No.

Capital District of Bogotá and Soacha Municipality
The Republic of Colombia

THE STUDY ON MONITORING AND EARLY WARNING SYSTEM FOR LANDSLIDES AND FLOODS IN SELECTED AREAS IN THE CAPITAL DISTRICT OF BOGOTÁ AND SOACHA MUNICIPALITY IN THE REPUBLIC OF COLOMBIA

FINAL REPORT

VOLUME 4 DATA BOOK 3

TEMPORARY WORKS AND COMMUNITY WORKSHOP

MARCH 2008

JAPAN INTERNATIONAL COOPERATION AGENCY

PACIFIC CONSULTANTS INTERNATIONAL
OYO INTERNATIONAL CORPORATION

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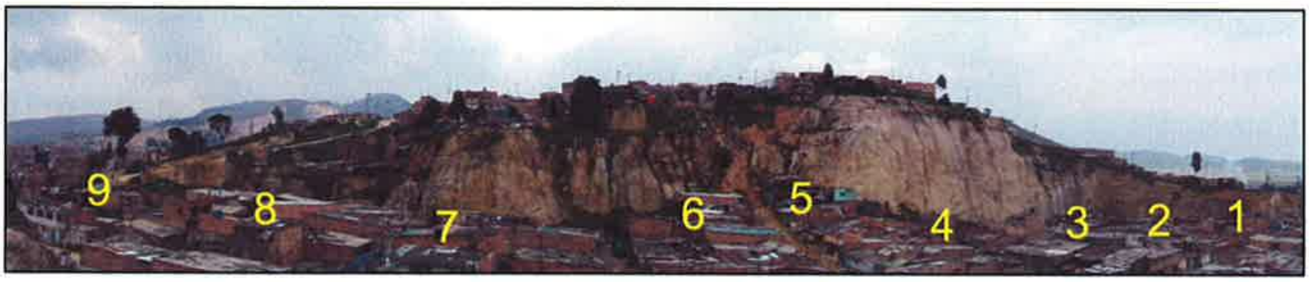
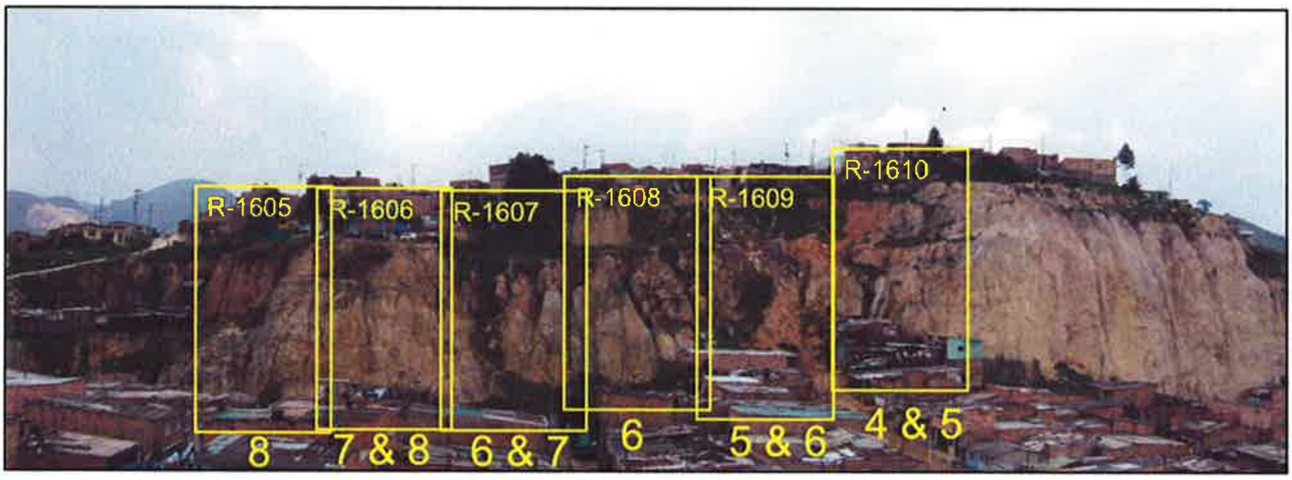
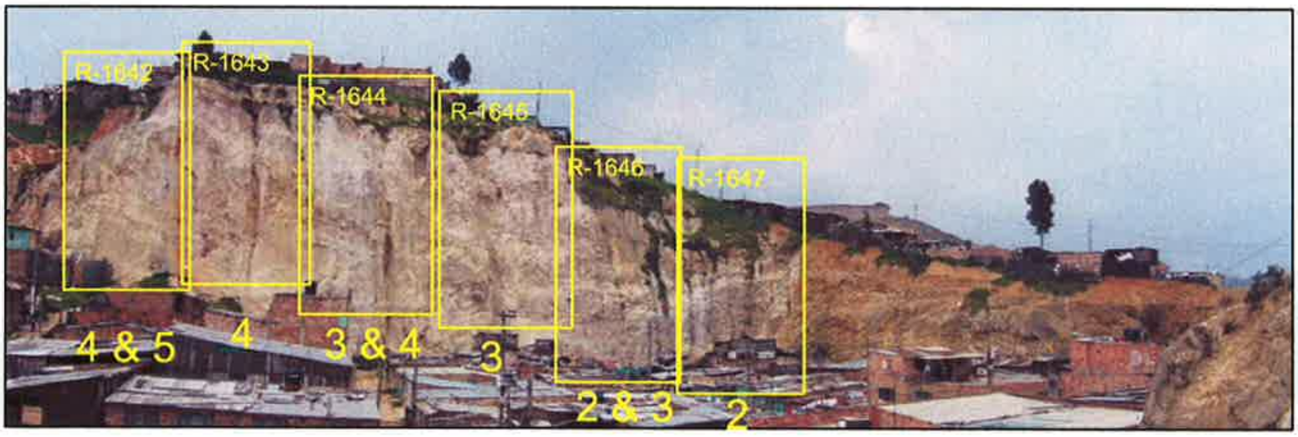
D3.1 STEREO PICTURES OF STEEP SLOPE IN DIVINO NINO



LOCATION OF THE RESIDENTIAL BLOCKS IN FRONT OF THE SLOPE



LOCATION OF PHOTOS (LEFT SIDE)



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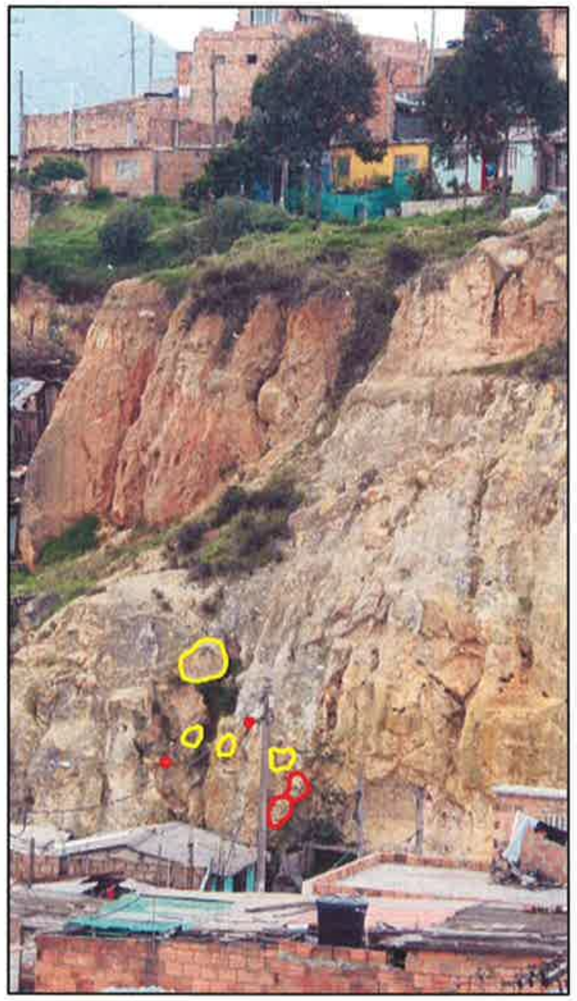
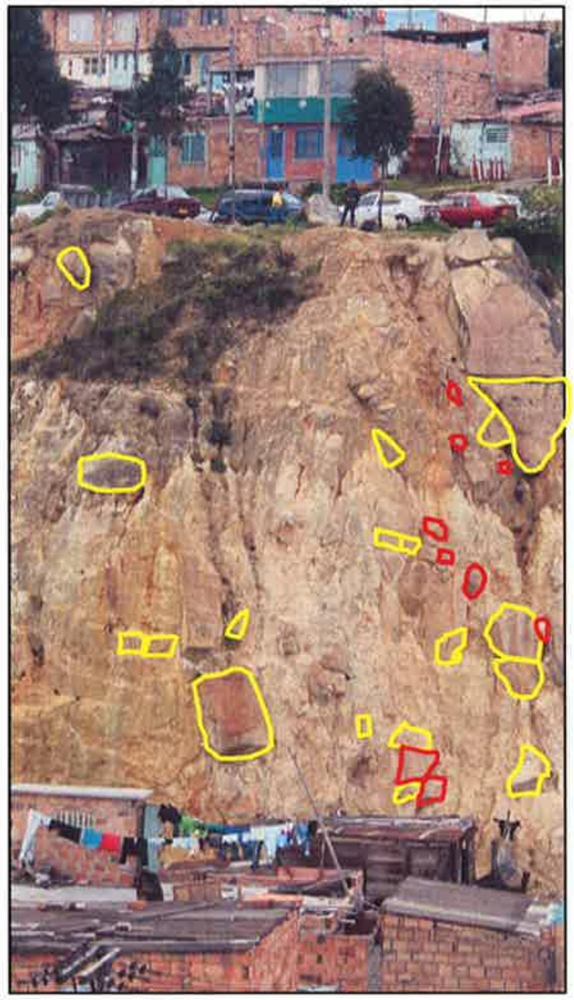
R-1605	R-1606	R-1607



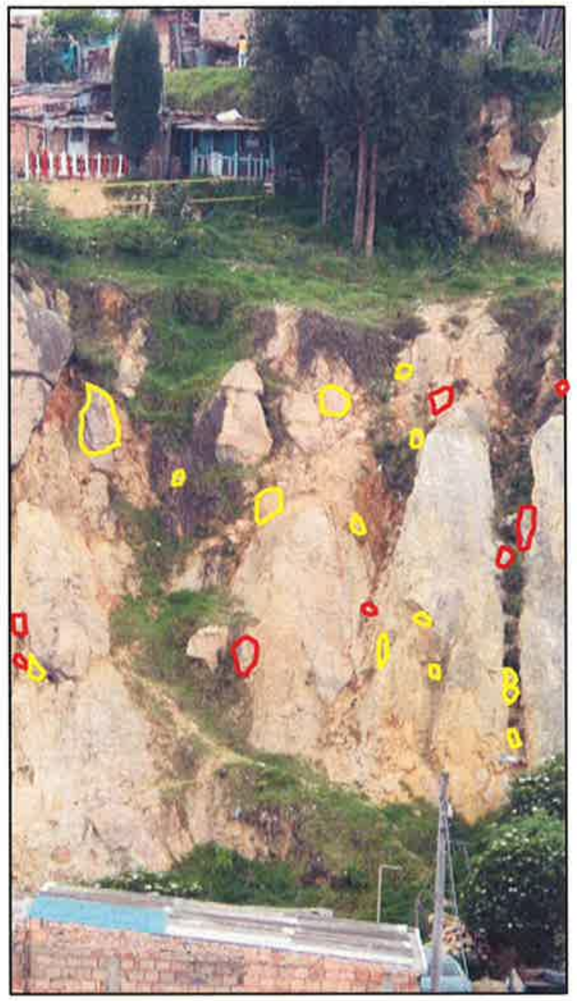
MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
8	A	4
	B	4



MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
7 & 8	A	9
	B	19



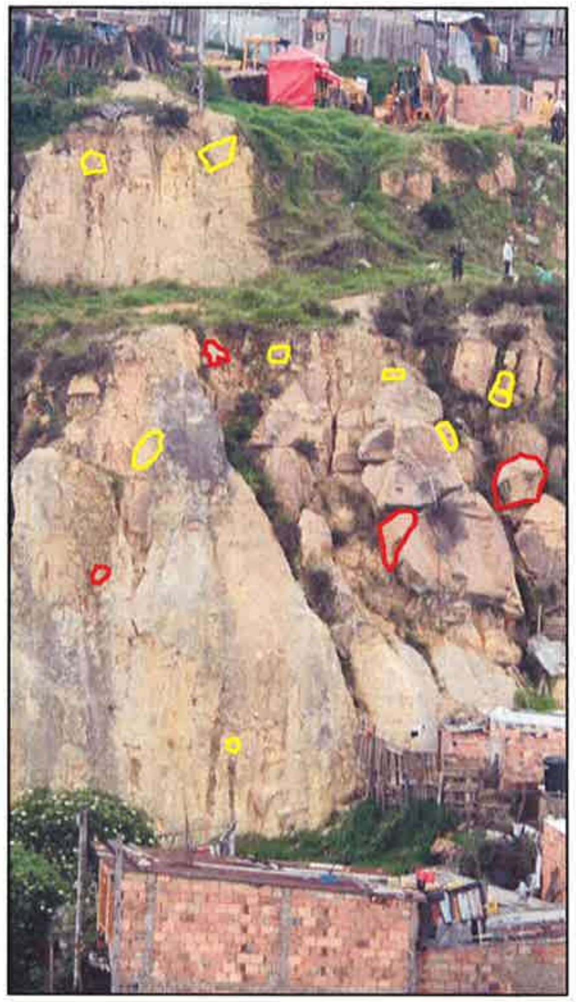
MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
6 & 7	A	8
	B	14

