Planning and Design (Draft) Volume IV: Natural Slope Failure Countermeasures" (March 2002), FCSEC of DPWH

Figure 10.34 Probability of Rainfall Intensity at Tacloban Synoptic Station

[Time of Concentrate for Rainfall : t]

$$t = t_1 + t_2$$

where, t = Time of concentrate of rainfall (min)

t_1 = Inflow time from farthest point of catchment area to the drainage ditch (min)

t_2 = Flow time to examination point in the drainage ditch (min)

$$t_2 = \frac{\ell}{60\,v}$$

ℓ = Length of drainage ditch to examination point (m)

v = Average water velocity in the drain ditch (m/sec)

The above "Inflow Time (t_1)" was determined from the "Kerby Formula" as follows;

$$t_1 = \left(\frac{2}{3} \times 3.28\, L \frac{n_d}{\sqrt{S}}\right)^{0.467}$$

where, L = Length of watercourse in the catchment area from farthest point to the drainage ditch (m)

n_d = Delay coefficient as shown in Table 10.13

S = Gradient of catchment area

Table 10.13 Delay Coefficient in Kerby Formula for Inflow Time

Run-off Condition	n_d	Application
Cement/Asphalt concrete	0.013	Road surface, concrete block
Smooth Impervious surface	0.020	Mortar spray
Smooth and stiff bare ground	0.100	Disposal site, borrow site
Poor grassland, cultivated area, moderately rough bare ground	0.200	
Meadow, ordinary grassland	0.400	Vegetation slope
Deciduous forest	0.600	
Conifer forest	0.800	

Source : Design Standard of Japan Highway Corporation (1983)

[Rational Formula]

$$Q = \frac{1}{360} \cdot C \cdot \gamma \cdot A$$

where, Q = Design run-off (m^3/sec)

C = Ratio of run-off

γ = Design rainfall intensity (mm/hr)

A = Catchment area (ha)

[Flow Capacity of Drainage Facility by Manning's Formula]

$$Q_h = A_h \cdot V_h$$

$$V_h = \frac{1}{n} \cdot R^{2/3} \cdot I_h^{1/2}$$

where, Q_h = Flow capacity of drainage facility (m^3/sec)

V_h = Average water velocity (m/sec)

A_h = Cross sectional Area of flow (80 % full) (m^3)

n = Coefficient of roughness as shown in Table 10.14

R = Hydraulic mean depth (m)

$$R = \frac{A_h}{P}$$

P = Length of wetted perimeter of drainage facility (m)

I_h = Average inclination

Table 10.14 Coefficient of Roughness (n) for Manning Formula

Classification of Drain Facility		n
Earth Ditch	Soil (uniform section)	0.016 ~ 0.025 (0.022)
	Soil (with weed)	0.016 ~ 0.033 (0.027)
	Gravel	0.022 ~ 0.030 (0.025)
	Rock	0.022 ~ 0.040 (0.035)
Site Construction	Cement Mortar	0.011 ~ 0.014 (0.013)
	Concrete (trowel smoothing)	0.011 ~ 0.015 (0.015)
	Concrete (gravel on bottom)	0.011 ~ 0.020 (0.017)
	Dry Masonry	0.023 ~ 0.035 (0.032)
	Wet Masonry	0.017 ~ 0.030 (0.025)
Factory Production	Centrifugal reinforced concrete pipe	0.013
	Reinforced concrete pipe	0.015
	Corrugated metal pipe (Type-I)	0.024
	Corrugated metal pipe (Type-II)	0.033

*Note : ( ) means standard value

Source : Manual for Drainage Design, Japan Road Association (June 1987)

