10.2.4 Landslide－Lagawe-Banaue Road, Km 301 (Region CAR)

(1) Geotechnical Investigation and Landslide Monitoring Results

The investigation and monitoring at the site included boring, core drilling, standard penetration tests, groundwater level observations, and landslide movement surveys by pipe strain gauges.

(a) Landslide Condition

A newly activated landslide block is located in the lower slope of an old and huge landslide area. The newly activated landslide caused damage to a section of more than 150 m in length at the toe of the landslide block along the Lagawe-Banaue road, as shown in Figure 10.37 along with the geological investigation location. Table 10.16 shows the scale of the landslide block.

Table 10.16 Scale of Landslide Blocks

Landslide Block	Width W	Length L	Depth H	Volume W×L×H×2/3
Newly activated sub-landslide	130 m	140 m	10.0 m	120,000 m^3
Old and huge Landslide	200 m	350 m	10.0 m	470,000 m^3

The sliding mass was a layer of boulders and cobbles with some sand and fines (with an N value of 5 to 20), which is the displaced soil of the old landslide.

Photo 10.3 Front View of Damaged Section (Lagawe-Banaue Road Km 301)

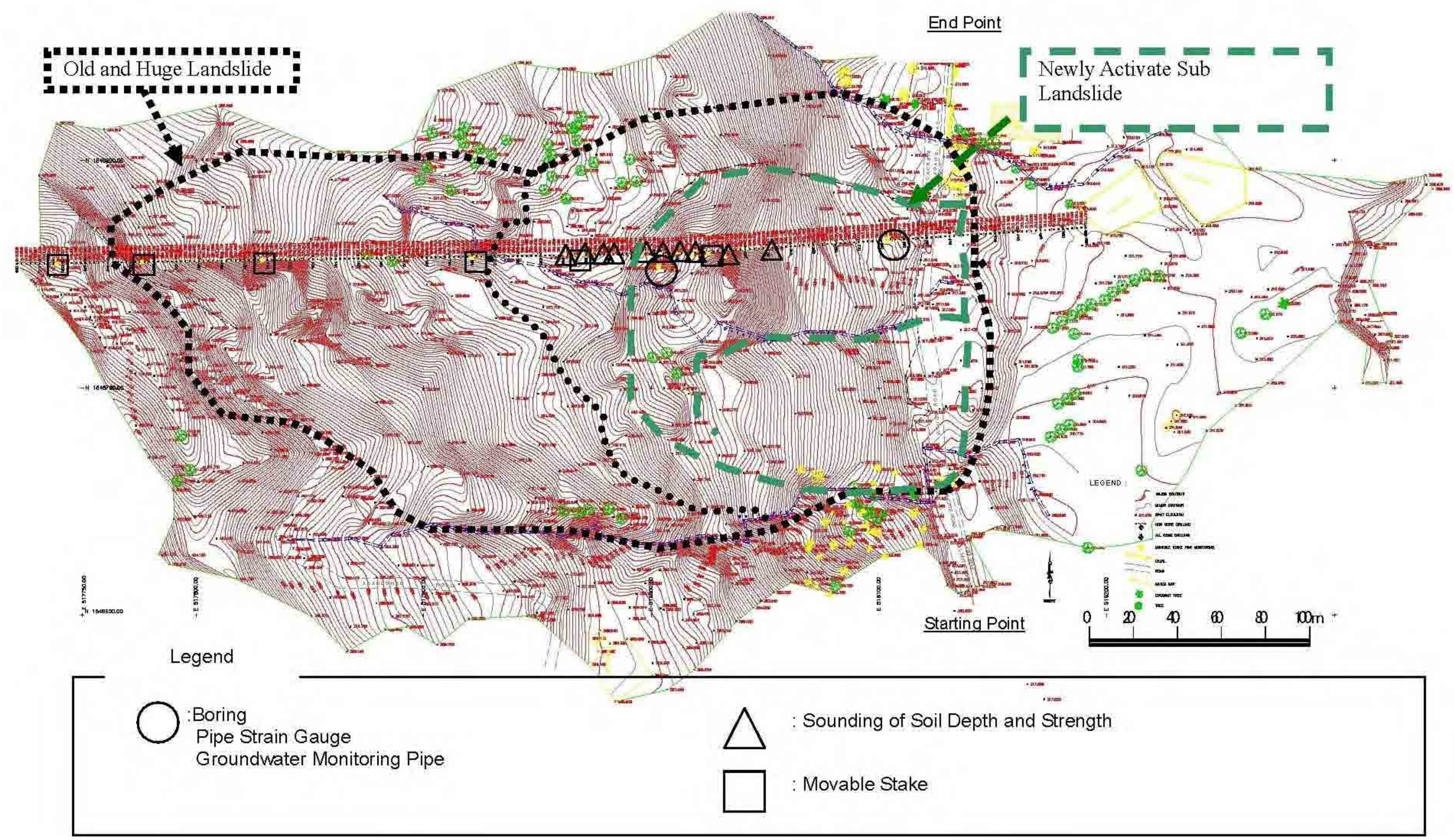

Figure 10.37 Configuration of Landslide and Engineering Geological Investigation Site of Lagawe Banaue Road Km 301

(b) Groundwater Level

The groundwater levels existed at a depth of 1 to 3 m below the ground surface and numerous springs indicated abundant groundwater in the landslide area.

