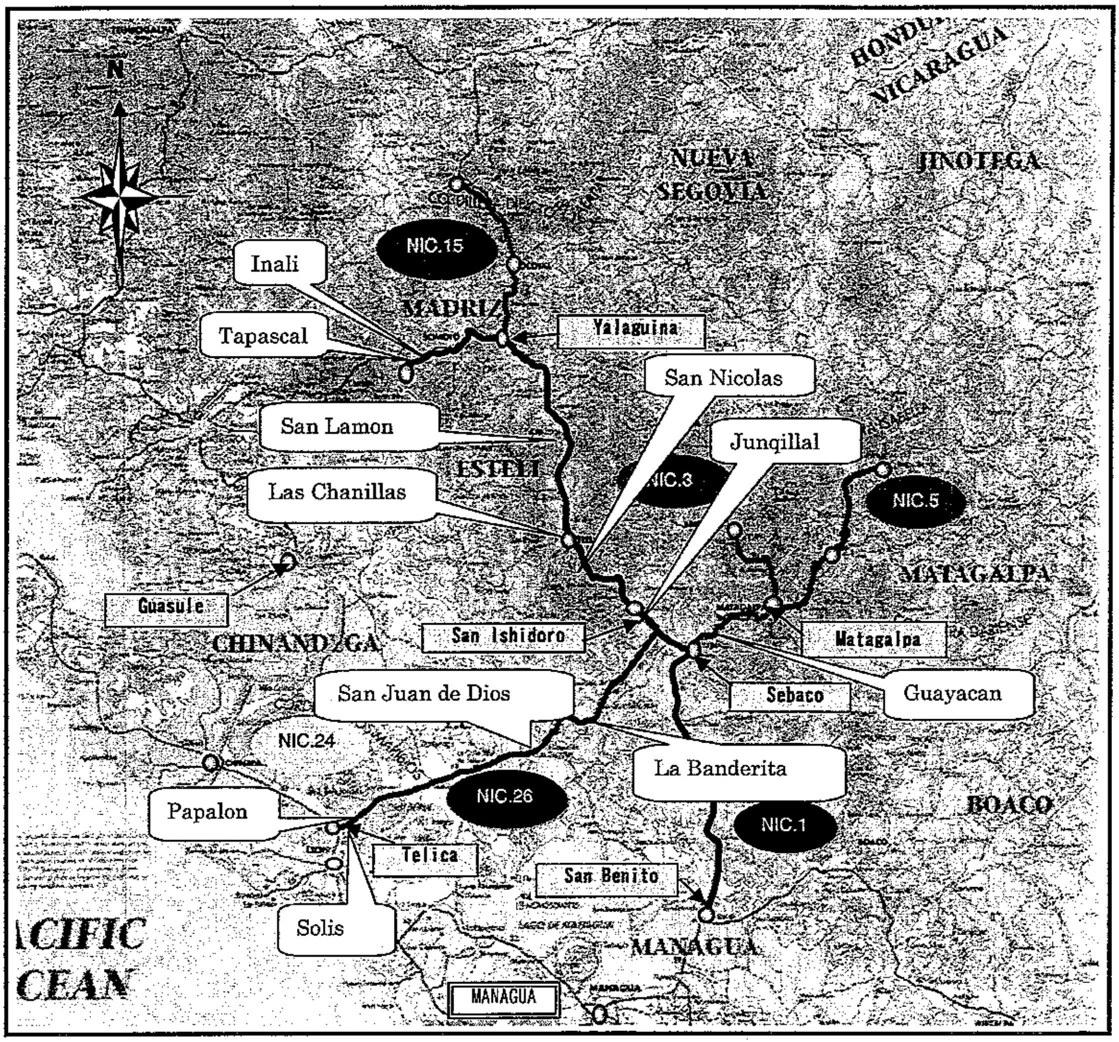


Figure 17.2.4 Disaster Prevention Spots Requiring Countermeasures for Bridge Foundation Scouring

Table 17.2.5 Present Condition of Objective Bridges (1/3)

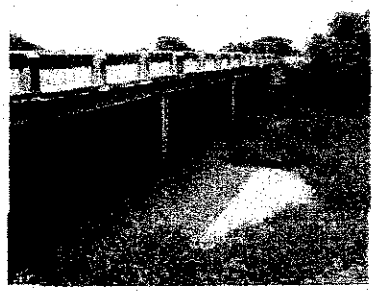
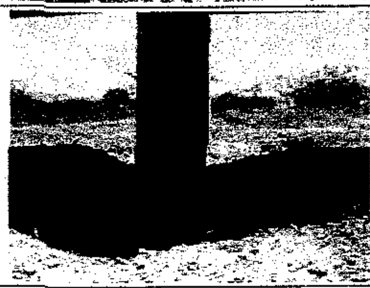
Bridge Name	Present Condition	Photo
Junqillal	(i) Land both upstream and downstream is used as paddy field. Even in the dry season, water remains around the bridge. (ii) Although the river does not seem to flow, even in the rainy season, traces of scouring can be seen on the upstream side of the bridge (the size of scouring is 5 meters from the pier to upstream channel). (iii) Because there is water, it is impossible to see the total extent of the scouring. However, judging from the amount of water on the upstream side, the depth of the scouring seems to be extensive. (iv) Mud generated by scouring seems to piling up on the downstream side.	
San Nicolas	(i) The revetment on the Managua side and the revetment on the upstream side are in a state of collapse. Soil behind abutments has been running off, resulting in holes occurring behind the abutments. (ii) The total amount of riverbed scouring is not serious.	
Las Chanillas	(i) There is a large amount of scouring around piers. (ii) The vertical alignment of the river channel is steep around the bridge. The level of the riverbed has become lower because of scouring of the bridge foundation. (iii) No large abnormality has been detected around the abutments.	
San Ramon	(i) Remains of abutment of former bridge are located in front of abutment of present bridge (3m away). The present abutment is being eroded by water flowing between the present abutment and former abutment. (ii) Because sand is heaped up between the abutments of the current and previous bridges, the center of the river channel has shifted to the end side abutment. Therefore, the position of the bridge does not correspond to the position of the river properly. (iii) River flow is turbulent near the bridge, but it is unknown whether this is a natural river flow or is artificially induced, making it impossible to judge which side of the bridge is upstream in the dry season. (iv) The vertical alignment of water that seems to flow all the time in the river channel is deepest at the bridge.	
Inali	(i) No major damage has been detected. However, the Mitti Flood did widen the river channel. (ii) Because the riverbed slightly dips at the bridge, scouring of the bridge foundation can be seen.	

Table 17.2.5 Present Condition of Objective Bridges (2/3)