(b) Determination of Catchment Area

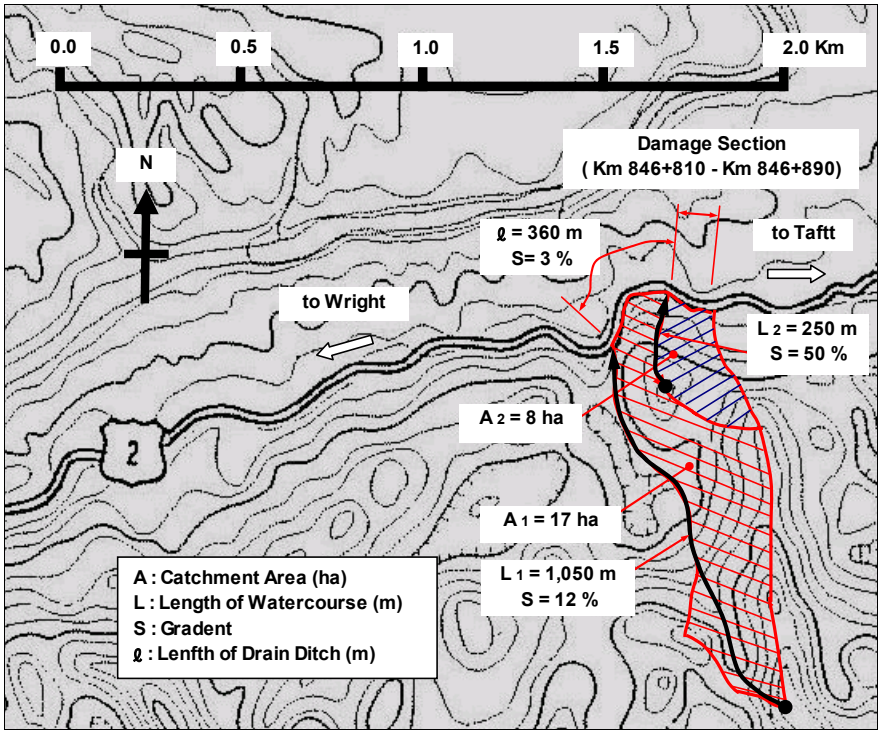


Figure 10.35 Catchment Area of Damaged Section Wright Taft Road, Km836

A map at a scale of 1/10,000 is normally utilized for the catchment area examination. However, because the said scale map was not available, a scale of 1/50,000 was used. Figure 10.35 shows the catchment area from which rainfall flows into damaged section.

(c) Discharge Volume of Drainage Facilities

Figure 10.36 shows a diagram of the surface water treatment system for the restoration works. The examination of discharge volume for the drainage facilities on the representative points, which are presented in Figure 10.36, are shown in Table 10.15.