(c) Survey of Landslide Movement

Two pipe strain gauges were installed to check the depth of the sliding surface of the Newly Activated Landslide. The strain was observed to be at depth of about 9.0 m below the ground surface at the lower slope (BV-1), and at a depth of 20.0 m below the ground surface at the upper slope (BV-2). The strains during the monitoring show no accumulative tendency. The monitoring results indicate that the Newly Activated landslide block probably moves intermittently.

(d) Mechanism and Main Causes of Landslide

The newly activated landslide block is a part of an old and huge landslide. It is a reactivation of some parts of the sliding surface of the old and huge landslide. The distribution of cracks, small surface failures and scarps indicate that the landslide block had a circular configuration head located 110 m from the road, and a straight toe along the valley side of the road. Boring investigation showed a weak surface between the sandstone layer and the overlying ‘layer of boulders and cobbles with sand and some fines’.

The failure mechanisms of the landslide were as follows.

- The geological survey indicated that the layer of cobbles and boulders with some sand and fines is distributed on the sandstone layer. The boundary between the two layers inclines toward the road and was the sliding surface of the old and huge landslide and the newly activated landslide.
- The road cut at the toe of the landslide block was the main trigger of the landslide reactivation.
- In the newly activated landslide block area, open and vertical cracks developed in large numbers. These cracks contributed to the infiltration of rain water. The infiltrated water, flowing down the pre-existing sliding surface, lead to not only an increase of pore-water pressure and the unit weight of the slide material, but also to softening of the sliding surface.

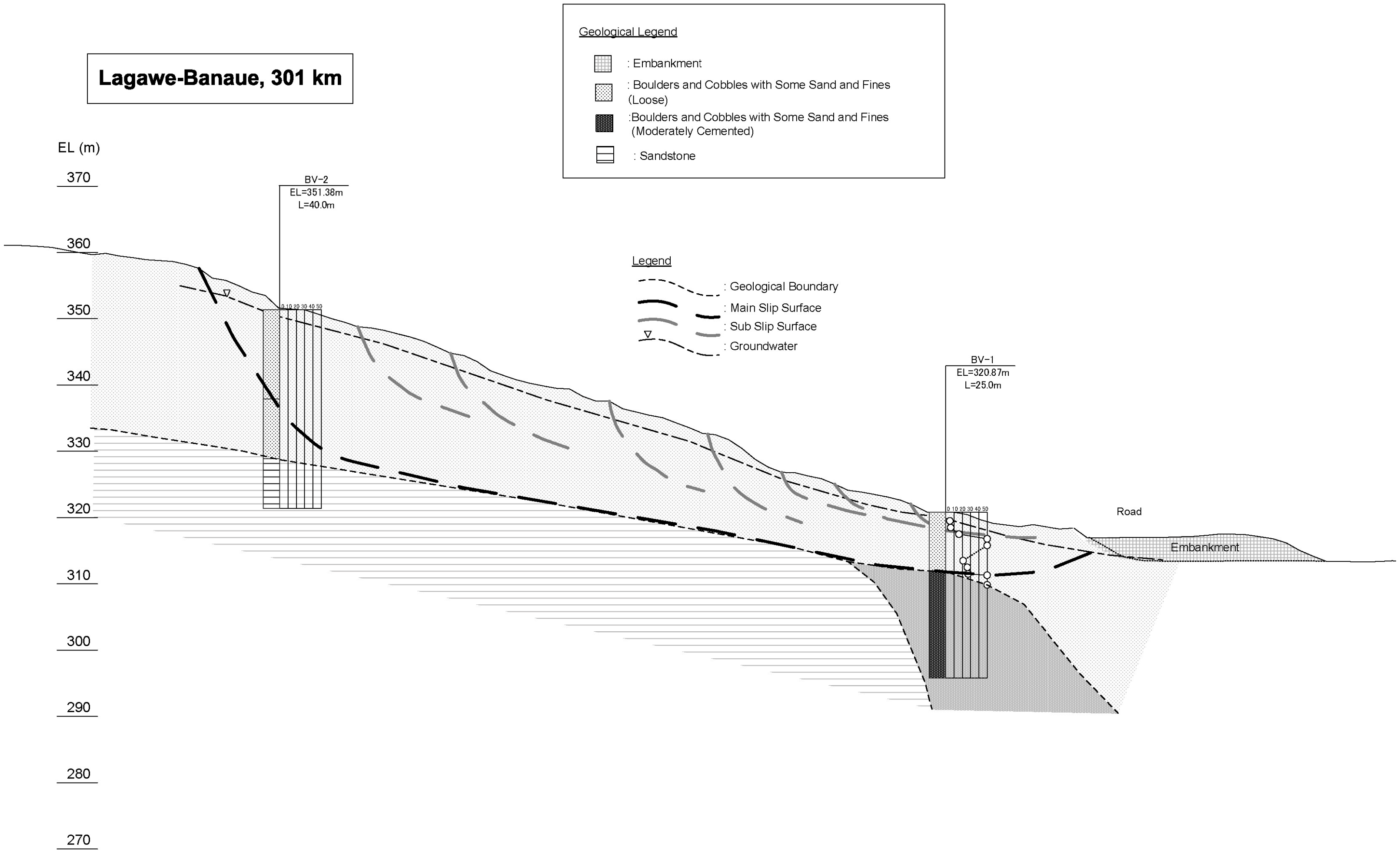

Figure 10.38 Engineering Geological Profile of the Newly Activated Landslide of Lagawe-Banaue Road, Km 301

(2) Stability Analysis

(a) Determination of Conditions and Parameters

The conditions and parameters for the stability analysis of the newly activated landslide block were determined on the basis of the investigation and monitoring results and the back calculation method* of common engineering practice. Table 10.17 lists the preconditions, parameters and calculation results.