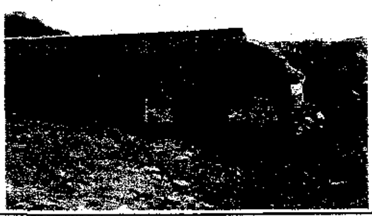
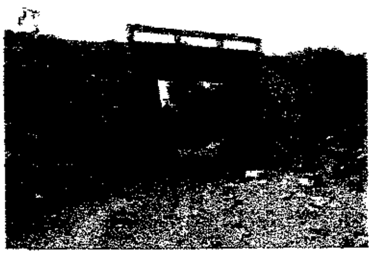
Bridge Name	Present Condition	Photo
Tacapali	(i) The revetment of start side abutment suffers from major damage (although the rehabilitation has been already done by the rainy season). (ii) Because the river is curving at right angle in front of river, the major tracks of river scouring can be seen at the start side abutment and start side pier. For the reason, the sand was heaped up at the end side of those structures. (iii) The scouring in upstream side of pier is local, 4m in width, 10m in length, and 1m in depth. (iv) Around the middle piers, it is not clear due to the water, but the depth is 0.5m-1m, the width is 15m, and the length is 30m.	
El Guayacan	(i) Since the bridge is an arch bridge, the obstruction ratio is large. (ii) Due to subsidence of the end side abutment owing to the scouring of the bridge foundation, the bridge wing broke off. (iii) The position of bridge does not correspond to that of the river.	
Solis	(i) Bridge foundation scouring is serious, and the level of the riverbed is 30-40 cm lower than the bottom of the abutment footing due to erosion. (ii) The level of the riverbed has become lower not only at the bridge but throughout the whole river channel as well. For that reason, there is no considerable change in the vertical alignment at the bridge. (iii) River width at the bridge spot is narrower that upstream and downstream. Because of the progress in erosion, the H.W.L and the free space under the beam are adequate. (iv) The riverbed is relatively solid, but it is covered by powdered fine-gained soil about 10cm thick, which would be washed away easily if there were a strong water flow. (v) The back of the bridge wing on the upstream side has been eroded away largely. (vi) The slope of the river is 2% and is relatively steep, and there are a few obstructions. Therefore, the velocity of water flow is fast even if total water volume is quite small.	
Papalón	(i) The scouring of the bridge foundation is serious, and the level of the riverbed is 30-40 cm lower than the bottom of the abutment footing due to erosion. (ii) The level of the riverbed has become lower not only at the bridge but throughout the whole river channel as well. For that reason, there is no considerable change in vertical alignment at the bridge. (iii) The width of the river at the bridge is narrower that upstream and downstream. Because of the progress of erosion, the H.W.L and the free space under the beam are adequate. (iv) The riverbed is relatively solid, but it is covered by the powdered fine-gained soil about 10cm thick, which would be washed away easily if there were a strong water flow. (v) The back of the bridge wing on the upstream side has been eroded away largely. (vi) The slope of river is 2% and relatively steep, and there are a few obstructions. Therefore, the velocity of water flow is fast even if the total water volume is quite small.	

Table 17.2.5 Present Condition of Objective Bridges (3/3)

Bridge Name	Present Condition	Photo
SanJuan de Dios	(i) Because the river flow splits into 2 ways upstream, soilis deposited between the columns of the bridge structure on the Telica side, which has small volumes of water flow. Furthermore, because the river channel inclines to the end side, scouring of the abutment is noticeable. (ii) At places, scouring has proceeded up to the floor surface.	
La Banderita	(i) Although piers are rigid-framed, there is little scouring around the piers. (ii) Although the bottom of the abutment is about 3 meters higher than the riverbed, erosion has proceeded to the front of the abutment due to the small distance between the pier and the slope(about 2 meters). (iii) The abutment seems to be bedded on weathered tuff. Note that Weathering is severe at the exposed part of the front of the abutment.	

17.3 Preliminary Engineering Design of Slope Stability

17.3. 1 The Examination Method of Slope Stability Based on the Geology Characteristic of an Investigation Place

It is estimated that the bedrock slopes here were exposed to weathering and erosion in terrigenous states through crustal movement, metamorphism or volcanic activities in the process of geological time of tens million years. Therefore, it can be considered that weathering force over a long time affected the slope bedrocks. For this reason, stability of the slopes in a loosened area with many different weathering modes should be reviewed. Though weathering-based looseness is totally observed, andesite rocks on the top in a table terrace shape is most weathered, becoming block with their cracks expanded and remaining on a rim of a natural slope as large loosened blocks and rolling stones. In some cases, the rocks fell to roads. Since these topography were formed before or in the Diluvial epoch, it can be considered that they are traces of weathering during long-term geological time. Therefore, slope stability measures for "Remained surface hard layeres" may be reviewed. In Nicaragua, cut is generally located at an erosion cliff or a hillside slope on the Quaternary lava plateau, but cut is made on the surface hard layer at higher altitudes. Since the lava plateau where the current roads exit is flat, some low embankments are placed on the shells. However, some slopes that became soil are washed out and they are also called Waste as barren soil.

Since andesite rocks of hard rocks are reportedly are weathered at a rate of some mm/1,000 years, a section of 20 to 30 m of the rocks from surface may be weathered by simple arithmetic. Considering that tuff rocks are softer than andesite rocks and are further deeply weathered, a bedrock weathering layer that should be studied in a slope is a most-weathered surface course. In addition to weathering force toward the depth from the surface, alterations such as hydrothermal alteration, which affects the depth to the surface, also contribute to degradation of bedrocks. Hydrothermal fluid does not simply mean high temperature, but has a high temperature enough for altering rocks. A method for indicating the level of hydrothermal alteration in terms of mineral chemistry is not still established.

It can be generally considered that instability of primary minerals due to hydrothermal alteration reduces bedrock strength, but some rocks increase their strength due to their formation of altered minerals. For instance, A060 (24.7km) tuff rocks in NIC.26 are welded and also white-discolored due to hydrothermal alteration, while increasing their strength becomes higher than that of regular tuff rocks. This phenomena frequently occurs in NIC.26, while their strength decreased due to hydrothermal alteration (plus falut) as could be seen in B140- A150 (34.0- 34.2km). This phenomena can be observed about tuff rocks on E170

(35.2km) or C160- C140 (35.9- 39.4km) slopes in NIC.3. Based on these observation, how to determine weathered and loosened slope gradients by height and by rock type is shown in the following flow chart (**Figure 17.3.1**).

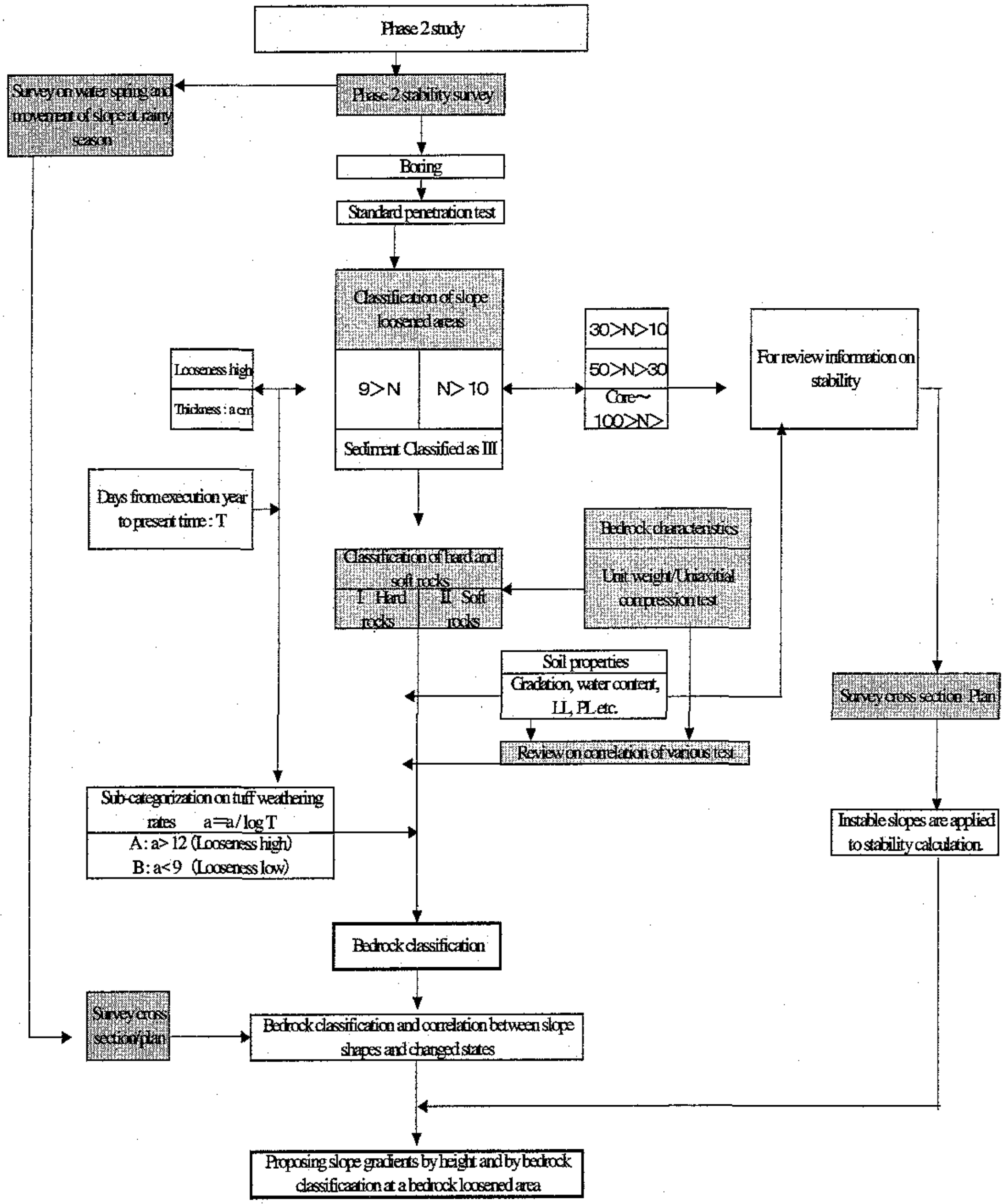


Figure 17.3.1 Analysis Method for Slope Gradient in a Bedrock Loosening Area

N-value, soil and bedrock characteristics and weathering rates used for analysis in this section are described in the following sub-section.