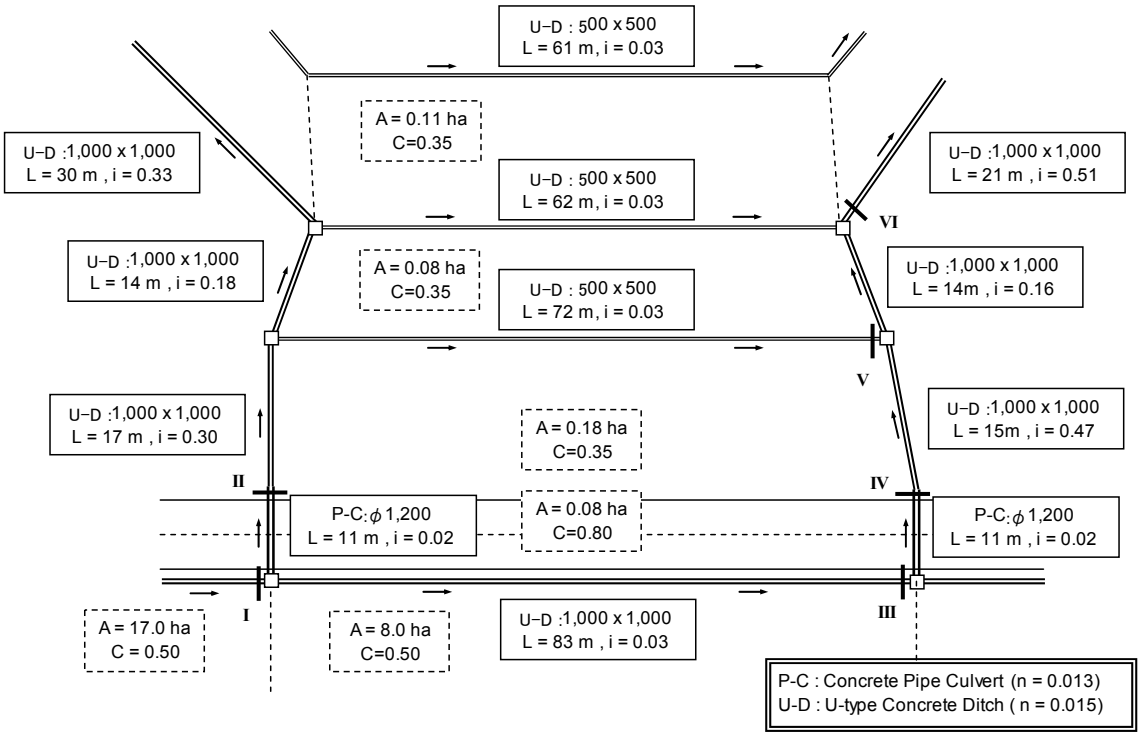


Figure 10.36 Diagram of Surface Water Drainage System

Table 10.15 Design Run-off and Flow Capacity of Drainage Facilities

a) at Section I in Figure 10.36

Design Run-off	t_1 (min)	t_2 (min)	t (min)	γ (mm/hr)	Σ (AxC) (ha)	Q (m^3/sec)
	54.9	2.3	57.2	73.6	8.75	1.79
Flow Capacity	A_h (m^2)		R (m)	V_h (m/sec)		$Q_{Ah80\%}$ (m^3/sec)
	100 %	80%		Calculated	Design	
	1.00	0.80	0.33	5.55	2.50	2.00
Judgment	$Q < Q_{Ah80\%}$		- OK -			

b) at Section II in Figure 10.36

Design Run-off	t_1 (min)	t_2 (min)	t (min)	γ (mm/hr)	Σ (AxC) (ha)	Q (m^3/sec)
	54.9	2.4	57.3	73.6	8.75	1.79

Flow Capacity	A_h (m^2)		R (m)	V_h (m/sec)		$Q_{Ah80\%}$ (m^3/sec)
	100 %	80%		Calculated	Design	
	1.31	0.91	0.30	5.55	2.50	2.26

Judgment	$Q < Q_{Ah80\%}$	- OK -

c) at Section III in Figure 10.36

Design Run-off	t_1 (min)	t_2 (min)	t (min)	γ (mm/hr)	Σ (AxC) (ha)	Q (m^3/sec)
	20.1	0.6	20.7	126.5	4.00	1.41

Flow Capacity	A_h (m^2)		R (m)	V_h (m/sec)		$Q_{Ah80\%}$ (m^3/sec)
	100 %	80%		Calculated	Design	
	1.00	0.80	0.33	4.88	2.50	2.00

Judgment	$Q < Q_{Ah80\%}$	- OK -

d) at Section IV in Figure 10.36

Design Run-off	t_1 (min)	t_2 (min)	t (min)	γ (mm/hr)	Σ (AxC) (ha)	Q (m^3/sec)
	20.1	0.6	20.7	126.3	4.00	1.40

Flow Capacity	A_h (m^2)		R (m)	V_h (m/sec)		$Q_{Ah80\%}$ (m^3/sec)
	100 %	80%		Calculated	Design	
	1.00	0.80	0.33	4.88	2.50	2.26

Judgment	$Q < Q_{Ah80\%}$	- OK -

e) at Section V in Figure 10.36

Design Run-off	t_1 (min)	t_2 (min)	t (min)	γ (mm/hr)	Σ (AxC) (ha)	Q (m^3/sec)
	5.9	0.5	6.4	194.1	0.13	0.07

Flow Capacity	A_h (m^2)		R (m)	V_h (m/sec)		$Q_{Ah80\%}$ (m^3/sec)
	100 %	80%		Calculated	Design	
	0.25	0.20	0.17	3.50	2.50	0.50

Judgment	$Q < Q_{Ah80\%}$	- OK -

f) at Section VI in Figure 10.36

Design Run-off	t_1 (min)	t_2 (min)	t (min)	γ (mm/hr)	Σ (AxC) (ha)	Q (m^3/sec)
	20.1	0.8	20.9	125.8	4.16	1.45

Flow Capacity	A_h (m^2)		R (m)	V_h (m/sec)		$Q_{Ah80\%}$ (m^3/sec)
	100 %	80%		Calculated	Design	
	1.00	0.80	0.33	22.90	4.00	3.20

Judgment	$Q < Q_{Ah80\%}$	- OK -