Under the condition that the present safety factor is assumed to be 1.0, the relationship between cohesion and internal friction angle of the sliding surface is shown in Figure 10.39 as a reference.

Table 10.17 Conditions and Parameters of Stability Analysis by Swedish Slice Method

Conditions and Parameters			Remarks
Precondition			
1	Cohesion of sliding surface	C = 10 kN/m^2	Thickness of sliding mass is 10.0 m
2	Unit weight of sliding mass	γt = 18 kN/m^3	Common value
3	Groundwater level before countermeasure execution	GL = 1.0 m	Using the observed results
4	Present safety factor	F_0 = 1.0	No noticeable ongoing movement
Obtained by Back Calculation*			
5	Internal friction angle	ϕ = 22.4 degrees	Obtained by back calculation*
Proposed Safety Factor and Effectiveness			
6	Proposed safety factor	PFs = 1.2	Object to be protected is a highway.

Back calculation*: Safety factor is set, and ground strength will be calculated.

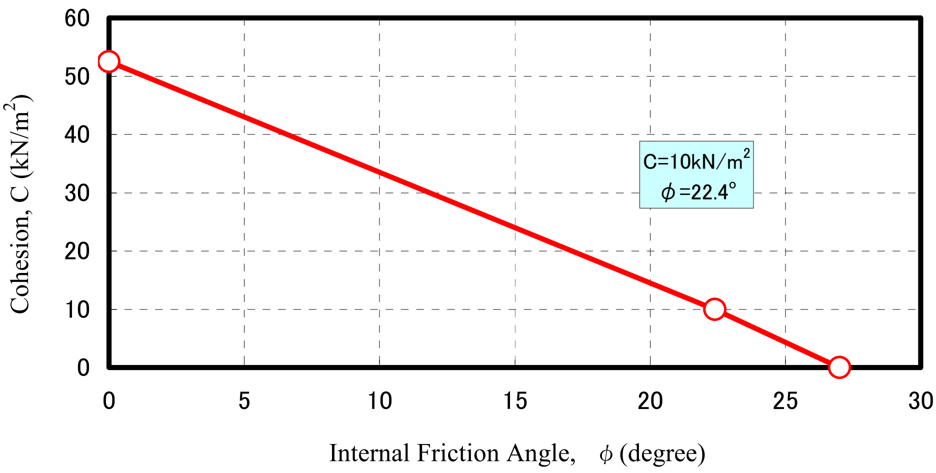

Figure 10.39 C- ϕ Relationship Chart using Back Calculation Method

In general, cohesion, C, is determined based on the vertical thickness of the sliding mass as shown in Chapter 5 of this Guide III. Because the thickness of the sliding mass is about 10 m, the cohesion is set to be 10 kN/m^2; and then the internal friction angle is obtained by using back calculation.

(b) **Result of stability analysis**

The stability analysis result is given in Table 10.18. A conceptual flow for the analysis is shown in Figure 10.40.

The expected reduction in the groundwater level by horizontal drain holes is generally about 1 to 3 m in the case of the standard-scale landslide with a landslide depth of 20 m. The studied landslide block is of about 10 m depth on average and therefore, horizontal drain holes may lower the groundwater level about 2.0 m.

Table 10.18 Stability Analysis Result in unit Width

Condition		Driving Force D (kN/m)	Resisting Force R (kN/m)	Safety factor Fs	Required Preventive Force P_R (kN/m)
Present Condition		8283	8311	1.000	1629
Reduction of Groundwater Level	1.0 m	8283	8825	1.065	1115
	2.0 m	8283	9333	1.127	605
	3.0 m	8283	9924	1.200	0