TYPE A: (Red)
High Possibility of the Rocks which may fall in near future

TYPE B: (Yellow)
A few possibility of Rocks which may fall in the future

R-1608	R-1609	R-1610

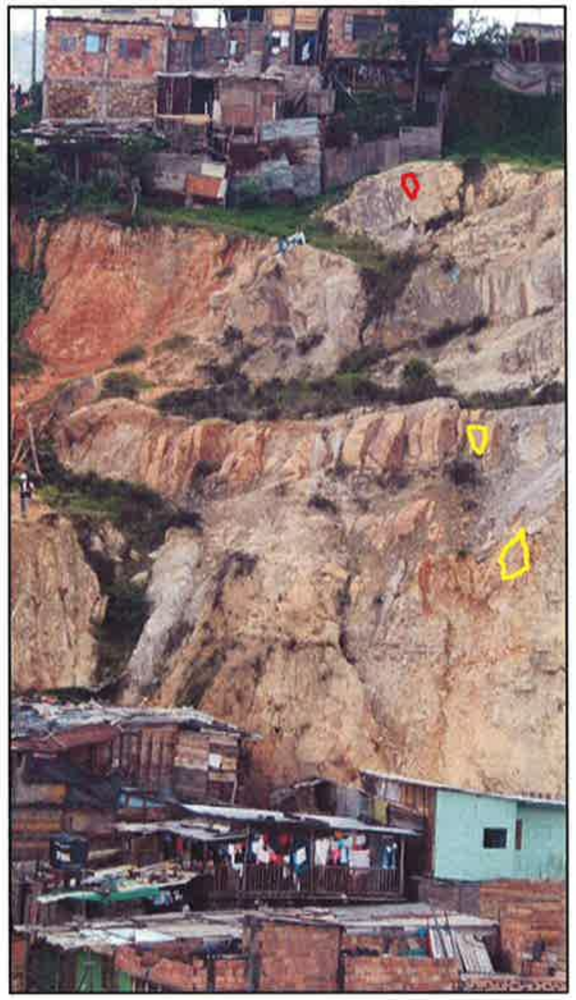


MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
6	A	4
	B	9

MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
5 & 6	A	2
	B	6



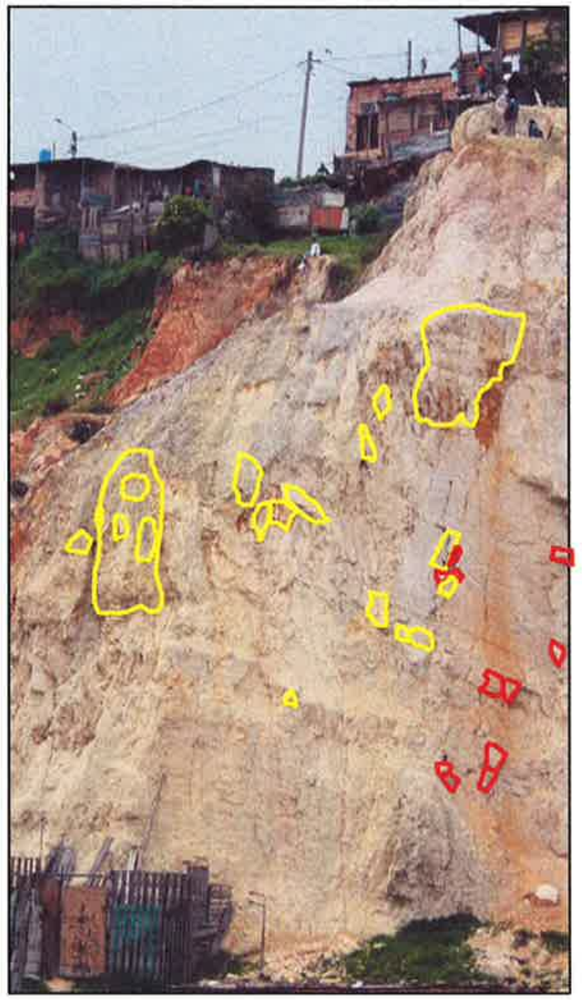
MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
4 & 5	A	1
	B	2

TYPE A: (Red)
High Possibility of the Rocks which may fall in near future

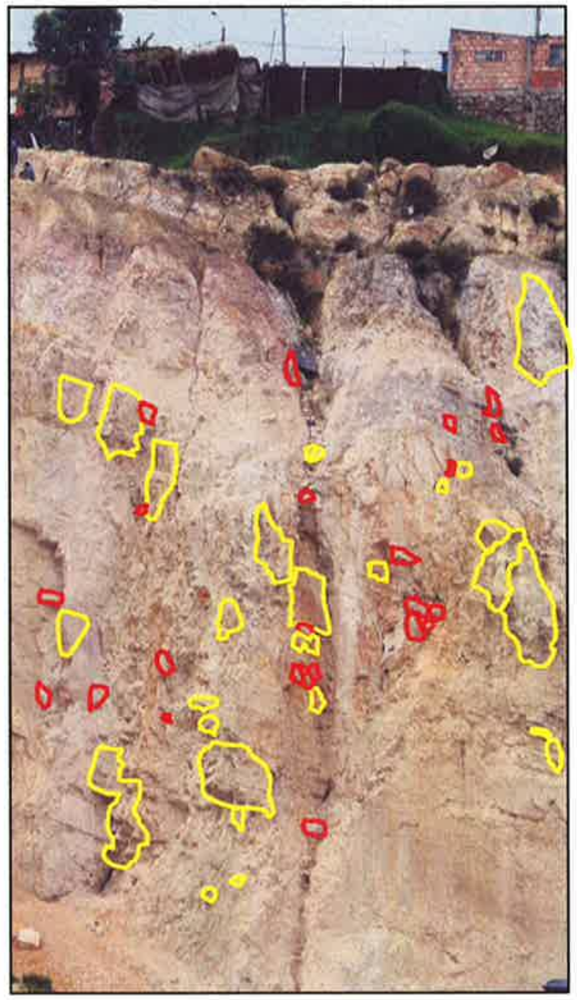
TYPE B: (Yellow)
A few possibility of Rocks which may fall in the future

R-1642	R-1643	R-1644



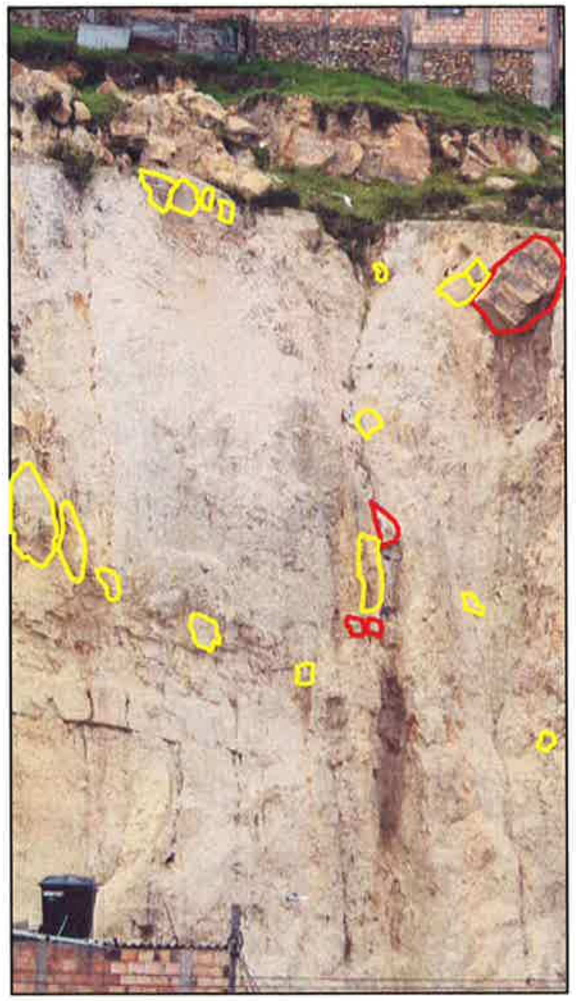
MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
4 & 5	A	11
	B	19



MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
4	A	23
	B	27



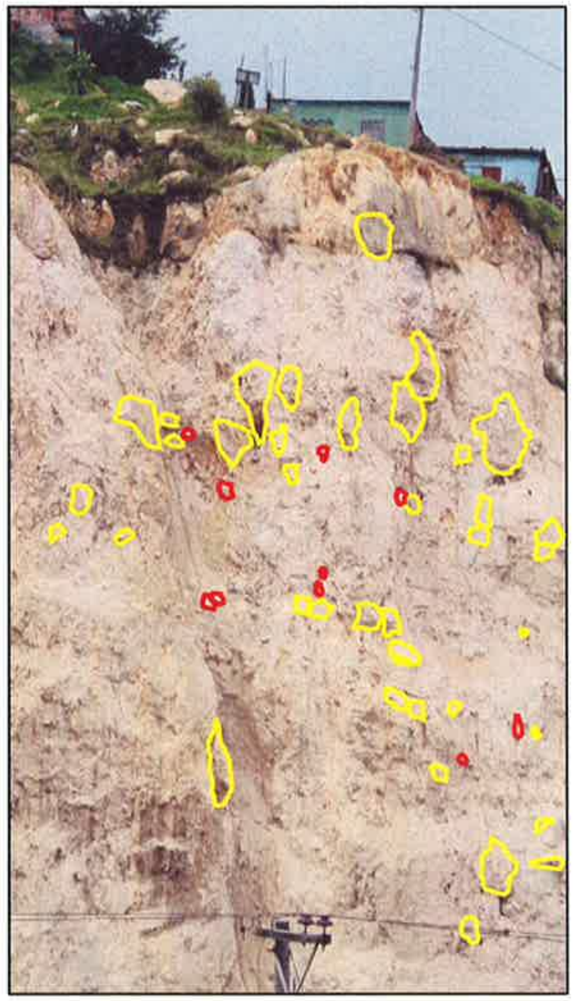
MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
3 & 4	A	4
	B	16

TYPE A: (Red)
High Possibility of the Rocks which may fall in near future

TYPE B: (Yellow)
A few possibility of Rocks which may fall in the future

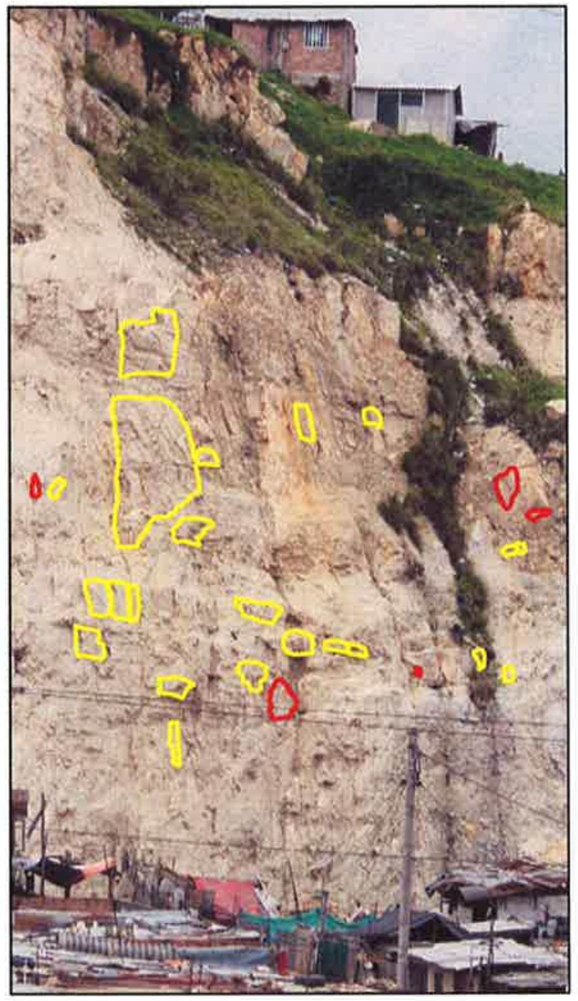
R-1645



MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
3	A	10
	B	39

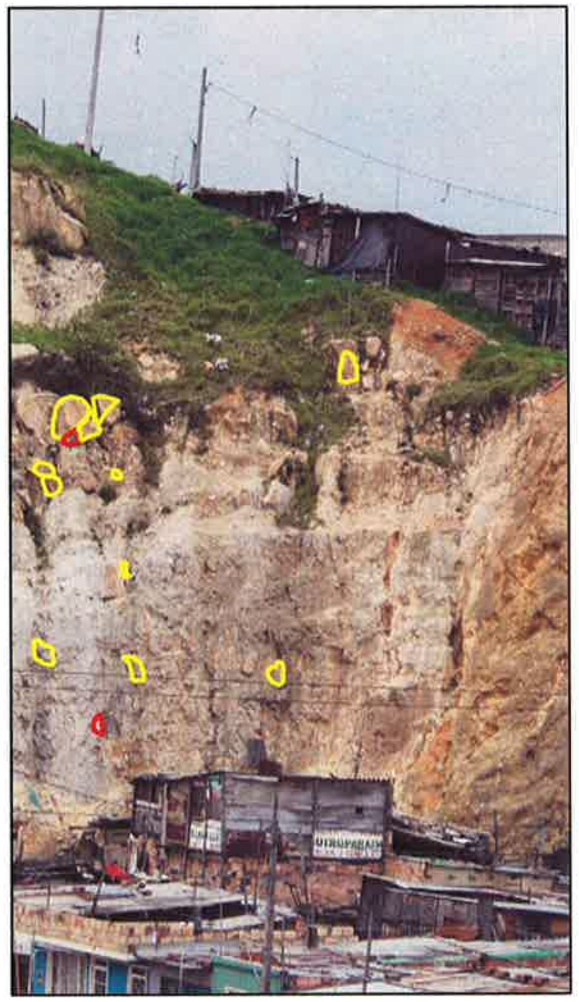
R-1646



MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
2 & 3	A	5
	B	22

R-1647



MARKED ROCKS

AFFECTED RESIDENTIAL BLOCKS	TYPE	QUANTITY
2	A	2
	B	11

TYPE A: (Red)
High Possibility of the Rocks which may fall in near future

TYPE B: (Yellow)
A few possibility of Rocks which may fall in the future

D3.2 STABILITY ANALYSIS OF THE TEMPORARY PROTECTION WALL

STABILITY ANALYSIS OF THE TEMPORARY PROTECTION WALL

1. General

The gabion wall was recommended as a temporary protection wall for small-scale falling rock from the slope. And the wall is installed after relocation program including the demolition works of existing houses in the emergency zone in Divino Nino area.