17.3.2 Description on Data Used for Analysis

1) Standard Penetration Test (SPT)

In the standard penetration test , when a 140±2lb hammer is made free fall from 30±1in in height, striking numbers required for its 30 cm penetration into the ground is called N-value after hitting 15cm penetration as pre-test. An open sampler with 50 mm in outer diameter and 80 cm in length is utilized in the test. It is said that the following correlation between N-value and sand relative density is shown (Terzaghi・Peck).

N-value	Relative density
0- 4	Very loose
4- 10	Loose
10- 30	Medium
30- 50	Dense
50 or more	Very dense

In the Phase 2 survey, in order to review a slope loosened area, weathering layers of penetrable tuff rocks SPT were divided into the following four levels in accordance with the above category.

9>N	(When the hitting number for first 10 cm-penetration equals to <3, <3 was tripled. When N-value is over 10 cm, the sampler is sometimes encountering to gravels and resulting in sudden increase of N-value. The value may return to a lower figure in medium N-value. So, the ground was determined to be generally a very loose layer consisting of fine gravels.)
30>N>10	
50>N>30	
N>50~100	(Up to core)

The identified N-values were determined to be a loosened area value for each slope. Concerning to these results, refer to Table 17.3.2 (1)- (5), analysis on loosened areas.

The bedrocks are originally exposed to weathering and may be affected by various inter-related factors such as slope excavation, local stress distribution, time-lapse alteration process of bedrocks, water-based physical and chemical actions and rainfall and temperature-induced time-lapse changes. So, it is uncertain which loosened area belongs to progressive creep collapse range. The area of N<9 was considered to represent at least a sign of stress release-based looseness after cutting the slope. Based on it, post-cut weathering rates or loosening rates were determined to categorize each slope bedrock as well as hard and soft rocks.

2) Laboratory Test

A laboratory test for the following items as shown Table 17.3.1 was conducted. The respected results are described in Table 17.3.2.

Table 17.3.1 Laboratory Test Items and Results

Item	
1. Soil Propaerties Test	**Results of Test**
Natural water content	Natural water content normally lies between liquid and plastic limits, but the test result shows it is below a plastic limit and in pretty dry condition.
Specific gravity	They specific gravity of 2.2~2.6, lower than the value of 2.6~2.8 in normal volcanic ash fall deposits. Even if they become soil, they are likely to have many voids like pumices.
Liquid and plastic limits	They generally consist of mainly coarse grained soils such as gravels and sand, both of them represent either lower water content.
Grading analysis Seiving only	If their uniformity coefficient, Uc, is greater than 10, it is said that they are well graded soils. However, according to this survey, Uc varies from 40 to 300, which shows a wide range of weathering from tuff spalling to disintegrated to soil.
2. Rock Property Test	**Results of Test**
Uniaxial compressive strength test	They were clearly divided into a 100kg/cm^2-below group and a 100kg/cm^2 or more one in unconfined compressive strength. The former was considered as soft rock, while the latter as hard rock. The Japan Highway Public Corporation also categorizes the rocks on the basis of 100kg/cm^2 in unconfined compression strength. In addition, according to the Stone Standards, hard rocks and soft rocks are classified as $>$ 500kg/cm^2 and $<$ 100kg/cm^2, respectively. The intermediate rocks between the two are classified as semi-hard rocks.
Unite weight test	Most of tuff has around 1.7t/m^3, while most of andesite has 2.5t/m^3 or more.

Refer to soil and rock test results lists by route and by slope shown in the next page.

Table 17.3.2 (1) NIC.1 Soil and Rock Test Results

Nic-1 (Km From MTI)	No.	Boring holes	Depth (m) of soil test	Grain size analysis Gravel (%)	Sand (%)	Fine grained soil (%)	Liquit limit (%)	Plastic limit (%)	Plastic index	Soil classification	natural water content (%)	Specific gravity	Unite weight (g/cm³)	Unconfined compression test (Kg/cm²)	Notes
60.9	01A290	1	1.3m	50	34	16	53	36	17	GM	17.40	2.14	2.63(11.45~11.50)	45.0(14.6~14.7m)	Andesite
73.2	01A280	boring 1	1.3m	0	28	72	46	30	16	ML	22.90	2.38	1.63(10.0~10.2m)	63.0(10.0~10.2m)	Tuff
			3.7m	12	39	49	65	43	22	SM	21.80	2.19	2.14(15.2~15.24m)	–	Andsite
		boring 2	1.2m	0	28	72	46	30	16	ML	23.00	2.24	1.39(6.14~6.35m)	–	Tuff
			2.2m	17	61	22	52	41	11	SM	–	2.24	1.85(14.6~14.7m)	31.0(6.2~6.3m)	Tuff
			4.5m	16	64	20	54	43	11	SM	–	–	Impossible		Tuff
		boring 3	1.0m	Impossible								–	1.53(9.8~9.9m)	51.85(9.5~9.7m)	Tuff
			14.2m	Impossible								–	1.53(14.0~14.2m)	66.0(14.1~14.3m)	Tuff
		boring 4	–	Impossible								–	Impossible		Tuff
		boring 5	1.4m	Impossible							21.75	–	–	16.0(10.8~10.9m)	Tuff
		boring 6	1.0m	Impossible								–	Impossible		Tuff
168.4	01A240	1	1.8m	40	37	23	50	28	22	GC	26.63	2.26	1.50(6.4~6.5m)	24.0(6.4~6.5m)	Tuff
			4.1m	0	80	20	42	35	7	SM	15.0	2.24	1.50(15.0~15.2m)	49.0(15.0~15.2m)	Tuff
168.6	01B230	1	1.0m	22	50	28	40	25	15	SC	–	2.36	1.57(3.5~3.7m)	36.0(3.5~3.6m)	Tuff
			1.8m	0	70	30	39	23	16	SC	8.90	2.25	1.45(15.0~~15.2m)	23.0(15.0~15.2m)	Tuff
171.3	01B170	1	1.6m	33	33	34	52	32	20	SM	8.98	2.90	1.82(6.3~6.4m)	144(6.1~6.3m)	Tuff
			6.4m	Impossible							8.74	2.40	2.62(15.0~15.1m)	256(14.8~15.0m)	Andesite
175.0	01B150	1	1.0m	21	54	25	47	29	18	SM	11.41	2.77	1.18(7.2~7.3m)	71.0(7.2~7.3m)	Tuff
			1.8m	39	47	3.5	35	26	9	SM	10.50	2.41	2.85(13.6~13.7m)	222(13.55~13.77m)	Andesite
176.0	01B120	1	2.7m	0	34	66	57	30	27	MH	5.76	2.35	2.09(11.1m)	–	Tuff
			8.3m	0	66	34	48	28	20	ML	4.65	2.23	2.14(14.0m)	86.0(14.0~14.1m)	Andesite
178.7	01A110	1	1.3m	27	31	42	54	31	23	SM	22.19	2.46	2.17(3.7~3.8m)	–	Tuff
			3.8m	Impossible							21.0	2.32	2.60(14.2~15.0m)	732(14.0~15.0m)	Andesite
204.7	01B070	1	2.7m	31	52	17	22	16	6	SC-SM	18.1	2.56	2.72(7.5m~7.6m)	445(7.4~7.6m)	Andesite
			7.6m	Impossible							19.0	–	2.60((14.6~14.7m)	354(14.6~14.8m)	Andesite
214.7	01A050	1	1.8m	29	52	19	35	23	12	SC	15.98	2.61	2.30(8.9~9.1m)	202(8.8~8.9m)	Welded Tuff
			2.3m	Impossible							4.03	2.58	2.19(15.0~15.1m)	255(15.1~15.2m)	Welded Tuff

Impossible:The core's Condition is Almost Pebble Type.