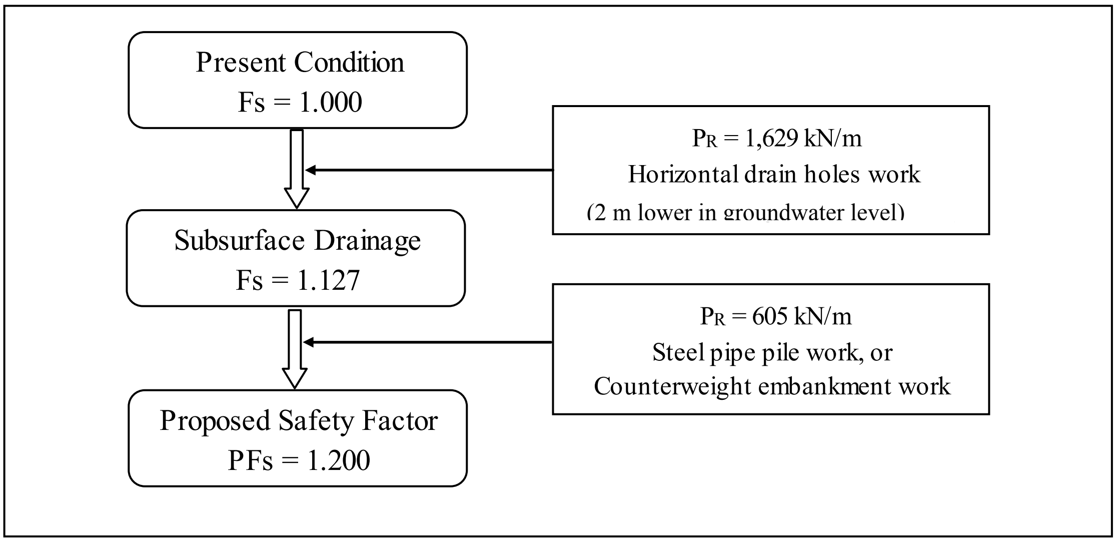

Figure 10.40 Conceptual Illustration of Stability Analysis

(3) Selection and Design of Countermeasure Works

(a) Selection of Countermeasure Works

As explained in Chapter 5 of this Guide, the countermeasures for stabilizing a landslide mainly include 1) Drainage Work (surface and subsurface), 2) Cutting Work, 3) Filling Work, 4) Piling Work, and 5) Anchoring Work.

As shown in the stability analysis of the landslide slope in the figure below, a 1 m reduction in groundwater level results in about a 6% increase in safety factor. This indicated that the drainage of groundwater is a key solution to stabilize the landslide.

The newly activated landslide block is located on the lower slope of the old and huge landslide area; cutting work at its head may cause further instability of the upper slope and is thus considered to be unsuitable. Moreover, gradients of the ground surface and the sliding surface are both gentle at about 10 degrees; therefore anchoring work is ineffective.

Filling work will lead to a shift of the present route; however it is cost-effective in this case.

Conceivable countermeasures for the prevention of the landslide block are 1) Drainage work, 2) Filling work and 3) Piling work.

The following three plans were proposed to stabilize and/or mitigate the landslide block.

Option-1: Horizontal drain holes + Steel pipe piles

To decrease the groundwater level, horizontal drain holes are proposed at three locations to cover the entire landslide area. The water collected by these holes is released into the drainage ditch and is safely discharged out of the landslide area.

The stability of the landslide slope is further improved by installing steep pipe piles to achieve the proposed safety factor (PFs=1.2).

Option-2: Horizontal drain holes + Filling + Route relocation (shift)

Similar to Plan I, horizontal drain holes are provided at three locations to reduce the groundwater level.

Instead of steel pipe piles, filling at the toe is provided to further improve the stability of the landslide block.

Because the present route at the target section will be filled, shifting of the road toward the valley side is necessary.

Option-3: Horizontal drain holes

Horizontal drain holes are first provided at three locations to reduce the groundwater level. Further countermeasures will be considered depending on the effectiveness of the horizontal drain holes.

The general plans and typical sections for the above-mentioned three options are shown in Figures 10.41 to 10.44.