As to the material, gabion net of 1.0m (width) x 1.0m (height) x 4.0m (length) will be used due to easy procurement in the area. According to the bouncing height of falling rocks in empirical knowledge and the rock size (D_{100}), the wall consisted in three (3) layers of gabion were required as shown in Figure 1-1. In order to minimize transportation cost of the debris produced in the demolition works, the debris which is available for construction material is utilized to gabion block “g02” located in center of the wall and sand mat behind of the wall.

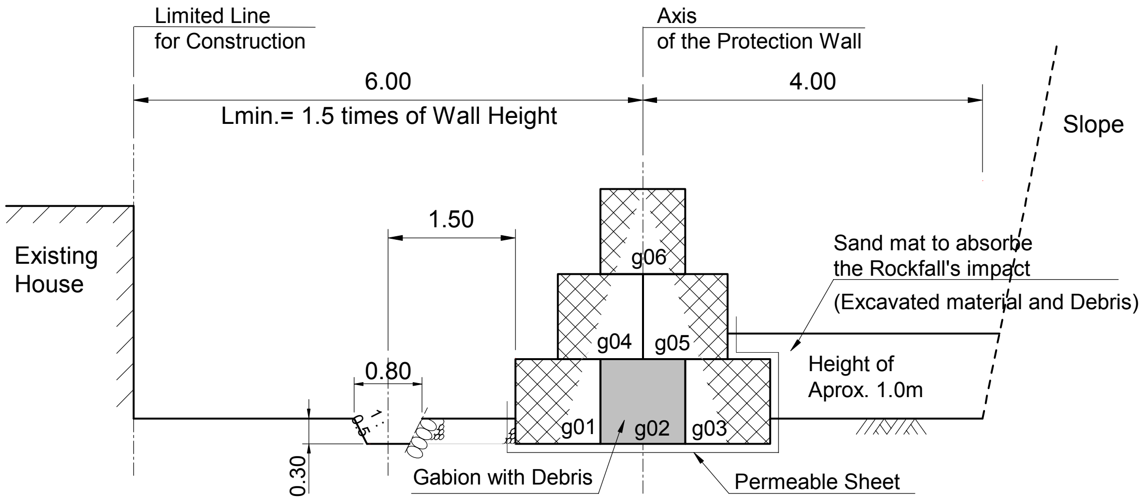


Figure 1- 1 Dimensions of the Gabion Wall

2. Method of Stability Analysis

2.1 Present Conditions of the Slope

There are nine (9) residential blocks in the emergency zone in Divino Nino area. From the topographic survey conducted in December 2006 to confirm the slope conditions behind of the residential blocks, the slope angles and height varied from 39 to 75 degrees and 7 to 36m respectively as shown in Table 2-1.

Table 2- 1 Conditions of the Slope

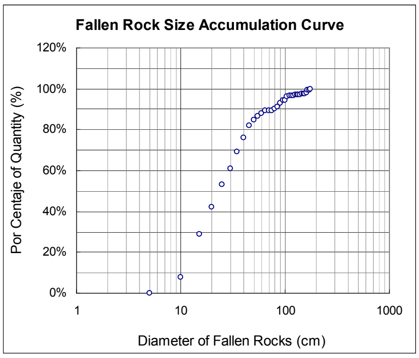
No. Perfiles	No.1	No.2	No.3	No.4	No.5	No.6	No.7	No.8	No.9	No.10
No. Residential Block	9	8	7	6	5	4	3	2 & 3	2	1
Slope Height: H	7.2	8.6	22.0	18.9	10.6	29.1	33.8	35.9	16.7	14.6
Slope Angle: Θ	53.0°	39.2°	65.7°	58.2°	56.5°	68.4°	66.6°	72.3°	71.5°	75.1°

(Source: JICA Study Team)

As to the rocks on the slope behind the residential blocks, 271 rocks in total which may fall in the future were confirmed using the stereo pictures prepared by the Study Team. In the measurement of existing rockfalls, measured existing rocks were converted to the spherical form using formula 2.1.1 and following size distribution of rock are given.

$$W = \gamma \cdot \pi \cdot D^3 / 6 \quad \cdots\cdots 2.1.1$$

Where, W: Mass of the material (kN), γ: Specific mass of the material (kN/m³), π: Ratio of the circumference of a circle to its diameter (m), D: Diameter of material (m)



Distribution	Diameter (cm)
D_{50}	20
D_{60}	25
D_{70}	35
D_{80}	40
D_{90}	80
D_{100}	175

Figure 2- 1 Size Distribution of the Existing Rockfalls

2.2 Method of Stability Analysis of the Temporary Protection Wall

(1) Concept of the Stability Analysis (Resistible Capacity of the Temporary Protection Wall)

The force of a falling rock which menaces the stability of protection wall is expressed by the multiplication of rock mass (*m*) and falling velocity (V_0).

The velocity of a falling rock to be applied for the wall stability analysis is controlled by slope height as following formula 2.3.1. And the rotating velocity of the wall after receiving the force of a falling rock is estimated using velocity (V_0) as shown in the formula 2.3.2.

The velocity (V_0) by this equation is materialized on the assumption that rocks fall directory to the slope toe and the slope conditions of 1) roughness of the slope surface, 2) protuberances on the slope and slope angles are not considered.

$$V_0 = \sqrt{2 \cdot g \cdot H} \quad \cdots\cdots 2.2.1$$

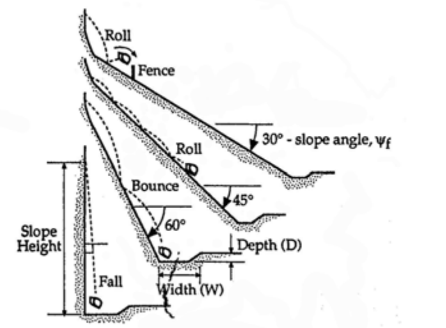
Where, *V_0: Falling velocity of the rocks (m/sec), g: Gravitational acceleration (m/s^2), H: Slope height (m)*

$$V = \frac{2 \cdot m'}{m' + \alpha' \cdot m'} \cdot V_0 \quad \cdots\cdots 2.2.2$$

Where, *V: Rotation velocity of the wall (m/sec), m': Weight of the wall (ton), α': coefficient of the wall form and the arm length from the center of foundation spring, V_0: Falling velocity of the rocks (m/sec)*

As shown in Table 2-2, the movement of a falling rock on the slope is controlled by its slope angles, and the actual falling velocity of rock should be reduced through the bouncing and spin due to the slope conditions. Even if the slope height which increases the falling velocity becomes higher, the falling velocity does not always increase in proportion to its slope height due to slope conditions. Therefore the velocity estimated by this equation is overestimated for the temporary protection wall in the viewpoint of its objective.

Table 2- 2 Falling Condition of the Rocks according to the Slope Angles

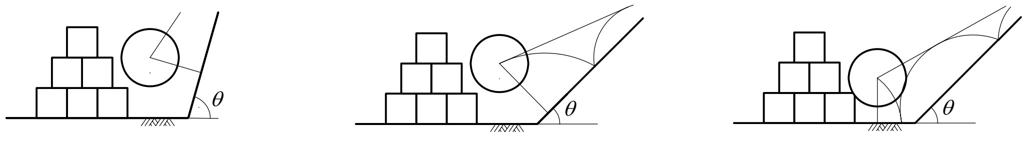
Slope Angles θ	Falling condition of Rocks	Falling Conditions by the slope angles
$75^o < \theta$	The rocks tend to stay close to the face and land near the toe of the slope.	
$55^o < \theta < 75^o$	The rocks tend to bounce and spin, with result that they can land a considerable distance from the toe and a wide ditch is required.	
$40^o < \theta < 55^o$	The rocks will tend to roll down the face and into the ditch and a steep outer face is required to prevent them from rolling out.	

(Source: Landslides Investigation and Mitigation Special Report 247, Transportation Research Board National Research Council, National Academy Press Washington, D.C. 1996)

Since the slope conditions can not be considered to the analysis of falling velocity, it is not suitable for the temporary protection wall to carry out the stability analysis of the wall based on the targeted rock size and slope height. Therefore the resistible capacity of the temporary protection wall is evaluated using several rock size and slope height.

(2) Method of Stability Analysis

As mentioned above, a) direct colliding condition as shown in Figure 2-2 is selected for the stability analysis. Furthermore the function of sand-mat installed behind the wall is not considered.

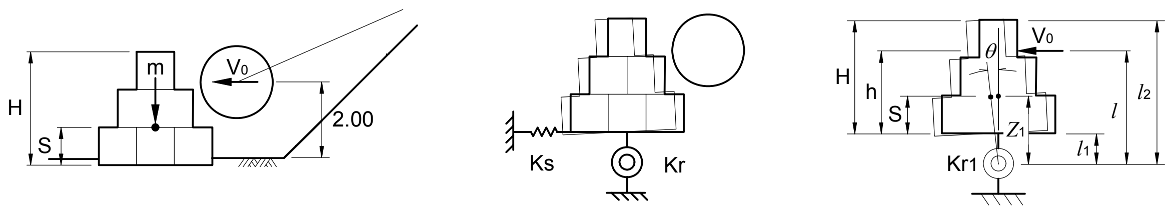


a) Direct Colliding b) Indirect Colliding-1 c) Indirect Colliding-2

Figure 2- 2 Colliding Conditions of a Falling Rock

The gabion wall belongs to a flexible body which deforms according to the conditions of the foundation, In this design works, assuming that the gabion wall is a rigid body as shown in Figure 2-3, the model for analysis is stated as follows.

- the protection wall is assumed a rigid body supported by the elastic foundation consisted of 1) share and 2) rotation spring
- the kinetic energy produced by a falling rock should be transmitted to the wall body
- the wall intends to move and rotate until that the transmitted kinetic energy becomes equivalent to the deformation energy produced by an elastic response of its foundation
- the stability of the wall is secured when the deformation energy produced by an elastic response of its foundation is less than the possible absorbing energy determined by admissible displacement of the wall.



1) Model of Loads 2) Structural Model 3) Movement model of the Wall

Figure 2- 3 Structural Model of the Temporary Protection Wall

The conditions for the stability analysis are described as follows.

i) One (1) falling rock which collides to the wall is considered for the stability analysis.
ii) Effective length of the wall for the wall stabilization is considered four (4) times of wall height (4H). If the wall length is shorter than 4H, actual wall length should be considered.
iii) Force of a falling rock acts horizontally to the wall
iv) Height of the wall from the ground where a falling rock collides is considered design bouncing height (= 2.0m)

(3) Analysis Flow

As mentioned above, the calculation of required items for stability analysis is shown in Figure 2-4.