Exam.:Depth of Sample(4.0m~5. 0m)

Table 17.3.2 (2) NIC.3 Soil and Rock Test Results

Nic-3 (Km)From Sebaco	No.	Boring holes	Depth (m)of soil test	Grain size analysis			Liquit limit(%)	Plastic limit(%)	Plastic index	Soil classification	natural water content(%)	Specific gravity	Unite weight(g/cm³)	Unconfined compression test (Kg/cm²)	Notes
				Gravel(%)	Sand(%)	Fine grained soil(%)									
3.9	03B420	1	8.2m	0	36	64	65	35	30	MH	13.5	2.11	Impossible		Tuff
			11.0m	35	36	29	73	25	48	SC	17.6	2.18	1.86(12.2～12.3m)	51.0(12.2～12.3m)	Tuff
6.9	03B400	boring 1	-	Impossible								-	2.40(8.0～8.23m)	523(8.0～8.2m)	Andesite
				Impossible								-	2.29(10.7～11.0m)	431(10.7～11.0m)	Andesite
		boring 2	1.0m	17	42	41	59	40	19	SM	16.47	-	1.68(4.9～5.3m)	97(4.9～5.3m)	Tuff
			3.2m	22	24	54	50	37	13	MH	20.45	-	Impossible	57.0(9.5～10.2m)	Tuff
7.4	03B370	1	2.9m	0	34	66	62	30	32	CH	24.24	2.66	Impossible		Tuff
			5.6m	Impossible							29.48	2.34	Impossible		Tuff
22.1	03B320	1	8.7m	64	27	9	35	23	12	GW-GC	24.0	2.93	Impossible		Tuff
			15.0m	7	45	48	70	39	31	SM	27.80	2.17	Impossible		Tuff
32.7	03B240	1	6.4m	26	19	55	63	20	43	CH	71.0	2.30	Impossible		Tuff
			7.6m	Impossible								-	Impossible		Tuff
32.9	03C230	boring 1	3.6m	45	36	19	36	26	10	GM	23.02	2.41	Impossible		Tuff
			9.3m	0	24	76	53	37	16	MH	31.04	2.58	Impossible		Tuff
		boring 2	5.6m	18	60	22	50	23	27	SC	-	2.60	2.20(4.7～5.6m)	Impossible	Tuff
		boring 3	1.4m	15	33	52	53	29	24	MH	-	2.41	1.91(6.4～7.8m)	274(5.6～5.9m)	Tuff
			7.8m	19	24	57	60	34	26	MH	-	-	Impossible	242(7.8～8.0m)	Tuff
35.2	03E170	boring 1	2.0m	Impossible								-	2.39(1.8～2.0m)	387(1.8～2.0m)	Welded Tuff
			6.1m	Impossible								-	2.49(5.9～6.1m)	392(5.9～6.1m)	Andesite
		boring 2	5.0m	22	48	30	41	24	17	SC	22.9	2.51	Impossible		Tuff
			14.8m	0	96	4	NP	NP	NP	SW	11.7	2.49	Impossible		Tuff
		boring 3	1.8m	22	48	30	41	24	17	SC	-	2.24	2.80(2.0～2.43m)	Impossible	Tuff
		boring 4	13.6m	0	96	4	NP	NP	NP	SW	-	2.81	Impossible		Tuff
			20.0m	Impossible								2.45	2.39(19.6～20.0m)	157(19.6～20.0m)	Tuff
35.9	03C160	boring 1	1.0m	26	46	28	33	19	14	SC	19.8	2.46	2.52(1.0～2.0m)	784(5.4～5.6m)	Andesite
			11.12m	0	33	67	69	31	38	CH	18.68	2.48	2.77(8.4～11.1m)	Impossible	Tuff
		boring 2	5.6m	0	34	66	64	25	39	CH	34.95	2.59	Impossible		Tuff
			15.4m	0	41	59	42	22	20	CL	23.01	2.75	Impossible		Tuff
		boring 3	1.3m	10	70	20	43	22	21	SC	24.07	2.34	2.65(10.7～12.5m)	Impossible	Tuff
			4.9m	8	62	30	46	33	13	SM	17.41	2.24	2.42(12.9～13.1m)	784(12.9～13.1m)	Andesite
38.9	03C150	boring 1	3.4m	0	34	66	64	31	23	MH	12.78	2.14	Impossible		Tuff
			8.2m	0	33	67	75	35	40	MH	24.26	2.09	Impossible		Tuff
		boring 2	3.7m	0	52	48	35	27	8	SM	18.30	2.20	2.54(6.2～6.4m)	768(6.2～6.4m)	Tuff
			8.2m	13	34	53	53	34	19	MH	16.80	2.23	2.54(10.3～10.5m)	760(10.3～10.5m)	Andesite
39.4	03C140	boring 1	8.4m	0	42	58	62	37	25	MH	46.92	2.22	Impossible		Tuff
			13.6m	7	62	31	42	32	10	SM	40.04	2.29	Impossible		Tuff
		boring 2	4.6m	0	17	83	69	52	17	MH	20.62	-	Impossible		Tuff
			9.1m	51	35	14	39	24	15	GC	17.83	-	Impossible		Tuff
40	03B120	1	6.1m	0	57	43	57	31	26	SM	15.6	2.78	Impossible		Tuff
			9.1m	0	65	35	44	29	15	SM	18.6	2.66	Impossible		Tuff

Impossable:The Core's Condition is Almost Pebble or Sandy Type. Exam.:Depth of Sample(4.0～5.0m)

Table 17.3.2 (3) NIC.5 Soil and Rock Test Results

Nic-5 (Km)From Matagalpa	No.	Boring holes	Depth (m) of	Grain size analysis Gravel(%)	Sand(%)	soil classificat	Liquit limit(%)	Plastic limit(%)	Plastic index	Soil classification	natural water content(%)	Specific gravity	Unite weight(g/cm³)	Unconfined compression test (Kg/cm²)	Notes
24.6	05A010	boring 1	4.5	33	47	20	54	29	25	SC	27.58	2.81	2.03(4.3~4.5m)	196(4.3~4.5m)	Andesite
			8.9	Impossible							18.34	2.75	1.88(8.60~8.80m)	91(8.6~8.9m)	Andesite
		boring 2	1.0m	Impossible							39.58	–	2.07(9.1~9.2m)	Impossible	Andesite
			12.8m	Impossible							Impossible	–	1.81(12.6~13.0m)	53.0(12.6~12.9m)	Tuff
		boring 3	1.0m	Impossible							23.7	–	Impossibke		Andesite
			1.8m	Impossible							27.3	–	2.55(13.0~13.4m)	52.0(14.6~14.8m)	Andesite
		boring 4	12.2m	Impossible							Impossible	–	2.43(12.2~12.3m)	Impossible	Andesite
		boring 5	16.5m	Impossible							17.1	–	2.61(16.5~17.6m)	Impossible	Tuff
			21.8m	Impossible							8.11	–	2.71(20.0~21.8m)	784(21.6~21.8m)	Tuff

Exam.:Depth of Sample(4.0~5.0m)