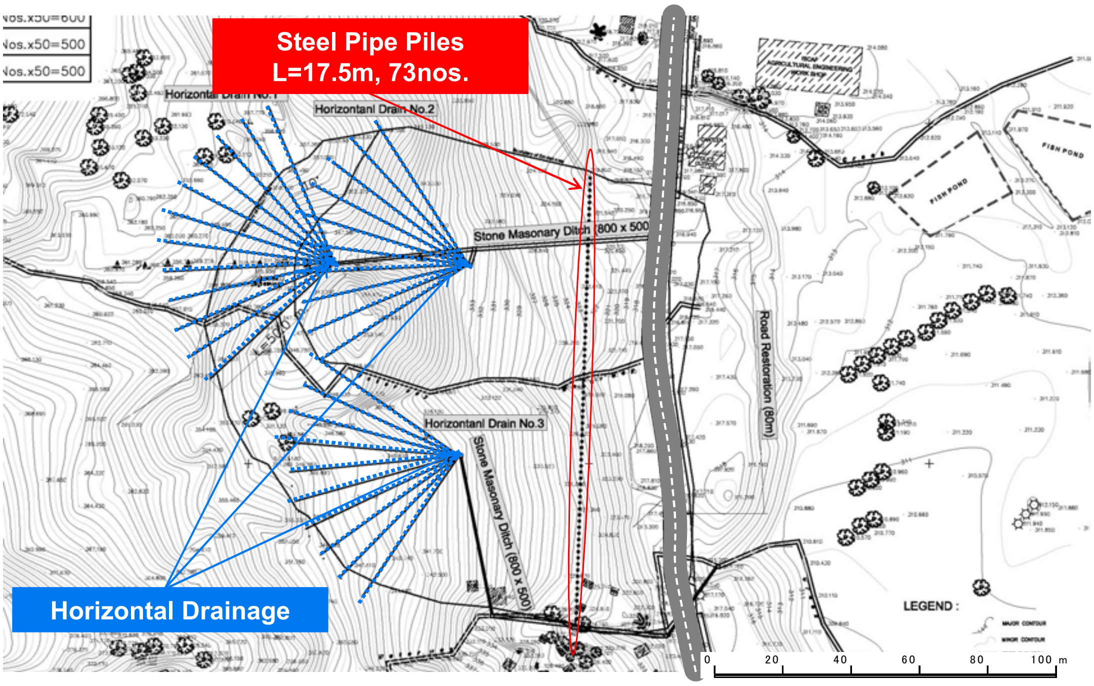

Figure 10.41 Countermeasure Plan of Option-1

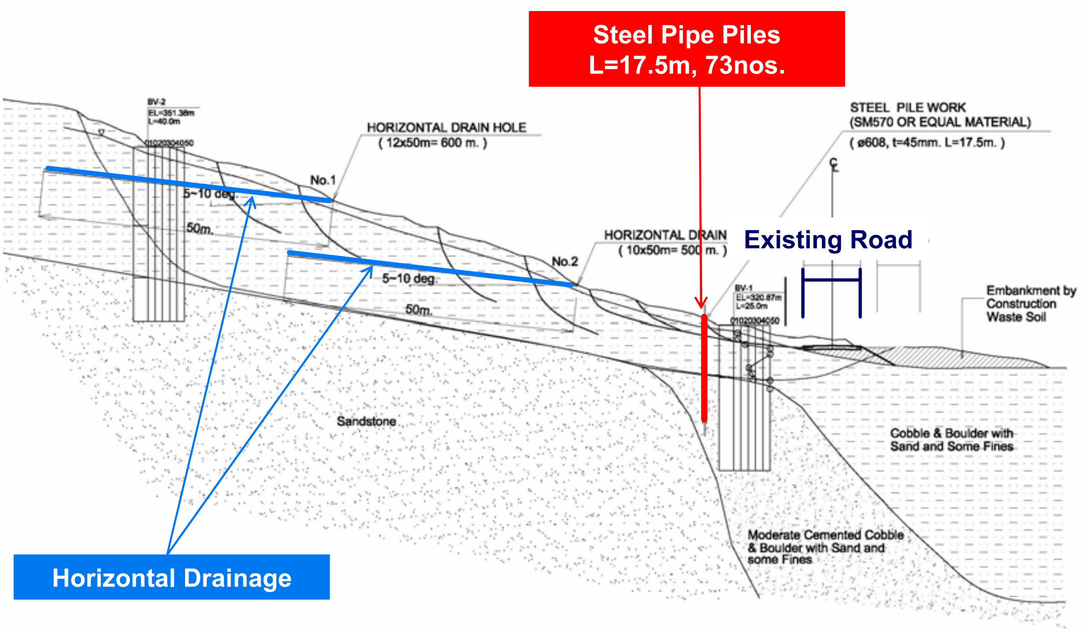

Figure 10.42 Typical Cross Section of Option-1

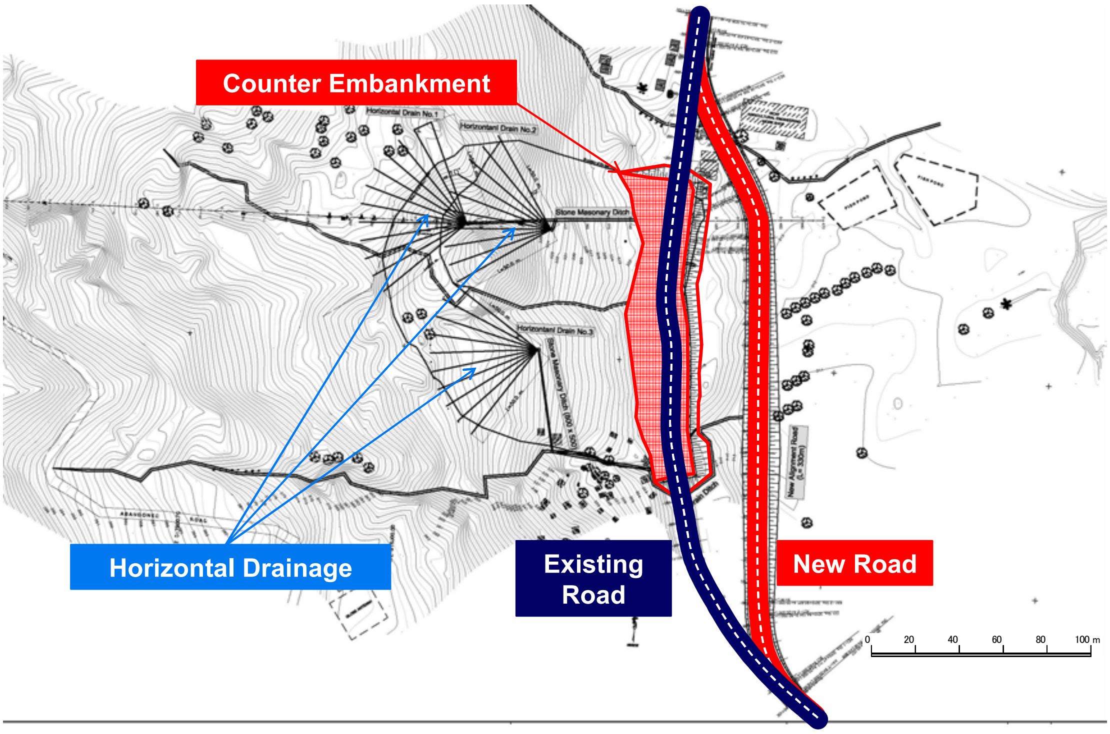

Figure 10.43 Countermeasure Plan of Option-2

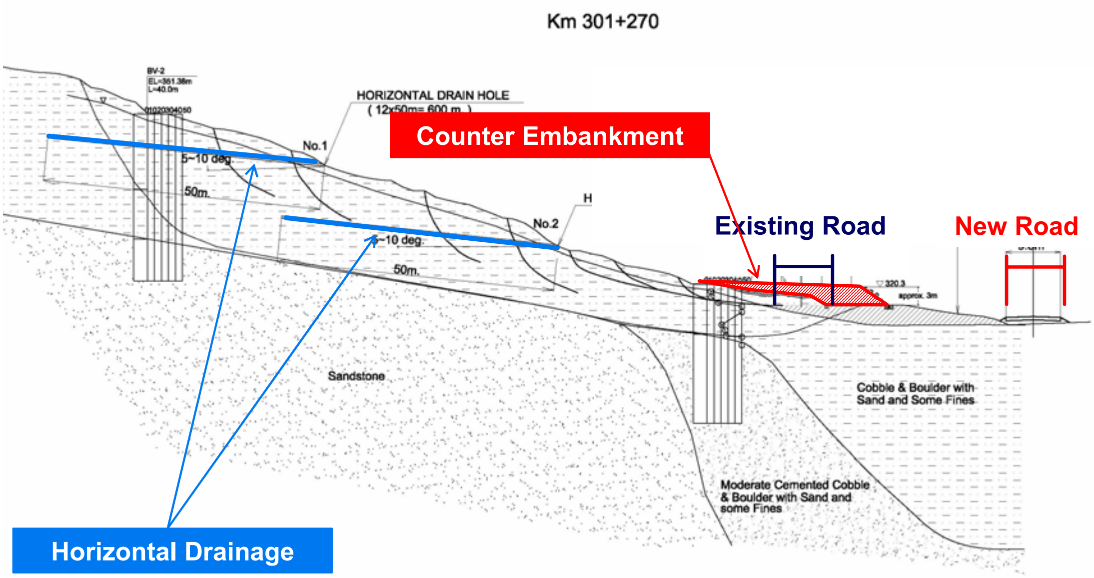

Figure 10.44 Typical Cross Section of Option-2

(b) Design of Horizontal Drain Holes with Drainage Ditch for All Options

The work is to bring down the groundwater level as low as possible so that long-term stability could be secured.

Three locations of horizontal drain holes are proposed so as to cover the whole landslide area. To drain groundwater effectively, the horizontal drain holes are designed to penetrate 5 to 10 m below the sliding surface, and the length of the drainage holes is 50 m at each location. The drain holes are drilled in a radial shape with an opening angle of about 10 degrees. Also, to facilitate the groundwater collection, the drain pipes are inserted in an incline of 5 to 10 degrees upward.

The total quantity of horizontal drain holes in the landslide block is summarized in table 10.19. Detail layout of the horizontal drain holes is shown Figure 10.41.

Table 10.19 Quantities of Horizontal Drain Holes

Location	Length (m)		Number of boreholes		Subtotal	Remarks
No.1	50	×	12	=	600	
No.2	50	×	10	=	500	
No.3	50	×	10	=	500	
Total			32		1,600	

(d) Design of Steep Pipe Piles for Option-1

The steep pipe piles were designed according to Chapter 5 of this Guide III.

1) Determination of conditions and parameters for pile design

The conditions and parameters for the design of steel pipe piles are listed in Table 10.20. They are based on either the investigation or other common engineering practice.

Table 10.20 Conditions and Parameters for Pile Design

Conditions and Parameters			Remarks