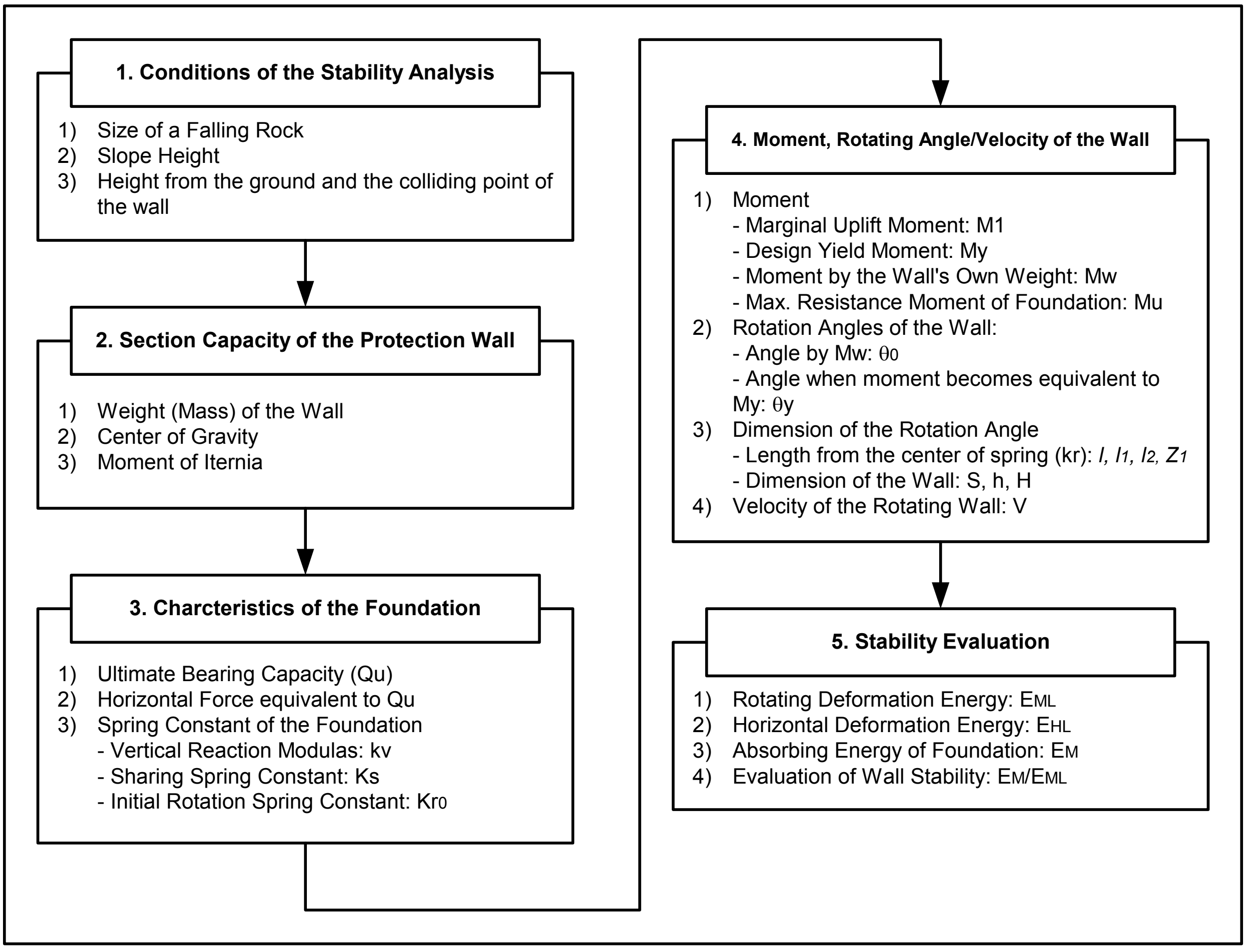


Figure 2- 4 Flow of the Stability Calculation

3. Stability Analysis of the Temporary Protection Wall (Gabion Wall)

In order to confirm the resistible capacity of the temporary protection wall, 98 cases of following combination of items were carried out. Furthermore a falling rock collides directory with the wall without bouncing and/or spins on the slope, and the function of sand-mat installed behind of the wall is not considered in this analysis.

- Rock size: from 50 to 180cm every 10cm
- Slope height: from 10 to 40m every 5m

From the result of the analysis, the temporary protection wall is available within the following conditions, and the wall endures satisfactory in case of a falling rock classified (D_{90}).

Table 3- 1 Result of the Stability Analysis

Slope Height (m)	10	15	20	25	30	35	40
Rock size (m)	1.3	1.2	1.1	1.1	1.0	1.0	1.0

As an example, in case that the rock with a diameter of 80cm (D90) on the slope of 40m height collides directory with the protection wall, the stability analysis of the protection wall is shown as follow.

3.1 Design Criteria

(1) Design Criteria

1) Specific Weight of the Materials to be used

Specific mass of the material to be used for the temporary protection wall is shown in Table 3-1.

Table 3- 2 Specific Mass of the Material

Material	**Mass (kN/m^3)**	**Material**	**Mass (kN/m^3)**
1. Rock/Stone	26.5	5. Concrete	23.0
2. Cobble Stone	18.0	6. Reinforced Concrete	25.0
3. Sand	18.0	7. Gabion*	18.6
4. Bricks	14.0		

Note: The mass of gabion considers 70% porosity of rock mass.

2) Characteristics of the Foundation

Assuming the soil of the foundation is classified the silty sand, the characteristics of the foundation for design are settled as follows:

Specific mass of the soil γ:	18.0 kN/m^3
Estimated N Value N:	30
Angle of sharing resistance Ø:	30^o
Cohesion c (= 12.5N):	10 kN/m^2

(2) Loads to be considered

1) Dead load

Only weight of the wall body is considered as a dead load. Horizontal pressure of sand-mat behind of the wall is not considered because of that the sand-mat is installed without compacting.

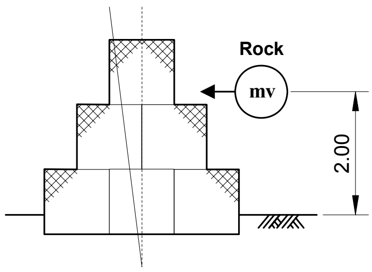
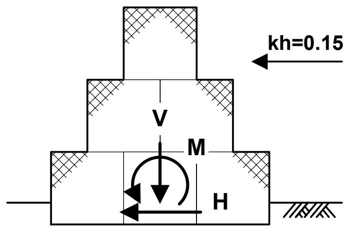
2) Live load

Live load and surcharge load which act to the protection wall are not considered due to its objective.

(3) Combination of the Loads

Load combination for the stability analysis of the wall is shown in Table 3-2. As to the seismic conditions, since the horizontal force produced by the seismic coefficient (kh) is too small, the analysis in this condition is omitted.

Table 3- 3 Combination of the Loads

Item	Normal Condition	Seismic Condition
1. Condition		
2. Loads (1) Dead Load	O	O
(2) Seismic Load	-	kh = 0.15
(3) Live Load	-	-
(4) Earth Pressure*	-	-
(5) Force of Falling Rock	1) Dia.: 50 ~ 180cm 2) Collision Position: 2.30m	-
3. Stability Conditions (1) Overturning	-	$e = 1/3\,B$ (B: Width of Wall)
(2) Sliding	δa = 0.05m	Fs = ΣV/ΣH = 1.2
(3) Bearing Capacity		$Q_{max} \le Q_a$
(4) Rotation Angle	θa = 3^{o}π/180 = 0.052rad	-

*Note: *Earth pressure is from sand mat in space between wall and slope.*

3.2 Section Capacity of Gabion Wall

Section capacity of the gabion wall is estimated by following formula.

i) Area:

$$A = \frac{1}{2}\Sigma\left(x_{i+1}\cdot y_i - x_i \cdot y_{i+1}\right) \cdots\cdots 3.2.1$$

ii) Geometrical Moment of Area:

$$Gy = \frac{1}{2}\Sigma(y_{i+1} - y_i)\left\{x_i^2 + \frac{1}{3}(x_{i+1} - x_i)\cdot(x_{i+1} + 2x_i)\right\} \quad \cdots 3.2.2$$

$$Gx = \frac{1}{2}\Sigma(x_{i+1} - x_i)\left\{y_i^2 + \frac{1}{3}(y_{i+1} - y_i)\cdot(y_{i+1} + 2y_i)\right\} \quad \cdots 3.2.3$$

$$x_G = \frac{G_y}{A} \quad \cdots 3.2.4$$

$$y_G = \frac{G_x}{A} \quad \cdots 3.2.5$$

iii) Geometrical Moment of Inertia:

$$Ix = \frac{1}{3}\Sigma(x_{i+1} - x_i)\left\{y_i^3 + \frac{3}{2}y_i^2(y_{i+1} - y_i) + y_i(y_{i+1} - y_i)^2 + \frac{1}{4}(y_{i+1} - y_i)^3\right\} \quad \cdots 3.2.6$$

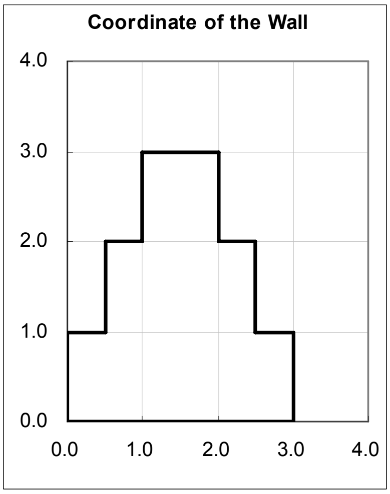
$$Iy = -\frac{1}{3}\Sigma(y_{i+1} - y_i)\left\{x_i^3 + \frac{1}{6}(x_{i+1} - x_i)\cdot(x_{i+1} + 2x_i)^2 + \frac{1}{12}(x_{i+1} - x_i)^3\right\} \quad \cdots 3.2.7$$

iv) Moment of Inertia

$$I_G = M\left(\frac{I_x + I_y}{A} - x_G^2 - y_G^2\right) \quad \cdots 3.2.8$$

Table 3- 4 Geometrical Moment of the Gabion Wall

No. *i*	x (m)	y (m)	A (m^2)	Gy (m^3)	Gx (m^3)	Iy (m^4)	Ix (m^4)
0	0.00	0.00	0.00	0.00	0.00	0.000	0.000
1	0.00	1.00	0.25	0.00	0.25	0.000	0.167
2	0.50	1.00	-0.25	-0.13	0.00	-0.042	0.000
3	0.50	2.00	0.50	0.00	1.00	0.000	1.333
4	1.00	2.00	-0.50	-0.50	0.00	-0.333	0.000
5	1.00	3.00	1.50	0.00	4.50	0.000	9.000
6	2.00	3.00	1.00	2.00	0.00	2.667	0.000
7	2.00	2.00	0.50	0.00	1.00	0.000	1.333
8	2.50	2.00	1.25	3.13	0.00	5.208	0.000
9	2.50	1.00	0.25	0.00	0.25	0.000	0.167
10	3.00	1.00	1.50	4.50	0.00	9.000	0.000
11	3.00	0.00	0.00	0.00	0.00	0.000	0.000
12	0.00	0.00	0.00	0.00	0.00	0.000	0.000
Total			**6.00**	**9.00**	**7.00**	**16.50**	**12.00**



From Table 3-2-1, the section capacity of gabion wall is shown as follows.

i) Total weight of the wall:

$$m = A \cdot \gamma \cdot L = 6.00 \times 18.6 \times 12.00 = 1{,}335.6kN$$

ii) Mass of the wall:

$$m' = m/g = 1{,}335.6/9.8 = 136.29t$$

iii) Center of gravity:

$$xG = Gy/A = 9.00/6.00 = 1.500m$$
$$yG = Gx/A = 7.00/6.00 = 1.167m = S$$

iv) The moment of inertia:

$$I_G = m'\cdot\left(\frac{Ix+Iy}{A} - x_G{}^2 - y_G{}^2\right)$$
$$= 136.29 \times \left(\frac{12.00+16.50}{6} - 1.500^2 - 1.167^2\right)$$
$$= 115.11tm = 1{,}520.1kNm$$

3.3 Specific Characteristics of the Foundation

As mentioned in 2.2 Method of Stability Analysis of the Temporary Protection Wall, the wall is assumed as a rigid body supported by share and rotating spring of its foundation.

The horizontal force produced by colliding of a falling rock is transmitted to the wall body, and the moment including wall own weight at the wall bottom is produced according to the intensity of the force. There is a point that the horizontal force which acts to the wall body becomes equal to ultimate bearing capacity of the foundation. The moment when the force reaches to ultimate bearing capacity is expressed as design yield moment.

In this section, the horizontal force which produces a design yield moment is estimated through the ultimate bearing capacity. Furthermore spring constant of the foundation is also estimated from the characteristics of the foundation.

1) Ultimate Bearing Capacity of the Foundation

Ultimate bearing capacity of the foundation is estimated using following formula which considers the eccentric slope of the loads (Horizontal force/Vertical weight).

$$Qu = Ae\left\{\alpha \cdot k \cdot c \cdot Nc + k \cdot q \cdot Nq + \frac{1}{2} \cdot \gamma_1 \cdot \beta \cdot Be \cdot Nr\right\} \cdots\cdots 3.2.9$$

Where, Qu: Ultimate bearing capacity considering eccentric load (kN), Ae: Effective area of wall footing considering eccentric load (m^2), α, β: Coefficient of the footing form (α = β = 1.0), k: Additional coefficient for footing depth efficacy (k = 1.0), c: Cohesion of the foundation (kN/m^2), q: Surcharge load (kN/m^2), γ_1, γ_2: Specific weight of foundation (kN/m^3), Be: Effective width of wall footing considering eccentric load (m), Nc, Nq, Nr: Bearing capacity factor considering eccentric load

Assuming that the horizontal force by a falling rock is **610kN**, the required items for the ultimate bearing capacity of the foundation is estimated as follows.

i) Eccentric Distance eB

$$e_B = B/2 - (m \cdot X_G - H_R \cdot h)/m = 3.00/2 - (1{,}335.6 \times 1.50 - 610.0 \times 2.30)/1{,}335.6 = 1.050m$$