Table 17.3.2 (4) NIC.15 Soil and Rock Test Results

Nic-15(Km)From Las Manos	No.	Boring holes	Depth (m) of soil test	Grain size analysis Gravel(%)	Sand(%)	Fine grained soil(%)	Liquit limit(%)	Plastic limit(%)	Plastic index	Soil classification	natural water content(%)	Specific gravity	Unite weight(g/cm³)	Unconfined compression test (Kg/cm²)	Notes
9.9	015E010		1.8m	48	48	4	NP	NP	NP	SW	7.98	2.47	2.95(2.4~2.6m)	634(2.4~2.6m)	Granite
		boring 1	–	Impossible								–	2.77(4.8~4.9m)	Impossible	Granaite
			–	Impossible								–	2.31(4.8~4.9m)	Impossible	Granaite
		boring 2	1.0m	Impossible								2.41	2.56(4.8~4.9m)	120(4.8~4.9m)	Granite
		boring 3	1.4m	Impossible								–	2.64(1.01.4m)	777(1.4m)	Porphyrite
			4.4m	Impossible								–	2.85(4.0~4.4m)	671(4.0~4.4m)	Porphyrite
		boring 4	4.1m	Impossible								–	2.42(4.0~4.1m)	Impossible	Granite
			4.7m	Impossible								–	2.20(4.7m)	Impossible	Granite,Porphyrite
		boring 5	2.2m	Impossible								–	2.85(2.2~2.5m)	769(2.2~2.5m)	Granite,Porphyrite
			4.4m	Impossible								–	2.4(4.34.4m)	494(4.3~4.4m)	Granite,Porphyrite
		boring 6	2.6m	Impossible								–	2.71(2.2~2.6m)	Impossible	Granite,Porphyrite
			4.9m	Impossible								–	2.68(4.8~4.9m9	478(4.8~4.9m)	Granite,Porphyrite
11.1	015E020	boring 1	2.3m	2	87	11	NP	NP	NP	SW-SM	11.28	2.59	2.76(4.6~4.7m)	802(4.6~4.7m)	Granite
			4.6m	1	81	18	29	21	8	SC	12.5	2.66	2.74(5.0~5.5m)	481(5.0~5.5m)	Granite
11.7	015E050	1	2.7m	19	65	16	26	16	11	SC	6.63	2.62	2.74(3.7~4.0m)	222(3.7~3.9m)	Granite
			5.2m	30	56	14	30	27	3	SM	10.13	2.47	2.17(5.8~15.0m)	Impossible	Granite
13.6	015E060	1	4.6m	26	62	12	25	19	6	SW-SC	5.40	2.55	Impossible	785(5.5~6.0m)	Granite
			14.8m	Impossible							6.79	2.69	3.03(14.7~14.8m)	481(14.7~14.8m)	Granite

Impossible:The Core's Condition is Almost Pebble or Sany Type. Exam.:Depth of Sample(4.0~5.0m)

Table 17.3.2 (5) NIC.26 Soil and Rock Test Results

Nic-26 (Km)From Nic-1	No.	Boring holes	Depth (m)	Grain size analysis Gravel (%)	Sand (%)	Fine grained soil (%)	Liquit limit (%)	Plastic limit (%)	Plastic index	Soil classification	natural water content (%)	Specific gravity	Unite weight (g／cm³)	Unconfined compression test (Kg/cm²)	Notes
24.7	026A060	1	1.4m	47	36	17	42	20	22	GC	15.9	2.64	2.34(4.0～4.2m)	523(4.0～4.2m)	Andesite
			–	Impossible							12.1	2.58	2.50(8.9～9.1m)	457(8.9～9.1m)	Andesite
29.3	026A100	1	1.0m	0	23	77	60	27	33	CH	21.22	2.37	1.79(5.2～5.3m)	78(5.2～5. 3m)	Tuff
			2.3m	0	19	81	50	35	15	MH	18.5	2.35	2.00(11.0～11.25m)	78(11.0～11.3m)	Tuff
33.6	026A130	1	4.6m	0	68	32	50	21	29	SC	–	2.31	1.79(4.3～4.6m)	52(4.3～4.6m)	Tuff
			12.5m	Impossible								–	2.09(12.2～12.5m)	89.0(9.5～9.8m)	Tuff
34.0	026B140	1	1.8m	5	63	32	38	21	17	SC	17.55	2.78	2.21(3.4～3.5m)	Impossible	Tuff
			7.0m	Impossible								–	2.04(6.9～7.0m)	Impossible	Tuff
34.2	026A150	boring 1	1.0m	27	45	28	43	19	24	SC	–	2.6	2.12(2.7～2.8m)	Impossible	
			6.4m	35	34	31	48	25	23	GC	–	2.36	2.48(9.7～9.8m)	Impossible	Andesite
		boring 2	1.0m	30	69	1	NP	NP	NP	SW	–	2.82	2.26(2.9～3.1m)	248(2.9～3.1m)	Andesite
			–	Impossible								–	2.26(5.4～5.6m)	256(5.4～5.6m)	Andesite
		boring 3	–	Impossible								–	2.09(6.1～6.2m)	65(4.1～4.3m)	Tuff
			–	Impossible								–	2.40(7.0～7.2m)	321(6.1～6.2m)	Welded Tuff
		boring 4	1.0m	20	57	23	44	20	24	SC	–	2.91	2.43(11.3～11.6m)	577(11.3～11.6m)	Aggromerate
			4.9m	14	61	25	48	28	20	SM	–	2.56	Impossible		Tuff
		boring 5	1.0m	39	32	29	45	31	14	GM	–	2.36	Impossible		Tuff
			8.2m	36	27	27	47	25	22	GC	–	2.64	Impossible		Tuff
37.0	026B160	1	1.0m	25	52	23	44	24	20	SC	14.9	2.53	1.73(4.0～4.2m)	98(4.0～4.2m)	Tuff
			2.3m	26	56	18	37	25	12	SM	12.13	2.27	1.73(10.5～10.7m)	43(10.4～10.7m)	Tuff
45.5	026B210	1	7.6m	20	44	36	33	21	12	SC	17.52	2.47	Impossible	53.0(14.6～14.9m)	Tuff
			14.0m	6	74	20	40	31	9	SM	12.53	2.47	Impossible	23(15.4～15.6m)	Tuff

Impossible:The Core's Condition is Almost Pebble or Sanndy Type. Exam.:Depth of Sample(4.0m～5. 0m)

17.3.3 Analysis on Tuff and Andesite Groups

While tuff rocks repeat their spalling and small-scale collapse, many cracks-bearing andesite rocks overlaid become overhang, which contributes to rock fall. Some tuff groups, however, hardly suffer from their spalling and small-scale collapse. For this reason, the strength, structural or lithologic difference between the two groups can be found (Figure17.3.2). So, it was reviewed whether it would be possible to determine bedrock classification-method applicable to bedrocks in Nicaragua, in order to consider stability, execution availability or measures against slope surfaces on tuff and andesite rock cut.

Various correlations between the results of laboratory tests were examined, but the results on which bedrocks could be grouped were not obtained. If correlation with another physical characteristics on bedrocks had been identified, in addition to unit weight such as crack coefficient, water absorption or P-S wave velocity, more bedrock characteristics would have been clarified. In this study, the following method was employed.

When incorporating slope condition where test samples were collected into the following correlation diagram of unit weight and uniaxial compression strength, the clear difference between slope collapses was considered on reaching uniaxial compression strength of 100kg/cm^{2}and unit weight of 2.5t/cm^3