1	Required preventive force	P_R=605 kN/m	Stability analysis
2	Landslide width	W = 130 m	Landslide investigation
3	Thickness of sliding mass	D = 10	Landslide investigation
4	Cohesion of sliding surface	C = 10 kN/m^2	Thickness of sliding mass is 10.0 m
3	Internal friction angle of slide surface	ϕ = 22.4 degrees	Obtained by back calculation
4	Unit weight of sliding mass	γ t = 18 kN/m^3	
5	Average N value of sliding mass	N = 20	Landslide investigation
6	Gradient of sliding surface	θ =10 degrees	Landslide investigation
7	Cohesion of stationary layer*	C = 100 kN/m^2	Geologically estimated
8	Internal friction angle of stationary layer	ϕ = 30 degrees	Geologically estimated
9	Unit weight of stationary layer	γ t = 20 kN/m^3	Geologically estimated
10	Average N value of stationary layer	N = 50	Landslide investigation

Note: Stationary layer means the stable ground below the sliding surface.

Moreover, allowable stresses of steel material are shown in the table below. In general, only the allowable short-term stress of steel material is checked when it is used for landslide piles.

Table 10.21 Pile Type and Allowable Stress

Type of Steel pipe pile	Allowable short-term stress		Allowable long-term stress	
	Shearing	Bending	Shearing	Bending
SKK400 (or equal material)	120	210	80	140
SKK490 (or equal material)	160	280	105	185
SM570 (or equal material)	220	380	145	255
Elastic modulus of steel material		E = 200,000 N/mm^2		

Unit: N/mm^2

2) Determination of location of piles

Since the road to be protected is located at the toe of the landslide block, the piles are thus determined to be installed immediately adjacent to the road on the mountain side of. The piles are positioned roughly parallel to the direction of the landslide movement, namely the direction of the greatest inclination of the landslide slope.