Where, e_B: Eccentric distance (m), B: Width of retaining wall (m), m: mass of the wall (kN), XG: Gravity center of the wall (m), HR: Horizontal force of falling rock (kN), h: Height from the ground to the hit point of a falling rock (m)

ii) Effective Width of the Wall considering Eccentric load: Be

$$Be = B - 2e_B = 3.00 - 2 \times 1.050 = 0.900m$$

Where, Be: Effective width of the wall (m), B: Width of retaining wall (m), eB: Eccentric distance (m)

iii) Effective Area of the Wall Footing

$$Ae = Be \cdot L = 0.900 \times 12.00 = 10.80m^2$$

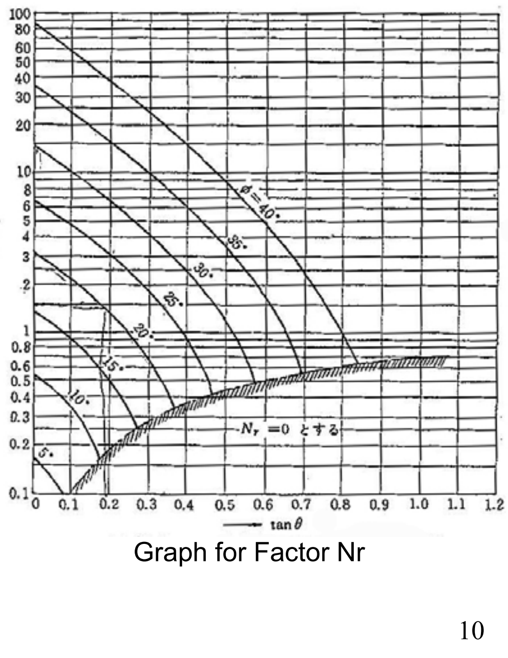
Where, Ae: Effective area of the wall footing (m^2), Be: Effective width of the wall (m), L: Effective length of the wall (m)

iv) Bearing Capacity Factor

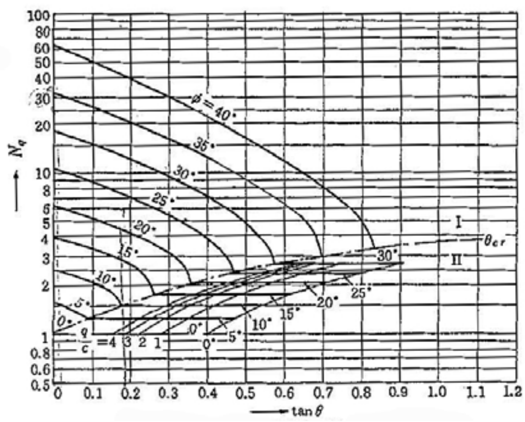
The factor of bearing capacity is estimated considering the eccentric slope of the loads. The angle of eccentric load is estimated as follow.

$$\tan\theta = H_R/m = 610.0/1{,}335.6 = 0.46 \quad (\theta = 24.55°)$$

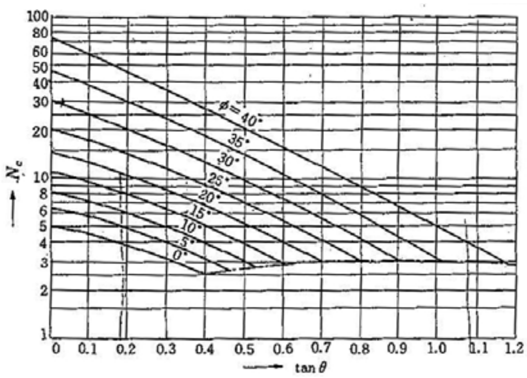
Based on the eccentric slope of the loads and Figure 3-1, the factors of bearing capacity are Nr: 1.60, Nq: 5.75 and Nc: 11.10 respectively.



Graph for Factor Nr



Graph for Factor Nq



Graph for Factor Nc

Figure 3- 1 Graph for Bearing Capacity Factor

From these factors, the ultimate bearing capacity is computed as follows.

$$Qu = Ae\left\{\alpha \cdot k \cdot c \cdot Nc + k \cdot q \cdot Nq + \frac{1}{2} \cdot \gamma_1 \cdot \beta \cdot Be \cdot Nr\right\}$$
$$= 10.80 \times \left\{1.0 \times 1.0 \times 10 \times 11.10 + 1.0 \times 0 \times 5.75 + \frac{1}{2} \times 18 \times 1.0 \times 0.900 \times 1.60\right\}$$
$$= 1{,}338.8 \approx m = 1{,}335.6kN$$

2) Spring Constant of the Foundation

As the spring constants of the foundation, following spring constants are estimated.

- Vertical reaction modulus of foundation kv
- Sharing spring constant of foundation: Ks
- Initial rotation constant of foundation: Kr_0

<u>i) Vertical Reaction Modulus of Foundation: kv</u>

$$kv = \frac{1}{0.3} \cdot \alpha \cdot E_0 \cdot \left(\frac{B_v}{0.3}\right)^{-3/4} \quad \cdots\cdots 3.2.10$$

Where, Kv: Vertical reaction modulus of foundation (kN/m^3), E_0: Deformation modulus of foundation (kN/m^2), Bv: Equivalent width of load (m) $Bv = \sqrt{B \cdot L}$,α: Coefficient related for deformation modulus based on N value (Normal condition = 1.0), B: Width of retaining wall (m), L(= min 4H): Effective length of the wall (m)

Assuming that the N value of foundation for the wall is 30, the deformation modulus of foundation (E_0) is given by following formula.

$$E_0 = 2{,}800 \cdot N = 2{,}800 \times 30 = 84{,}000kN/m^2$$

Therefore the vertical reaction modulus of foundation is given as follows.

$$kv = \frac{1}{0.3} \cdot \alpha \cdot E_0 \cdot \left(\frac{B_v}{0.3} \right)^{-3/4} = \frac{1}{0.3} \times 1.0 \times 84{,}000 \times \left(\frac{\sqrt{3.00 \times 12.00}}{0.3} \right)^{-3/4} = 29{,}606 kN / m^3$$

ii) Sharing Spring Constant of Foundation: Ks

$$Ks = \frac{B \cdot L}{4} \cdot k_v \quad \cdots\cdots 3.2.11$$

Where, Ks: Sharing spring constant (kN/m), B: Width of the retaining wall (m), L: Effective length of the wall (m), kv: Vertical reaction modulus of foundation (kN/m^3)

Using the modulus kv estimated as before, the sharing spring constant ks is given by follows.

$$Ks = \frac{B \cdot L}{4} \cdot kv = \frac{3.00 \times 12.00}{4} \times 29{,}606 = 266{,}454 kN / m$$

iii) Initial Rotation Spring Constant: Kr_0

$$K_{ro} = \frac{B^3 \cdot L}{12} \cdot kv \quad \cdots\cdots 3.2.12$$

Where, Kr_0: Initial rotation spring constant of the foundation (kNm/rad), B: Width of retaining wall (m), L: Effective length of the wall (m), kv: Vertical reaction modulus of fundaiton (kN/m^3)

Using modulus kv estimated as before, the initial rotation spring constant of the foundation is given by follows.

$$K_{ro} = \frac{B^3 \cdot L}{12} \cdot kv = \frac{3.00^3 \times 12.00}{12} \times 29{,}606 = 799{,}362 kNm / rad$$

3.4 Rotation of the Temporary Protection Wall

(1) Moments of the Wall

i) Marginal Uplift Moment: M_l

$$M_1 = m \cdot B/6 = 1{,}335.6 \times 3.00/6 = 667.8 kN \cdot m$$

Where, Ml: Marginal uplift moment (kNm), m: Mass of retaining wall (kN), B: Width of the wall (m)

ii) Design Yield Moment: My

$$My = H_R \cdot h = 610.0 \times 2.30 = 1{,}403.0 kN \cdot m$$

iii) Moment produced by the weight of Wall Footing: Mw

$$Mw = m \cdot (B/2 - X_G) = 1{,}335.6 \times (3.00/2 - 1.50) = 0 kN \cdot m$$

iv) Maximum Resistance Moment of the Foundation: Mu

$$Mu = My + Mw = 1{,}403.0 + 0 = 1{,}403.0 kN \cdot m$$

(2) Rotation Angle of the Wall

$$\theta y = \left(\frac{2Mu}{Ml} - 1 \right) \cdot \frac{Ml}{Kro} = \left(\frac{2 \times 1{,}403.0}{667.8} - 1 \right) \times \frac{667.8}{799{,}362} = 0.00267 rad = 0.15°$$

$$\theta_0 = \frac{Mw}{Kro} = \frac{0}{799{,}362} = 0$$

$$Kr = \frac{My}{\theta y - \theta_0} = \frac{1{,}403.0}{0.00267 - 0} = 525{,}468 kNm$$

(3) Dimension of the Rotated Wall

$$Z_1 = \frac{1}{2S} \cdot \left(S^2 + e_0{}^2 - i_0{}^2 \right) + \sqrt{\frac{1}{4 \cdot S^2} \cdot \left(S^2 + e_0{}^2 - i_0{}^2 \right)^2 + i_0{}^2} \quad \cdots\cdots 3.2.13$$

$$l_1 = Z_1 - S \quad \cdots\cdots 3.2.14$$

$$l_2 = l_1 + H \quad \cdots\cdots 3.2.15$$

$$l = l_1 + h \quad \cdots\cdots 3.2.16$$

Here,

$$e_0{}^2 = \frac{Kr}{Ks} = \frac{525{,}468}{266{,}454} = 1.972$$

$$i_0{}^2 = \frac{I_G}{m} = \frac{1{,}520.1}{1{,}335.6} = 1.138$$

Therefore,

$$Z_1 = \frac{1}{2S} \cdot \left(S^2 + e_0{}^2 - i_0{}^2 \right) + \sqrt{\frac{1}{4 \cdot S^2} \cdot \left(S^2 + e_0{}^2 - i_0{}^2 \right)^2 + i_0{}^2}$$

$$= \frac{1}{2 \times 1.167} \times \left(1.167^2 + 1.972 - 1.138 \right) + \sqrt{\frac{1}{4 \times 1.167^2} \times \left(1.167^2 + 1.972 - 1.138 \right)^2 + 1.138}$$

$$= 2.363 m$$

$$l_1 = Z_1 - S = 2.363 - 1.167 = 1.196 m$$

$$l_2 = l_1 + H = 1.196 + 3.000 = 4.196 m$$

$$l = l_1 + h = 1.196 + 2.300 = 3.496 m$$

(4) Velocity of the Rotating Wall

$$V = \frac{2 \cdot m'}{m' + \alpha' \cdot m'} \cdot V_0 \quad \cdots\cdots 3.2.17$$

$$\alpha' = \frac{4 \cdot (b_2 \cdot l_2 - b_1 l_1) \cdot (l_2{}^2 + l_1 \cdot l_2 + l_1{}^2) - 3 \cdot (b_2 - b_1) \cdot (l_2 + l_1) \cdot (l_2{}^2 + l_1{}^2)}{6 \cdot l^2 \cdot (b_1 + b_2) \cdot H} \text{.......................} 3.2.18$$

Where, V: Rotation velocity of the wall (m/sec), m': Weight of the wall (ton), α': coefficient of the wall form and the arm length from the center of foundation spring, V_0: Falling velocity of the rocks (m/sec)

Here,

$$\begin{aligned}
\alpha' &= \frac{4 \cdot (b_2 \cdot l_2 - b_1 l_1) \cdot (l_2{}^2 + l_1 \cdot l_2 + l_1{}^2) - 3 \cdot (b_2 - b_1) \cdot (l_2 + l_1) \cdot (l_2{}^2 + l_1{}^2)}{6 \cdot l^2 \cdot (b_1 + b_2) \cdot H} \\
&= \frac{4 \times (3.00 \times 4.196 - 1.00 \times 1.196) \times (4.196^2 + 1.196 \times 4.196 + 1.196^2) - 3 \times (3.00 - 1.00) \times (4.196 + 1.196) \times (4.196^2 + 1.196^2)}{6 \times 3.496^2 \times (1.00 + 3.00) \times 3.00} \\
&= 0.546
\end{aligned}$$

Therefore,

$$\begin{aligned}
V &= \frac{2 \cdot Ws'}{Ws' + \alpha' \cdot m'} \cdot V_0 \\
&= \frac{2 \times 0.72}{0.72 + 0.546 \times 136.29} \times 28.0 = 0.54 m / \sec
\end{aligned}$$

3.5 Stability Evaluation

(1) Wall Rotation Angle and Displacement Distance

1) Wall Rotation Angle and Displacement Distance

$$Kr_1 = Ks \cdot (e_0{}^2 + l_1{}^2) = 266{,}454 \times (1.972 + 1.196^2) = 906{,}587 kNm$$

$$\begin{aligned}
\delta d &= \sqrt{\frac{\alpha' \cdot m' \cdot l^2 \cdot V^2}{Kr_1}} \\
&= \sqrt{\frac{0.546 \times 136.29 \times 3.496^2 \times 0.54^2}{906{,}587}} \\
&= 0.017 m
\end{aligned}$$

Here,

$$\theta_L = \frac{\delta d}{l} = \frac{0.017}{3.496} = 0.0049 = 0.28°$$

$$\delta_L = \delta d - h \cdot \theta_L = \delta d \cdot \left(1 - \frac{h}{l}\right) = 0.017 \times \left(1 - \frac{2.300}{3.496}\right) = 0.0058 m$$

2) Rotating Deformation Energy: E_{ML} and Horizontal Deformation Energy: E_{HL}

$$E_{ML} = \frac{1}{2} \cdot kr \cdot \theta_L{}^2 = \frac{1}{2} \times 525{,}468 \times 0.0049^2 = 6.31 kJ$$

$$E_{HL} = \frac{1}{2} \cdot Ks \cdot \delta_L{}^2 = \frac{1}{2} \times 266{,}454 \times 0.0058^2 = 4.48 kJ$$

3) Absorbing Energy: E_M

$$E_M = \frac{My \cdot (\theta y - \theta_0)}{2} + My \cdot (\theta a - \theta y) \quad \cdots\cdots 3.2.19$$

$$\theta a = \mu \cdot \theta y \quad \cdots\cdots 3.2.20$$

Here,

$$\theta a = \mu \cdot \theta y = 5.0 \times 0.00267 = 0.0134 rad = 0.76° \leq 2.0°$$

$$\begin{aligned} E_M &= \frac{My \cdot (\theta y - \theta_0)}{2} + My \cdot (\theta a - \theta y) \\ &= \frac{1{,}403.0 \times (0.00267 - 0)}{2} + 1{,}403.0 \times (0.0134 - 0.00267) \\ &= 16.93 kJ \end{aligned}$$

4) Stability Evaluation

Stability condition of the wall is that the rotating deformation energy E_{ML} should be less than the absorbing energy of the foundation E_M.