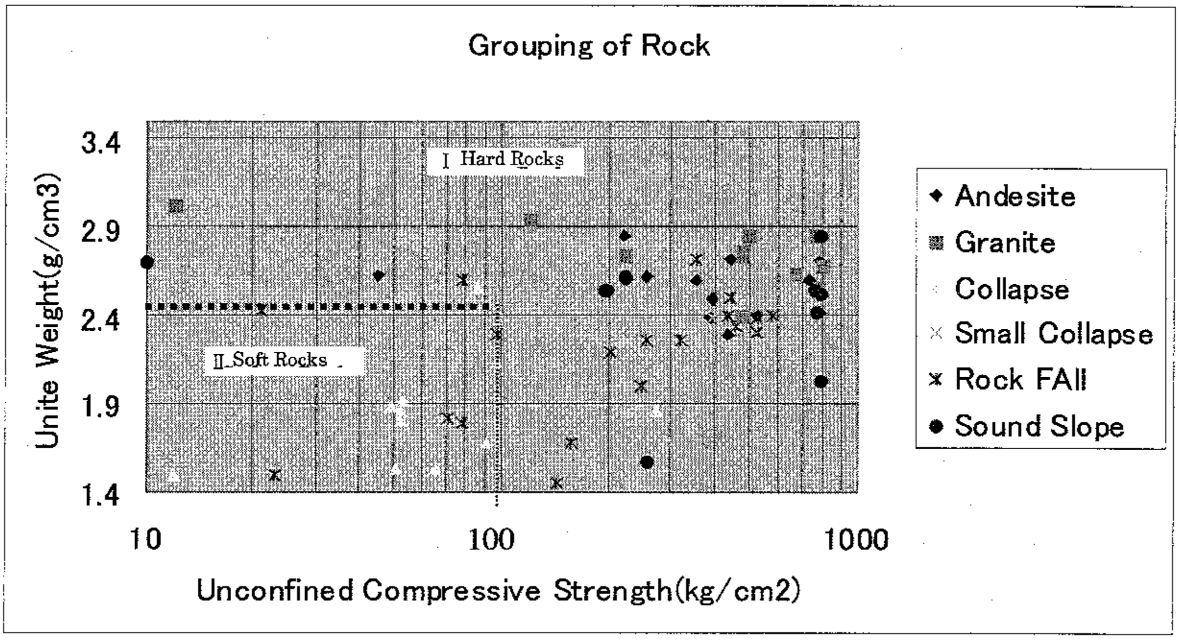


Figure 17.3.2 Grouping of Rock

Based on this figure, it was determined as follows;

Group I	Andesite, Granite, Welded tuff, Qquartz sandstone, and Paleozoic/ Mesozoic stratums, Solider mudstone, Shale
Group II	Terteary tuff/mudstone/shale including metamorphic rock.
Group III (Sediment)	Granite weathered soil, talus deposits.

It was found from this chart that rock fall would occur irrespective of bedrock types and cut gradients if the conditions for falling stones were provided after a loosening of a slope surface, expansion of its crack and development of weathering. Therefore, it can be understood that there is a construction method on which free fall is converted to rolling fall by berms and flattering slope gradient through Ritche's rock fall prevention groove works described in the former section, although rock fall can be avoided with slope protection works.

17.3.4 Analysis on Loosening Rate in a Weathering Layer

Focusing on the existence of very loose weathered layers with N＜10 in N-values based on the standard penetration test, it was determined that the layers were determined to be probably part of a weathered layer of stress released at least with cut. Information for those judgment was based on N-values resulting from boring conducted at a slope toe, not at the upper part or intermediate position of the slope affected by weathering force. Lapsed days after completion of construction were obtained from MTI's execution records (Table17.3.3).

Table 17.3.3 Execution Completion Year

Execution completion year		
Road	Section	
NIC.1	San Benito～Sebaco	1976
	Sebaco～Buenos Aires	81k 1961
	Buenos Aires～Somoso	31k 1957
	Condega～Somoso	65k 1960
	Sebaco～Condega	78k 1959
	El Espino～Buenos Aires	56k 1958
	Maderas～Sebaco	1943
	Tipitapa～Maderas	1940
NIC.5	Matagalpa～El Tuma	32k 1955
NIC.3	Matagalpa～Jinotega	1955
NIC.15	Jalaguina～Ocotal	1970
	Ocotal～Las Manos	1972
NIC.26	Tega～Los Zarzales	1965
	Los Zarzales～San Isidro	1966

The loosening rate in a weathered layer,

$\alpha = a / \log T$

α : Loosening rate

a : N<10 in thickness (cm)

T : Days from completion to the present time

These calculation conditions were summarized in Table 17.3.4 Analysis on loosening rate. This result is shown as followed:

$\alpha > 12$ Fast loosening group

$\alpha < 9$ Slow loosening group

When soil characteristics of a fast loosening group are examined, its liquid limit lies within 50%- 70%. This group with 50- 70%- liquid limit contains mainly CH (volcanic cohesive soil: Fat clay)- MH (volcanic cohesive soil : Silt with Sand) and SC (Clayey Sand) . Also in Japan, it is said that most of weathered soil represent their liquid limit of 60%<. In addition, it is reported that natural absorptive expansion coefficient for $\alpha > 12$ represents1.5 or more while one for $\alpha < 9$ provides less than 1.5. Considering these soil characteristics, it was determined that a $\alpha > 12$ group has faster slaking than a $\alpha < 9$ group, or may be a group that has a lot of spalling on a slope, leading to rock fall and collapse. Therefore, secondary alteration-based lithology was classified as Table17.3.4.

Table 17.3.4 Secondary Alteration-Based Lithology Category

Category	Descriptions	Loosing rate
A	Without any protection, it always causes secondary strength reduction after cutting.	$\alpha > 12$
B	Under normal conditions, its secondary strength reduction is low and does not damage slope stability.	$\alpha < 9$

Based on this category, local bedrocks were classified into four groups, I B, II B, I A and II A. Finally, adding sediment III to them, they were totally grouped into 5. For more information, refer to analysis data summarized in Table 17.3.5(1)- (5) (The above data: text of the 1976 report issued by Japan Highway Corporation Laboratory).

17.3.5. (1) NIC.1 Analysis on Loosened Speed in a Weathering Layer

NIC-1

ID NO.	Distance from MTI(km)	Thickness of Weathering Bed (m) N:Value of Standard Penetration Test						Type of Soil's Clafissication	Clafissified Type of Weathering Bed	Passing Days from Completion: T	Velocity of Weathering α = a/logT(a:cm)	Sub-Clafissication α>12 A α<9 B	Clafissication of Rock
		N>5～9 : a	30>N	50>N>30	Subtotal(m)	N>>50～100	Total(m)						
A290	60.9	0.1	0.9	0.0	1.0	1.8	2.8	GM	Ⅲ	21.240	2.3	B	Ⅱ B
A280	73.2	0.3	0.7	1.5	2.5	1.6	4.1	ML～SM	Ⅲ	〃	6.9	B	Ⅱ B
A240	168.4	0.2	1.8	0.5	2.5	1.8	4.3	SM～GL	Ⅲ	15.480	4.8	B	Ⅱ B
B230	168.6	0.6	0.4	0.5	1.5	1.7	3.2	SC～SM	Ⅲ	〃	14.3	A	Ⅱ A
B170	171.3	0.0	0.0	1.0	0.5	2.7	3.2	SM	Ⅲ	〃	0.0	B	Ⅰ B
B150	175.0	0.2	0.3	0.0	0.5	2.7	3.2	SM	Ⅲ	〃	4.8	B	Ⅰ B
B120	176.2	0.4	0.7	0.5	2.0	2.1	4.1	ML～MH	Ⅲ	〃	9.1	B	Ⅱ B
A110	178.7	0.8	0.7	0.0	1.5	4.2	5.7	SM	Ⅲ	〃	19.1	A	Ⅰ A
A070	204.7	0.1	0.9	0.0	1.0	3.6	4.6	SC～SM	Ⅲ	15.120	2.4	B	Ⅰ B
A050	214.7	0.3	0.2	0.0	0.5	2.7	3.2	SC	Ⅲ	〃	7.2	B	Ⅰ B