3) Determination of pile design method

The soil mass behind the piles is thin and weak, and no reaction of the ground behind the piles can be expected, the piles are thus designed as bending (pressure) piles.

Accordingly, the design pile horizontal load in unit width, H, is calculated as follows:

$H=P_R\times\cos\theta=605\times\cos10=596$ kN/m

4) Determination of pile type

In order to determine the proper pile type, the following three types of piles with different diameters and thicknesses were initially selected and then compared as to cost and safety.

Table 10.22 Pile Types and their Features

Type of Pile	A	B	C
1. Pile diameter d (mm)	500.0	550.0	608.0
2. Pile thickness t (mm)	70.0	55.0	45.0
3. Cross-sectional area of pile A (cm^2)	945.6	855.3	795.9
4. Geometrical moment of inertia I (cm^4)	224348.2	265196.0	317368.1
5. Section modulus Z (cm^3)	8973.9	9643.5	10439.7
6. Allowable bending stress σ_a (N/mm^2)	380.0	380.0	380.0
7. Allowable shearing stress τ_a (N/mm^2)	220.0	220.0	220.0

5) Calculation of modulus of deformation of stationary ground in lateral direction

The modulus of deformation of stationary ground, Es2, is calculated using the following equations and the calculated result is shown in the table below.

$$E_s = K_h \times d \quad \cdots\cdots (10.1)$$

$$K_h = \left\{ \frac{(\alpha \times E_o)^{32}}{30^8 (4EI)^3 \times d^9} \right\}^{1/29} \quad \cdots\cdots (10.2)$$

Where,

Es (N/cm^2)=Modulus of deformation of ground

Kh (N/cm^3)=Coefficient of horizontal subgrade reaction of ground

d (mm)=pile diameter

α=Coefficient (α =1.0, depending on the estimating method for *Eo*)

Eo (N/cm^2)=Modulus of deformation of ground at design point of pile

Eo=8×N (kgf/cm^2) (N: Value of standard penetration test)

EI (Ncm^2)=Rigidity of steel pile to bending

Table 10.23 Coefficient of Horizontal Subgrade Reaction and Modulus of Deformation

Type of Pile	A	B	C
1. Coefficient of horizontal subgrade reaction *Kh2* (N/cm^3)	185.86	177.35	168.75
2. Modulus of deformation *Es2* (N/cm^2)	9292.98	9754.09	10259.97

6) Calculation of β value of pile

The β value of the pile is calculated using the following equations and the calculated result is shown in the table below.

$$\beta = \sqrt[4]{\frac{K_h \times d}{4 \times EI}} \quad \cdots\cdots (10.3)$$

Table 10.24 β Value of Pile

Type of Pile	A	B	C
β value of pile (m^{-1})	0.477	0.463	0.448

7) Sectional calculation of pile

The maximum bending moment acting on a pile, Mmax, and maximum shear force, Smax, were calculated using the following respective equations.

$$M_{\max} = -\frac{H}{2\beta}\sqrt{(1+2\alpha \times \beta \times l_e)^2 + 1} \times \exp\left[-\tan^{-1}\frac{1}{1+2\alpha \times \beta \times l_e}\right] \quad \cdots\cdots (10.4)$$

$$S_{\max} = H\sqrt{2(\alpha \times \beta \times l_e)^2 + 2\alpha \times \beta \times l_e + 1} \times \exp\left[-\tan^{-1}\frac{\alpha \times \beta \times l_e + 1}{\alpha \times \beta \times l_e}\right] \cdots\cdots\cdots\cdots (10.5)$$

Where,

H (N/cm)=Pile horizontal load in unit width

le (m)=Effective length of pile

α (mm)=Ratio of the height between point of action and sliding surface and the effective length of pile, in general, α=1/3 to 1/2.

Moreover, the depths of the points at which maximum bending moment occurs from sliding surface, *lm*, and at which maximum shear stress occurs from sliding surface, *ls*, are calculated according to the following equations.

$$l_m = -\frac{1}{\beta}\tan^{-1}\frac{1}{1 + 2\alpha \times \beta \times l_e} \cdots\cdots (10.6)$$

$$l_S = \frac{1}{\beta} \times \tan^{-1}\frac{1 + \alpha \times \beta \times l_e}{\alpha \times \beta \times l_e} \cdots\cdots (10.7)$$

The calculated results are shown in the table below.

Table 10.25 Sectional Calculation Results

Type of Pile	A	B	C
1. Horizontal load of pile in unit width H (kN/m)	1098.0	1098.0	1098.0
2. Effective length *le* (m)	10.0	10.0	10.0
3. Ratio of the distance between point of action and the sliding surface to the effective length of pile α	1/3	1/3	1/3
4. Maximum bending moment M_{max} (kN・m)	-2122.45	-2129.43	-2137.39
5. Depth of maximum bending moment *lm* (m)	0.49	0.52	0.55
6. Maximum shear force S_{max} (kN・m)	652.82	635.80	617.91
7. Depth of maximum shear force *ls* (m)	2.14	2.21	2.30
8. Allowable bending moment Ma=$\sigma_a \times Z$ (kN・m)	3410.09	3664.53	3967.10
9. Allowable shearing force Sa=$\tau_a \times A$ (kN)	20803.63	18816.57	17510.29

8) Determination of pile intervals

The interval between the piles, D, is generally obtained by dividing the preventive force (P_R) required to satisfy the proposed safety factor by the design power force of the piles. Also, the interval must be such as to prevent soil from moving between the piles and the failure of the foundation.