E_{ML} = 6.31kJ

E_M = 16.93kJ

Therefore,

$$Fs = \frac{E_M}{E_{ML}} = \frac{16.93}{6.31} = 2.68 \geq Fs = 1.00 \quad \text{OK}$$

3.6 Result of the Stability Analysis

Result of the stability analysis is shown in Table 3-5.

Table 3- 5 Result of the Stability Analysis (1/3)

Dia. Rock	Weight (kN)	Slope Height (H)	10	15	20	25	30	35	40
		Velocity (V_0)	14.00	17.15	19.80	22.14	24.25	26.19	28.00
		V	0.07	0.08	0.10	0.11	0.12	0.13	0.14
0.5	1.73	θ_L	0.001	0.001	0.001	0.001	0.001	0.001	0.001
	(0.18)	δ_L	0.0007	0.0009	0.0010	0.0012	0.0013	0.0014	0.0015
		E_{ML}	0.10	0.15	0.20	0.25	0.30	0.35	0.39
		E_{HL}	0.07	0.11	0.14	0.18	0.21	0.25	0.29
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	171.25	114.17	85.63	68.50	57.08	48.93	42.81
		V	0.12	0.14	0.16	0.18	0.20	0.22	0.23
0.6	3.00	θ_L	0.001	0.001	0.001	0.002	0.002	0.002	0.002
	(0.31)	δ_L	0.0013	0.0015	0.0018	0.0020	0.0022	0.0024	0.0025
		E_{ML}	0.29	0.44	0.58	0.73	0.87	1.02	1.17
		E_{HL}	0.21	0.32	0.42	0.53	0.63	0.74	0.84
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	57.94	38.63	28.97	23.18	19.31	16.55	14.48
		V	0.18	0.22	0.26	0.29	0.32	0.34	0.37
0.7	4.76	θ_L	0.002	0.002	0.002	0.003	0.003	0.003	0.003
	(0.49)	δ_L	0.0020	0.0024	0.0028	0.0031	0.0034	0.0037	0.0040
		E_{ML}	0.72	1.09	1.45	1.81	2.17	2.54	2.90
		E_{HL}	0.52	0.79	1.05	1.31	1.57	1.84	2.10
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	23.30	15.53	11.65	9.32	7.77	6.66	5.83
		V	0.27	0.33	0.38	0.42	0.47	0.50	0.54
0.8	7.10	θ_L	0.002	0.003	0.003	0.004	0.004	0.005	0.005
	(0.72)	δ_L	0.0029	0.0036	0.0041	0.0046	0.0050	0.0054	0.0058
		E_{ML}	1.56	2.33	3.11	3.89	4.67	5.44	6.22
		E_{HL}	1.13	1.69	2.25	2.81	3.38	3.94	4.50
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	10.86	7.24	5.43	4.34	3.62	3.10	2.71
		V	0.38	0.47	0.54	0.60	0.66	0.72	0.77
0.9	10.12	θ_L	0.003	0.004	0.005	0.005	0.006	0.006	0.007
	(1.03)	δ_L	0.0041	0.0051	0.0059	0.0065	0.0072	0.0077	0.0083
		E_{ML}	3.16	4.73	6.31	7.89	9.47	11.05	12.63
		E_{HL}	2.28	3.43	4.57	5.71	6.85	8.00	9.14
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	5.35	3.57	2.68	2.14	1.78	1.53	1.34

Table 3-5 Result of the Stability Analysis (2/3)

Dia. Rock	Weight (kN)	Slope Height (H)	10	15	20	25	30	35	40
		Velocity (V0)	14.00	17.15	19.80	22.14	24.25	26.19	28.00
		V	0.52	0.64	0.74	0.83	0.91	0.98	1.05
1.0	13.88	θ_L	0.005	0.006	0.007	0.008	0.008	0.009	0.010
	(1.42)	δ_L	0.0057	0.0070	0.0080	0.0090	0.0098	0.0106	0.0114
		E_{ML}	5.94	8.91	11.88	14.84	17.81	20.78	23.75
		E_{HL}	4.30	6.45	8.60	10.74	12.89	15.04	17.19
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	2.84	1.90	1.42	1.14	0.95	0.81	0.71
		V	0.69	0.85	0.98	1.09	1.20	1.29	1.38
1.1	18.47	θ_L	0.006	0.008	0.009	0.010	0.011	0.012	0.013
	(1.88)	δ_L	0.0075	0.0092	0.0106	0.0118	0.0129	0.0140	0.0149
		E_{ML}	10.28	15.42	20.57	25.71	30.85	35.99	41.13
		E_{HL}	7.44	11.16	14.89	18.61	22.33	26.05	29.77
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	1.64	1.09	0.82	0.66	0.55	0.47	0.41
		V	0.89	1.09	1.26	1.41	1.55	1.67	1.79
1.2	23.98	θ_L	0.008	0.010	0.011	0.013	0.014	0.015	0.016
	(2.45)	δ_L	0.0097	0.0118	0.0137	0.0153	0.0167	0.0181	0.0193
		E_{ML}	17.21	25.81	34.41	43.01	51.62	60.22	68.82
		E_{HL}	12.45	18.68	24.91	31.13	37.36	43.59	49.81
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	0.98	0.65	0.49	0.39	0.33	0.28	0.25
		V	1.12	1.38	1.59	1.78	1.95	2.10	2.25
1.3	30.48	θ_L	0.010	0.012	0.014	0.016	0.018	0.019	0.020
	(3.11)	δ_L	0.0122	0.0149	0.0172	0.0192	0.0211	0.0228	0.0243
		E_{ML}	27.25	40.88	54.51	68.13	81.76	95.39	109.01
		E_{HL}	19.73	29.59	39.45	49.32	59.18	69.04	78.90
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	0.62	0.41	0.31	0.25	0.21	0.18	0.15
		V	1.39	1.70	1.96	2.20	2.41	2.60	2.78
1.4	38.07	θ_L	0.013	0.015	0.018	0.020	0.022	0.024	0.025
	(3.88)	δ_L	0.0150	0.0184	0.0213	0.0238	0.0260	0.0281	0.0301
		E_{ML}	41.59	62.38	83.18	103.97	124.76	145.56	166.35
		E_{HL}	30.10	45.15	60.20	75.25	90.31	105.36	120.41
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	0.41	0.27	0.20	0.16	0.14	0.12	0.10

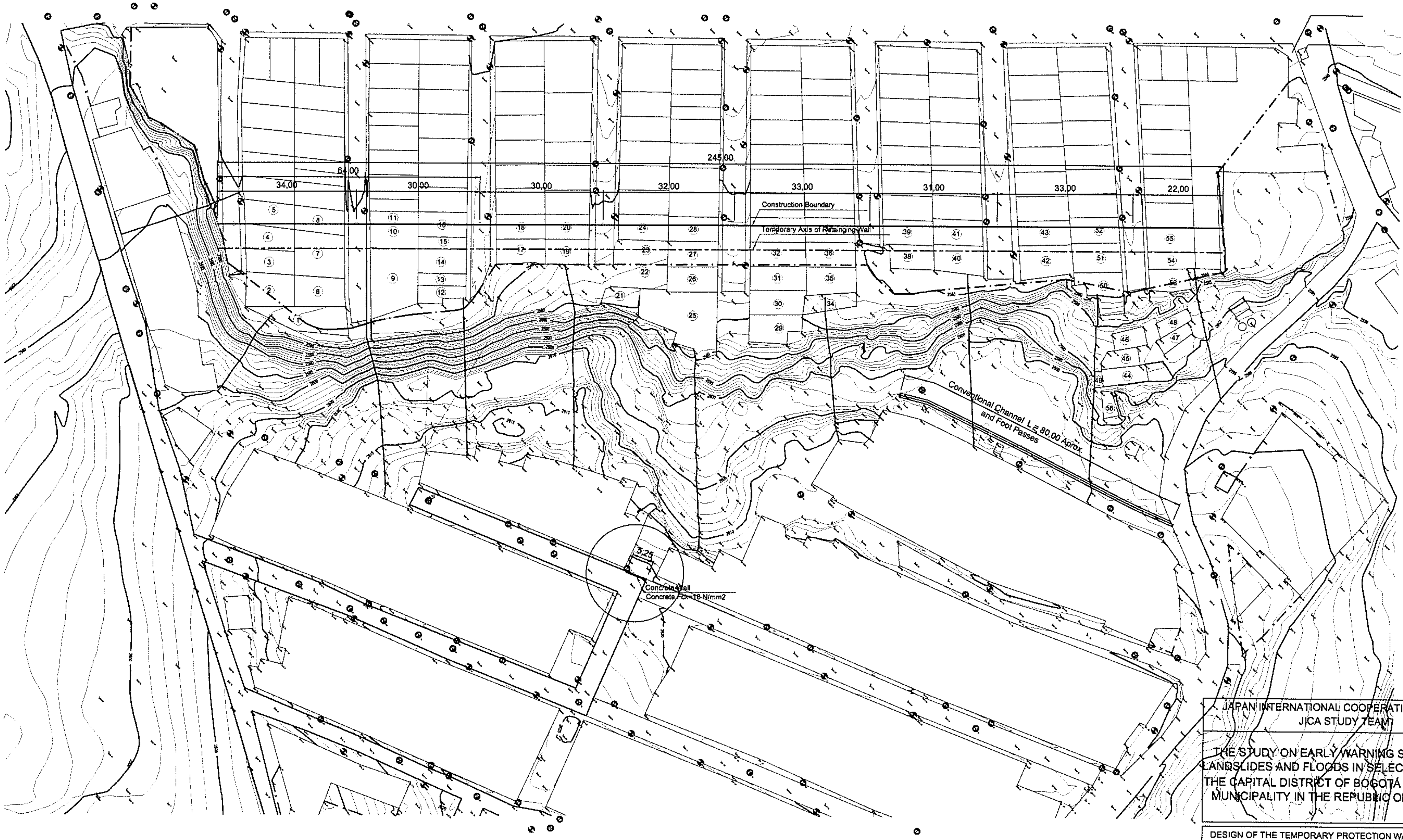
Table 3-5 Result of the Stability Analysis (3/3)

Dia. Rock	Weight (kN)	Slope Height (H)	**10**	**15**	**20**	**25**	**30**	**35**	**40**
		Velocity (V0)	**14.00**	**17.15**	**19.80**	**22.14**	**24.25**	**26.19**	**28.00**
		V	1.69	2.07	2.39	2.67	2.93	3.16	3.38
1.5	46.83	θ_L	0.015	0.019	0.022	0.024	0.027	0.029	0.031
	(4.78)	δ_L	0.0183	0.0224	0.0259	0.0289	0.0317	0.0342	0.0366
		E_{ML}	61.69	92.54	123.38	154.23	185.07	215.92	246.77
		E_{HL}	44.65	66.98	89.31	111.63	133.96	156.29	178.61
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	0.27	0.18	0.14	0.11	0.09	0.08	0.07
		V	2.03	2.48	2.87	3.20	3.51	3.79	4.05
1.6	56.83	θ_L	0.018	0.023	0.026	0.029	0.032	0.034	0.037
	(5.80)	δ_L	0.0219	0.0269	0.0310	0.0347	0.0380	0.0410	0.0439
		E_{ML}	88.53	132.80	177.06	221.33	265.60	309.86	354.13
		E_{HL}	64.08	96.12	128.16	160.20	192.24	224.28	256.32
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	0.19	0.13	0.10	0.08	0.06	0.05	0.05
		V	2.40	2.94	3.39	3.79	4.15	4.48	4.79
1.7	68.17	θ_L	0.022	0.027	0.031	0.034	0.038	0.041	0.043
	(6.96)	δ_L	0.0259	0.0318	0.0367	0.0410	0.0449	0.0485	0.0519
		E_{ML}	123.88	185.81	247.75	309.69	371.63	433.56	495.50
		E_{HL}	89.66	134.49	179.32	224.16	268.99	313.82	358.65
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	0.14	0.09	0.07	0.05	0.05	0.04	0.03
		V	2.80	3.43	3.96	4.43	4.85	5.24	5.60
1.8	80.92	θ_L	0.025	0.031	0.036	0.040	0.044	0.047	0.051
	(8.26)	δ_L	0.0303	0.0371	0.0429	0.0479	0.0525	0.0567	0.0606
		E_{ML}	169.02	253.54	338.05	422.56	507.07	591.59	676.10
		E_{HL}	122.34	183.51	244.68	305.85	367.02	428.20	489.37
		E_M	16.89	16.89	16.89	16.89	16.89	16.89	16.89
		Fs	0.10	0.07	0.05	0.04	0.03	0.03	0.02