Table 17.3.5(2) NIC.3 Analysis on Loosened Speed in a Weathering Layer

NIC-3

ID NO.	Distance from Sebaco(km)	Thickness of Weathering Bed (m) N:Value of Standard Penetration Test						Type of Soil's Clafissication	Clafissified Type of Weathering Bed	Passing Days from Completion: T	Velocity of Weathering α = a/logT(a:cm)	Sub-Clafissication α>12 A α<9 B	Clafissication of Rock
		N>5～9 : a	30>N	50>N>30	Subtotal(m)	N>>50～100	Total(m)						
B420	3.9	0.3	0.3	0.0	t	0.0	0.6	—	Ⅲ	17.280	7.1	B	Ⅱ B
B400	6.9	0.4	1.6	1.0	3.0	4.0	7.0	MH～SC	Ⅲ	〃	9.5	B	Ⅱ B
B370	7.4	0.2	0.3	0.0	0.5	10.5	11.0	CH～SM	Ⅲ	〃	4.8	B	Ⅱ B
B320	22.1	0.1	0.9	4.0	5.0	10.0	15.0	SM～GW	Ⅲ	16.920	2.4	B	Ⅰ B
B240	32.7	0.3	0.2	0.0	0.5	0.5	1.0	CH	Ⅲ	〃	7.1	B	Ⅱ B
B230	32.9	0.6	3.4	1.5	5.5	3.7	9.2	MH～GM	Ⅲ	〃	14.3	A	Ⅰ A
B170	35.2	2.7	0.3	3.0	6.0	8.5	14.5	SC～SW	Ⅲ	〃	64.3	A	Ⅱ A
B160	35.9	0.9	4.1	4.5	9.5	5.5	15.0	CH～SC	Ⅲ	〃	21.4	A	Ⅰ A
B150	38.9	0.0	1.0	0.0	1.0	4.0	5.0	MH～SM	Ⅲ	〃	0.0	B	Ⅰ B
B140	39.4	1.0	1.7	11.0	13.7	1.3	15.0	MH～SM	Ⅲ	〃	23.8	A	Ⅱ A
B120	40.0	0.0	0.0	0.0	0.0	0.0	0.0	SM	Ⅲ	〃	0.0	B	Ⅱ B

17.3.5. (3) NIC.5 Analysis on Loosened Speed in a Weathering Layer

NIC-5

ID NO.	Distance from Matagalpa(km)	Thickness of Weathering Bed (m) N:value of Standard Penetration Test						Type of Soil's Clafissication	Clafissified Type of Weathering Bed	Passing Days from Completion: T	Velocity of Weathering $\alpha =$ a/logT(a:cm)	Sub-Clafissication $\alpha>12$ A $9<\alpha$ B	Clafissication of Rock
		N>5~9 : a	30>N >10	50>N>30	Subtotal(m)	N>>50~100	Total(m)						
A010	24.6	0	0	1	1	1	2	SC	—	16.920	—	—	Ⅰ B
		0	1	4	4	19	23	—	Ⅲ	〃	0	B	Ⅱ B

17.3.5. (4) NIC.15 Analysis on Loosened Speed in a Weathering Layer

NIC-15

ID NO.	Distance from Matagalpa(km)	Thickness of Weathering Bed (m) N:value of Standard Penetration Test						Type of Soil's Clafissication	Clafissified Type of Weathering Bed	Passing Days from Completion: T	Velocity of Weathering $\alpha =$ a/logT(a:cm)	Sub-Clafissication $\alpha>12$ A $9<\alpha$ B	Clafissication of Rock
		N>5~9 : a	30>N >10	50>N>30	Subtotal(m)	N>>50~100	Total(m)						
E010	9.9	0	1	1	2	3	5	SW	Ⅲ	11.520	—	—	Ⅰ B
E020	11.1	0	4	1	5	2	7	SM~SW	Ⅲ	〃	—	—	Ⅰ B
E050	11.7	0	0	0	0	5	5	SC~SM	Ⅲ	〃	—	—	Ⅰ B
E060	13.6	0	4	0	4	1	5	SC~ SW	Ⅲ	〃	—	—	Ⅰ B

17.3.5. (5) NIC.26 Analysis on Loosened Speed in a Weathering Layer

NIC-26

ID NO.	Distance from NIC-1(km)	Thickness of Weathering Bed (m) N:Value of Stanndard Penetration Test						Type of Soil's Clafissication	Clafissified Type of Weathering Bed	Passing Days from Completion: T	Velocity of Weathering $\alpha =$ a/logT(a:cm)	Sub-Clafissication $\alpha>12$ A $\alpha<9$ B	Clafissication of Rock
		N>5~9 : a	30>N	50>N>30	Subtotal(m)	N>>50~100	Total(m)						
A060	24.7	0.1	0.4	0.0	0.5	1.5	2.0	GC	Ⅲ	12.960	2.4	B	Ⅰ B
A100	29.3	0.5	0.5	0.5	1.5	2.0	3.5	CH~MH	Ⅲ	〃	12.2	A	Ⅱ A
A130	33.6	0.5	0.0	0.0	0.5	1.0	1.5	SC	Ⅲ	〃	12.2	A	Ⅱ A
B140	34.0	0.6	6.4	1.0	8.0	5.0	13.0	SC	Ⅲ	〃	14.6	A	Ⅱ B
A150	34.2	0.3	1.2	1.0	2.5	2.5	5.0	SC~GC	Ⅲ	〃	7.3	B	Ⅱ B
B160	37.0	0.1	0.0	0.0	0.1	2.2	2.3	SM~SC	Ⅲ	〃	2.4	B	Ⅱ B
B210	45.5	0.1	0.0	0.0	0.1	2.0	2.1	SM~SC	Ⅲ	〃	2.4	B	Ⅱ B

For more information, visual lithology for principle category signs of I, II and III of this bedrock is as Table17.3.6.

Table 17.3.6 Hardness-Based on Lithology Classification

Lithology Class	Rock Appearance	Hammer Approach	Representative Rock Types
I	It is hard and dense, and has fresh colors, hardly including weathered brown portinos. Rock structure can be perfectly found.	It has clear sound or dull sound. The head of a hammer cannot penetrate into it due to its hardness. Mudstone and siltstone can be broken only with difficult by hand. Block rock samples can be obtanied.	Plutonic rocks such as granite, fresh metamorphic rocks such as schist group, volcanic rocks such as andesite, rhyolite, agglomerate and basalt, hard sedimentary rocks such as sandstone
II	Softened rocks by weathering, low consolidation rocks. Fine structure of the rocks is difficult to identify.	It sounds dull when being struck. The head of a hammer is struck into it. It is easily broken, and its small pieces are broken with fingers. Large samples are difficult to take.	Terteary mudstone, shell, siltstone, and tuff. Weathered parts of the Class I rocks
III	Rocks strongly exposed to weathering and alteration, or they show no rock feature. They should be treated as sediment.	When being struck by a hammer, it will be broken as if being collapsed, or the hammer will stick into it. The head of a hammer is easily struck into it. A piece of rock is crushed with figures.	Unconsolidated layers, bedrocks weathered into soil, and talus

17.3.5 Analysis on Slope Gradients of a Loosened Area in a Weathering Layer

Based on the above bedrock category, slopes for disaster prevention at 55 points with some problems were examined in terms of stability, and their results were shown in the following table. The horizontal axis shows bedrock categories and slope height, while the vertical axis indicates slope gradients.

In particular, bedrock categories in the horizontal axis are originally independent of each other. The axis shows that rock quality deteriorates from the left hard rock slope to the right direction, a trend of increasing loosened quantity. Sediment on the right end is an unconsolidated layer that consists of mainly bedrock weathered sediment. The down arrow described there indicates a slope that was re-cut after its collapse and the slope whose gradient was modified. Also, the vertical axis shows only slope gradients. Concerning to the level of changed conditions of slopes, refer to explanatory notes.

It is a slope that is originally located in a weathered layer. Even if hard bedrocks appear on the surface, they rarely cover across the slope. Bedrock weathering modes on a continuous slope vary, while a loosened area is subject to local variations. So, it is necessary to determine a loosened area that bedrocks belong to.

Another feature in this volcanic rock area is that the concept for slope stability depends on distribution modes of tuff rocks or alternate modes of tuff and andesite rocks. That is to say, tuff rocks with higher loosening rate and with their wide appearance on slopes frequently deteriorate slope stability. On the contrary, tuff rocks, which alternately co-exist with andesite rocks and are partially by weathering force, increase their slope stability. So, this geologic structure could be reflected in measures against slope surfaces.

A step-wise line in the chart shows a proposed value that around 5 ° can be added to, when tuff rocks alternately co-exist with andesite rocks and when tuff spalling little occurs.

Figure 17.3.3 shows the results of analysis on slope gradients of a loosened area in a weathered layer.