The following table lists some factors in determining the intervals of the piles, together with the calculated results.

Table 10.26 Determination of Pile Intervals

Type of Pile	A	B	C
1. Regarding bending moment, $D=\sigma_a \times Z / M_{max}$ (m)	1.60	1.72	1.85
2. Regarding shearing, $D=\tau_a \times A / Smax$ (m)	31.87	29.60	28.34
3. Regarding the thickness of the sliding mass*	≦2.0	≦2.0	≦2.0
4. Regarding the diameter of the pile, D≦8d	≦ 4.00	≦ 4.40	≦ 4.86
5. Regarding the diameter (d_b) of core drilling $D \geqq 1+d_b$ (m)	1.55	1.70	1.70
6. Final determination of pile intervals	**1.60**	**1.70**	**1.80**

Note: Refer to Chapter 5 of this Guide III

9) Determination of embedment depth of piles

In designing bending (pressure) piles, the embedment depth of the piles, *lr*, is generally determined using the following equation, in view of the bearing capacity of stationary ground. If the ground is hard rock, embedment depth of piles is shorter; otherwise it is longer.

$$l_r = (1.0 \sim 1.5)\frac{\pi}{\beta} \quad \cdots\cdots (10.8)$$

At this landslide site, the stationary ground is hard sandstone, and therefore, the embedment length of the piles is obtained by the equation below and the calculated result is shown in Table 10.27.

$$l_r = \frac{\pi}{\beta} \quad \cdots\cdots (10.9)$$

Table 10.27 Determination of Embedment Depth

Type of Pile	A	B	C
1. β (m^{-1})	0.477	0.463	0.448
2. $lr = \pi / \beta$ (m)	6.59	6.78	7.01
3. $lr = 1.5\pi / \beta$ (m)	9.88	10.18	10.51
4. Determination of embedment depth (m)	**7.00**	**7.00**	**7.50**

10) Economic comparison of pile types

In order to select cost-effective pile types, the above calculated three types of piles were compared in terms of the cost of steel material and the cost of core drilling for installing the steel pipe piles, as shown in Table 10.28.

Table 10.28 Economic Comparison of the Piles

Type of Pile	A	B	C
1. Effective length (thickness of sliding mass) (m)	10.0	10.0	10.0
2. Embedment depth of pile (m)	7.0	7.0	7.5
3. Length of one pile (m)	17.0	17.0	17.5
4. Number of piles (nos.)	81	76	72
5. Total length of piles (m)	1377.0	1292.0	1260.0
6. Diameter of pile (mm)	500.0	550.0	608.0
6. Diameter of core drilling (mm)	550.0	700.0	700.0
7. Volume of grout material (m^3)	196.9	386.7	384.6
8. Weight of steel material in unit length (kg/m)	742.3	671.4	624.8
9. Total weight of steel material (ton)	1022.1	867.4	787.2
10. Unit price of core drilling (peso/m)	3000	5000	5000
11. Unit price of steel material (peso/ton)	41170	41170	41170
12. Unit price of grout material (peso/m^3)	4500	4500	4500
13. Total cost of core drilling (×1000 peso)	4131.0	6460.0	6300.0
14. Total cost of steel material (×1000 peso)	42079.9	35710.9	32409.0
15. Total cost of grout material (×1000 peso)	886.1	1740.2	1730.7
16. Total direct construction cost (×1000 peso)	47097.0	43911.1	40439.7

11) Details of steel pipe pile work

On the basis of the above calculation and comparison, the final steel pipe pile is determined and summarized in Table 10.29.

Table 10.29 Summary of Steel Pipe Pile Work

Item	Specification	Remarks
1. Pile material	SM570 (or equal material)	
2. Diameter of pile	608 mm	
3. Thickness of piles	45 mm	
4. Horizontal intervals of piles	1.8 m	
5. Diameter of core drilling	550 mm	
6. Embedment depth of pile	7.5 m	
6. Length of one pile	17.5 m	
7. Number of piles	72 nos	
7. Total length of piles	1,260 m	

Moreover, the above selection was based only on a simple comparison of construction cost and the effectiveness of piles. Because the landslide slope is underlain by gravelly soil with some boulders and cobbles, it is a little difficult to implement larger diameter core drilling. Accordingly, from the viewpoint of ease of construction and installation, small-diameter piles combined with anchors should be considered.

(c) Design of Fill (Counterweight Embankment) for Option-2

Fill as a permanent measure is designed to increase the resisting force against the landslide thrust by piling up earth materials on the toe of the landslide area. On the basis of the results of stability analysis, an embankment 3 m high is required to achieve the target safety factor (PFs=1.20).

The embankment gradient is set at 1:2.0, and the foot of the embankment is protected with 2 m high gabions. Because of the large amount of spring water along the toe of the landslide, filter material consisting of sand and gravel, about 20 cm to 50 cm wide, is placed along the mountain side in order to discharge the seepage and spring water.

Figure 10.43 and Figure 10.44 shows the details of the filling.