D3.3 DRAWINGS

OF THE TEMPORARY WORKS


983000
984000
985000
986000
998000
997000
ALCALDIA DE SOACHA
EL DIVINO NIÑO
CANTERA RECURSO
246-II-A2
246-II-B1
2550
2600
2650
2700
0 0,05 0,1 0,2 0,3 0,4 0,5 Km.
AGENCIA DE COOPERACIÓN JAPONESA - EQUIPO DE ESTUDIO JICA
ESTUDIO SOBRE SISTEMA DE MONITOREO Y ALERTA TEMPRANA DE DESLIZAMIENTOS E INUNDACIONES EN BOGOTÁ DISTRITO CAPITAL Y MUNICIPIO DE SOACHA EN LA REPÚBLICA DE COLOMBIA
DISEÑO DEL MURO DE CONTENCIÓN SECTOR DIVINO NIÑO, MUNICIPIO DE SOACHA
Titulo del Plano: MAPA DE UBICACION DEL PROYECTO
Plano No. 01
Escala 1:10.000
Diseño
Revisión
Aprobación
Bogotá, 2007



General Plan

Scale: 1/1000

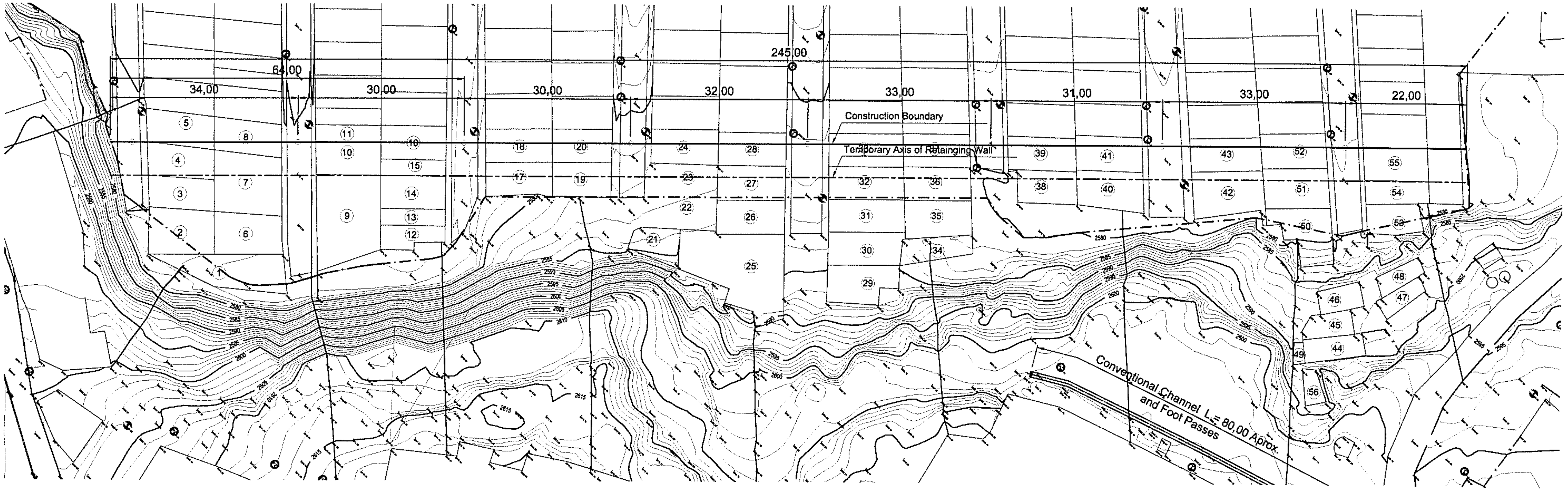
JAPAN INTERNATIONAL COOPERATION AGENCY
JICA STUDY TEAM

THE STUDY ON EARLY WARNING SYSTEM FOR LANDSLIDES AND FLOODS IN SELECTED AREAS IN THE CAPITAL DISTRICT OF BOGOTÁ AND SOACHA MUNICIPALITY IN THE REPUBLIC OF COLOMBIA

DESIGN OF THE TEMPORARY PROTECTION WALL, DIVINO NINO SECTOR, SOACHA MUNICIPALITY

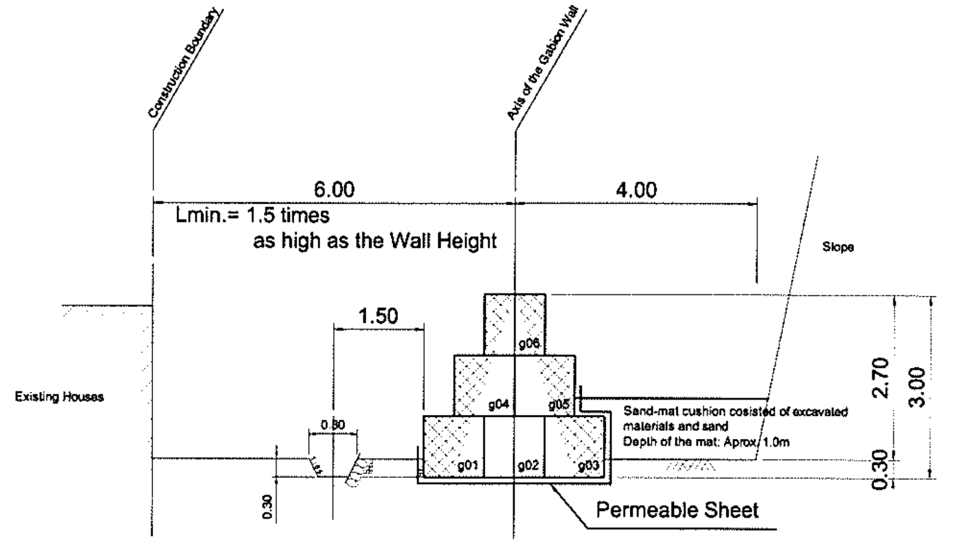
Title of Drawing: GENERAL PLAN OF THE PROJECT		
Drawing No. 02		
Design	Revision	Approval

Bogotá, 2007



General Plan of The Temporary Protection Wall

Scale: 1/750



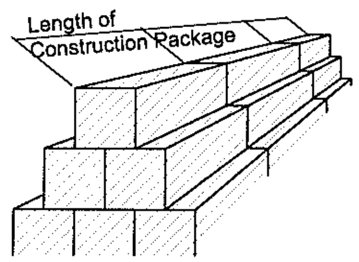
Standard Section of the Temporary Protection Wall

Scale: 1/125

Gabion
The gabion blocks of No. g01 and g03 to g06 are the conventional type consisted of cobble stone. As to the gabion block g02, the debris produced by the demolition works is used. For this reason, some sheets or sandbags should be prepared to both vertical sides of the gabion block g02 so as not to scattering of the internal material.

Sand-mat Cushion behind the Wall
The sand-mat cushion is installed behind all of wall installed in order to absorb the kinetic energy of falling rocks. The material to be used is consisted of the excavated materials. In case that the debris produced by the demolition works is used as the material, the Contractor should inform it to the JICA Study Team. Compaction of the cushion is not required due to its function.

Permeable Sheet
The part of wall which is between the gabion and sand-mat cushion should be covered by permeable sheet.



Wall Body
In order to resist a hit of falling rocks, the installed gabions in the site should be integrated each gabions with any method so as to be a body.

Wall Length
The gabion wall should have at least a length of 6.0m as its body. For this reasons, the Contractor should select adequate gabion net at the market. If adequate gabion net is not found out in the marcket, conventional gabion net can be utilized combining with the wire which is applied to the four corners of gabion net.

Note:

1. Construction area as shown in the drawings should be available with the authorization by the Soacha Municipality. The Contractor can no execute any works in the contract without the document of authorization of the Municipality.
2. Under the authorization by the Municipality, the wall axis as shown ir the drawings can be modified/changed according to the relocation procedure by the Municipality. For this reasons, the Contractor shou inform to JICA Study Team about the possibility of the construction.
3. Construction period is 90 calendar days from 22 of November to 19 February 2008. If some inconveniences on its execution are confirmed, the Contractor should inform usually to JICA Study Team about the possibility of execution of the works.

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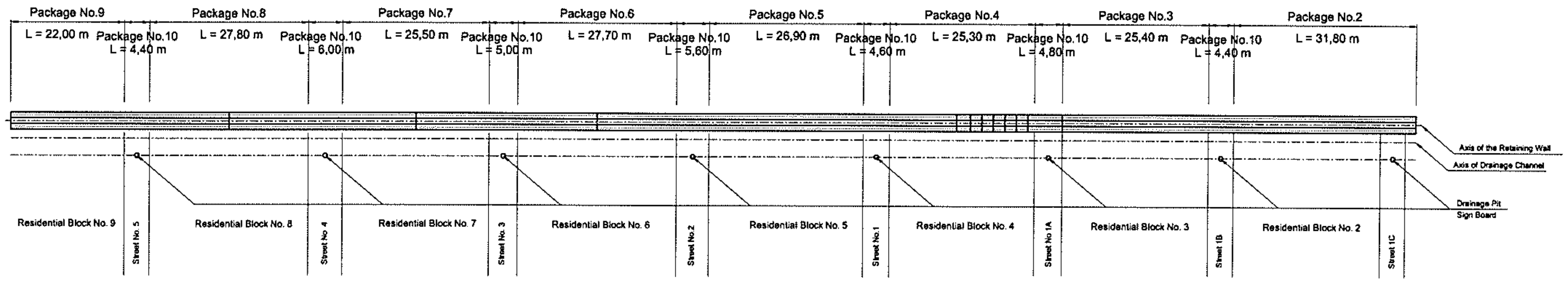
THE STUDY ON EARLY WARNING SYSTEM FOR LANDSLIDES AND FLOODS IN SELECTED AREAS IN THE CAPITAL DISTRICT OF BOGOTÁ AND SOACHA MUNICIPALITY IN THE REPUBLIC OF COLOMBIA

DESIGN OF THE TEMPORARY PROTECTION WALL, DIVINO NINO SECTOR, SOACHA MUNICIPALITY

Title of Drawing:
TEMPORARY PROTECTION WALL

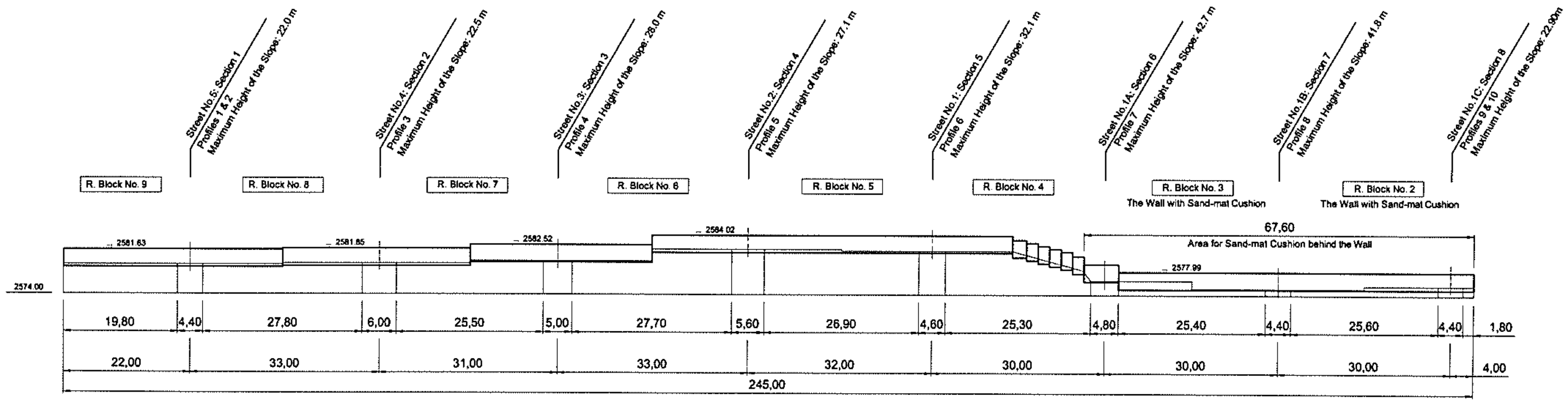
Drawing No.	03	
Design	Revision	Approval

Bogotá, 2007



Location of the Structures

Scale: 1/ 1,000



Frontal View of Temporary Protection Wall

Scale: 1/ 1,000

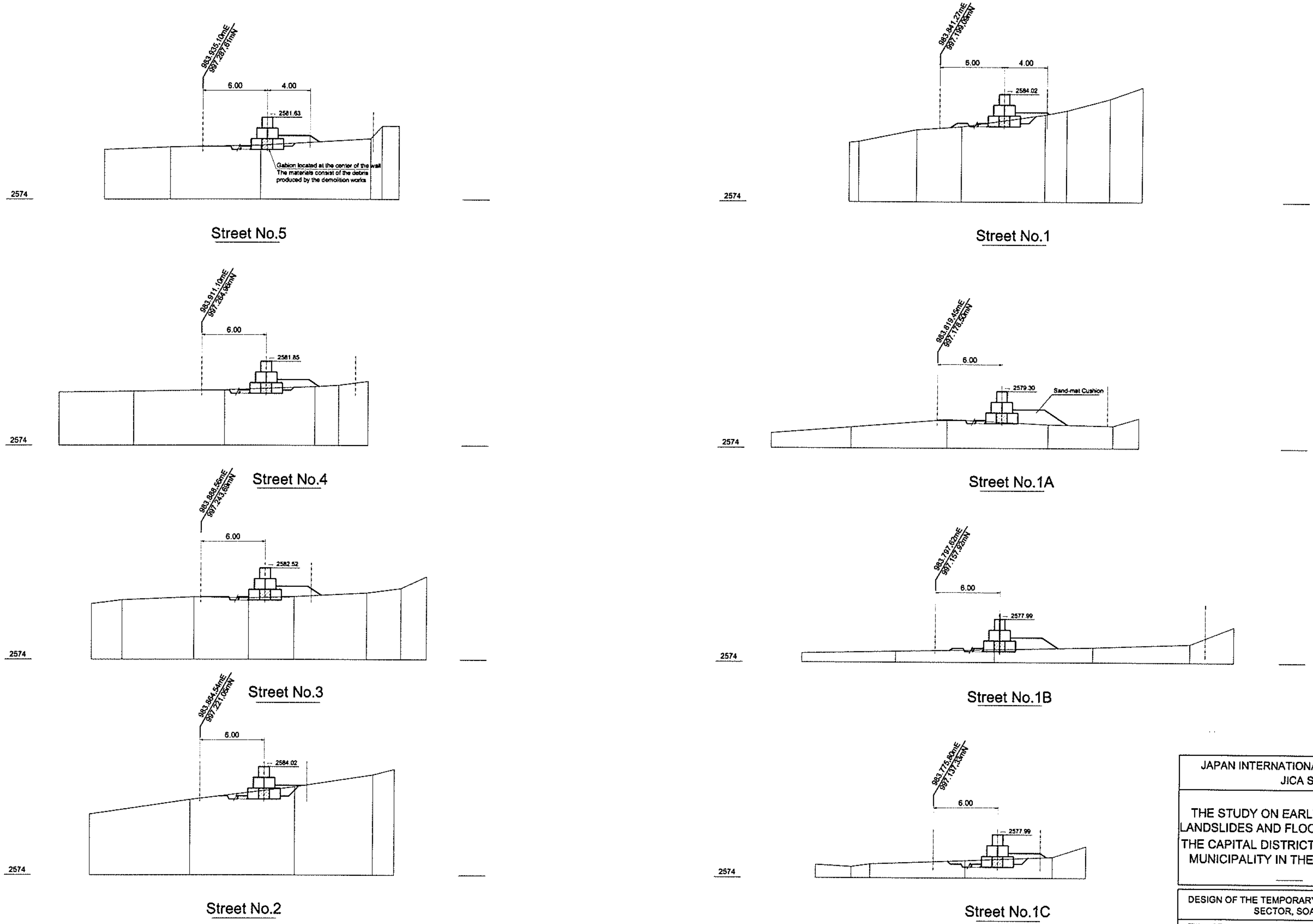
Note:

Elevation of wall foundation is applied an average ground level in each residential blocks and is leveled for wall installation according to the drawings. For define the wall foundation level, the Contractor should confirm actual elevation of each residential blocks by means of the topographic survey in the site. As to the elevation of wall foundation, the approval by the JICA Study Team should be required before the commencement of the construction.

The elevation of wall top as shown in the drawings can be modified according to the foundation level approved by the JICA Study Team.

JAPAN INTERNATIONAL COOPERATION AGENCY JICA STUDY TEAM	
THE STUDY ON EARLY WARNING SYSTEM FOR LANDSLIDES AND FLOODS IN SELECTED AREAS IN THE CAPITAL DISTRICT OF BOGOTÁ AND SOACHA MUNICIPALITY IN THE REPUBLIC OF COLOMBIA	
DESIGN OF THE TEMPORARY PROTECTION WALL, DIVINO NINO SECTOR, SOACHA MUNICIPALITY	
Title of Drawing: INSTALLATION OF THE TEMPORARY PROTECTION WALL	
Drawing No. 04	
Design	Revision / Approval

Bogotá, 2007



Cross Sections

Scale: 1/400

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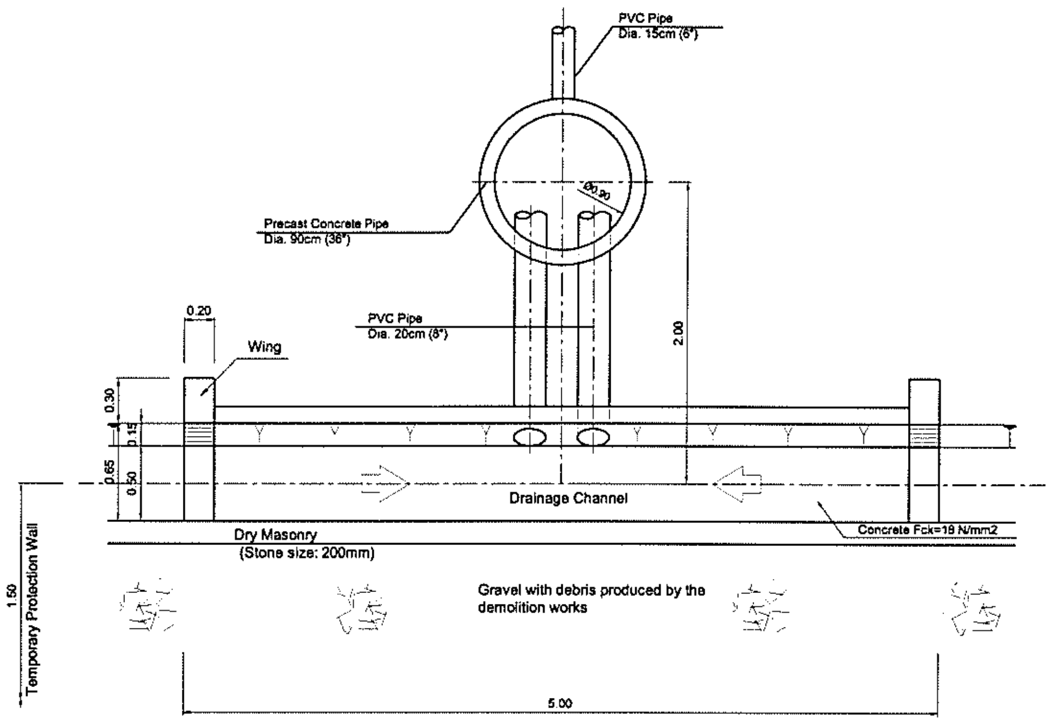
DESIGN OF THE TEMPORARY PROTECTION WALL, DIVINO NINO SECTOR, SOACHA MUNICIPALITY

Title of Drawing:
INSTALLATION OF THE TEMPORARY PROTECTION WALL

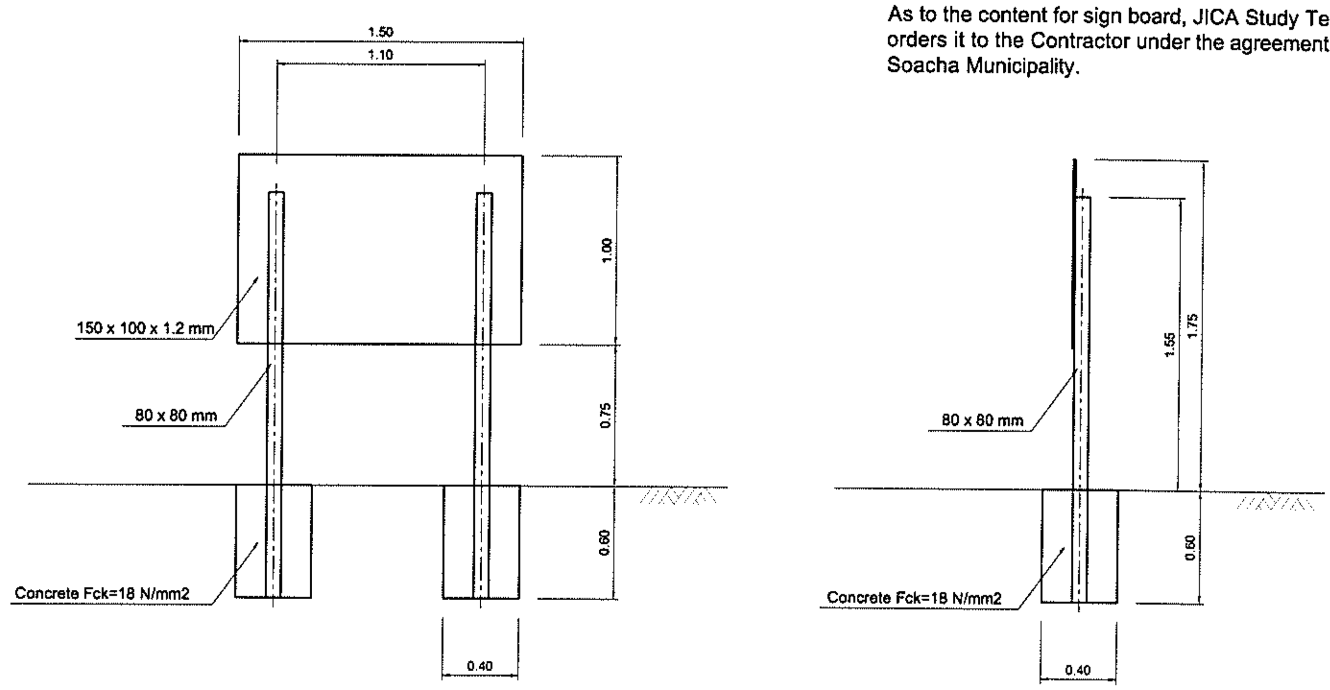
Drawing No.	05	
Design	Revision	Approval

Bogotá, 2007



Plan: Drainage Pit

Scale: 1/50

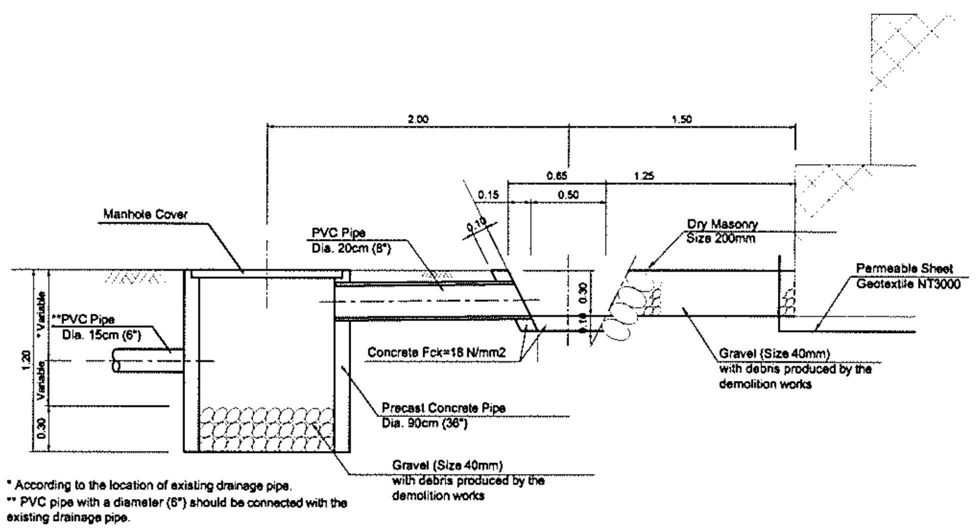


Note:

As to the content for sign board, JICA Study Team orders it to the Contractor under the agreement of the Soacha Municipality.

Sign Board

Scale: 1/40



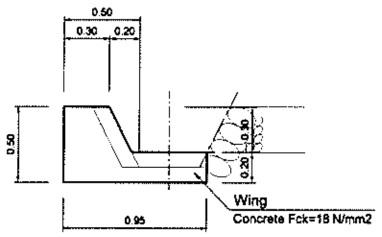
* According to the location of existing drainage pipe.
** PVC pipe with a diameter (6") should be connected with the existing drainage pipe.

Specification of Geotextil for permeable sheet

- Tension resistance: 760 N
- Elongation: > 50 %
- Puncture Resistance: 430 N
- Trapezoid Tearing Strength: 350 N
- Bursting Resistance: 2208 kPa
- Apparent Opening Size: 0,15 mm

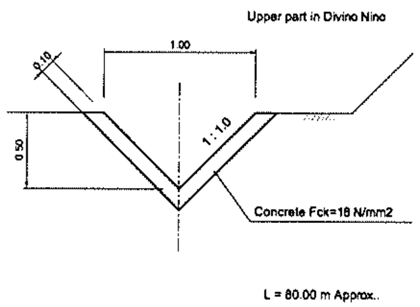
Cross Section of Drainage Pit

Scale: 1/50



Wings of Drainage Channel

Scale: 1/50

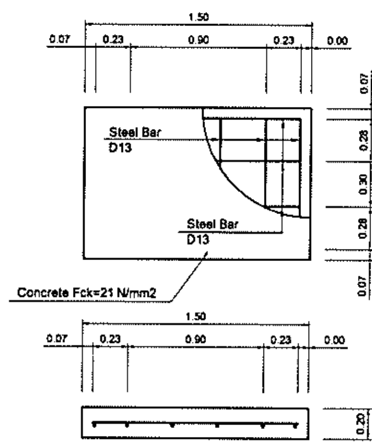


L = 80.00 m Approx.

Conventional Channel

Scale: 1/50

Conventional Channel
at the Upper Part of Divino Nino



Foot Passes

Scale: 1/50

Note:
1) 10 foot passes are installed in front of the existing house.
2) Fck = f'c

JAPAN INTERNATIONAL COOPERATION AGENCY
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THE STUDY ON EARLY WARNING SYSTEM FOR LANDSLIDES AND FLOODS IN SELECTED AREAS IN THE CAPITAL DISTRICT OF BOGOTÁ AND SOACHA MUNICIPALITY IN THE REPUBLIC OF COLOMBIA

DESIGN OF THE TEMPORARY PROTECTION WALL, DIVINO NINO SECTOR, SOACHA MUNICIPALITY

Title of Drawing:
DRAINAGE PIT, SIGN BOARD AND CONVENTIONAL CHANNEL

Drawing No.	06	
Design	Revision	Approval

Bogotá, 2007