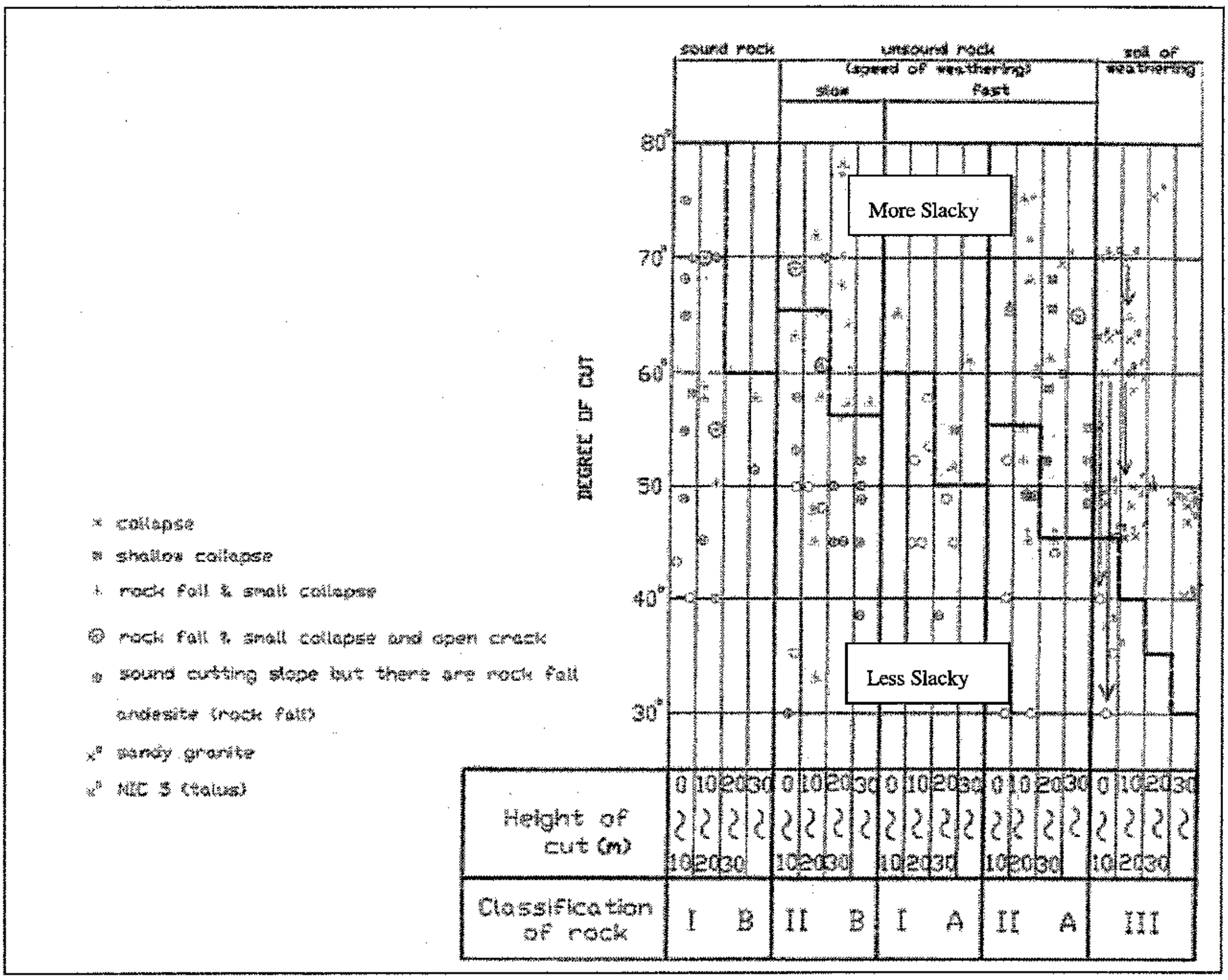


Figure 17.3.3 Analysis on Slope Gradients of a Loosened Area in a Weathering Layer

17.3.6 Checking Current Slope Gradients

When checking actual slope gradients with the use of the gradients based on analysis on slope gradients, what kind of results would be obtained? It shall be checked if a real gradient is steep. If it is steep, it will be necessary to consider measures for slope stabilization, establishing berms, re-cutting the slope and determining a stable gradient. The checking results were summarized in Table 17.3.7 (1)- (4). In these tables, ○ mark shows no particular problems about slope gradients, while × mark indicates steep gradient points requiring some measures against slope surfaces and — is marked on non-analysis points.

※ is marked on potential danger points of rock fall and on points requiring stability calculation, and rock fall and stability analyses were conducted. These results were reflected in the design.

For points with two slope gradients in the table, where bedrock categories cross, their gradients must be naturally slow according to their loosened conditions.

These results were shown in Table 17.3.8.

Table 17.3.7. (1) NIC.1 Suitability of Current Slopes

NIC-1

ID NO.	Distance from MTI(km)	Present State		Type of Disaster		Score		Grouping of Rock	θ' by mean of new Analysis	Judgment of Stability	Stability Analysis	Analysis of Rock Fall
		H(m)	θ (°)	First Phase	Second Phase	First Phase	Second Phase					
A290	60.9	20~40	50~45	RF	RF	70	78	Ⅱ B	55	○	—	○
A280	73.2	11~25	75 ~45	RF	RF	78	84	Ⅲ/ⅡB	45~35/ 65~55	×/×	○	—
A240	168.4	15~18	57~45	RF	RF	84	84	Ⅱ A	60	○	—	○
A230	168.6	13~33	65~55	RC	RC	72	75	Ⅱ B	65~55	○	—	○
A170	171.3	13~37	60~57	RC	RC	78	81	Ⅰ B	75~60	○	—	—
B150	175.0	10~15	70~60	RC	RC	76	79	Ⅰ B	75	○	—	—
B120	176.2	28~50	75~60	RC	RC	74	76	Ⅱ A	45	×	—	—

Table 17.3.5(2) NIC.3 Suitability of Current Slopes

NIC-3

ID NO.	Distance from Sebaco(km)	Present State		Type of Disaster		Score		Grouping of Rock	θ' by mean of new Analysis	Judgment of Stability	Stability Analysis	Analysis of Rock Fall
		H(m)	θ (°)	First Phase	Second Phase	First Phase	Second Phase					
B400	6.9	5~18	30~48	RC	RC	72	75	ⅡA/(ⅡB)	45~55/60	○	—	○
B370	7.4	10~18	53~35	RC	RC	80	80	Ⅲ /(Ⅱ B)	40/60	○	—	○
B320	22.1	7~9	75~48	RC	RC	74	76	Ⅰ B	65	×	—	—
C230	32.9	10~16	60~45	SS	SS	73	73	Ⅲ /(ⅠA)	40/55	×	○	—
E170	35.2	10~26	40~67	DF	DF &RF	83	83	ⅡA/(ⅠA)	50/55	×	—	○
C150	38.9	20~29	48~50	SS	SS	90	90	Ⅲ/(ⅠB)	40/65	×	○	—
C140	39.4	15~26	77~62	SS	SS	90	90	Ⅲ/(ⅡA)	40/50	×	○	—

Table 17.3.7. (3) NIC.5 Suitability of Current Slopes

NIC-5												
ID NO.	from MTAGALPA (km)	Present State		Type of Disaster		Score		Grouping of Rock	θ' by mean of new	Judgment of Stability	Stability Analysis	Analysis of Rock Fall
		H(m)	θ(°)	First Phase	Second Phase	First Phase	Second Phase					
A010	24.6	20~37	48~41	RF	RF	76	80	Ⅲ/ⅡA	45~35	×	○	○

Table 17.3.7. (4) NIC.26 Suitability of Current Slopes

CNI-26												
ID NO.	Distance from NIC-1 (km)	Present State		Type of Disaster		Score		Grouping of Rock	θ' by mean of new	Judgment of Stability	Stability Analysis	Analysis of Rock Fall
		H(m)	θ(°)	First Phase	Second Phase	First Phase	Second Phase					
A060	24.7	10~14	63~57	RF	RF	70	78	ⅠB	75	○	—	○
B140	34	20~33	68~55	RC	RC	80	80	ⅡA/ⅡB	60~45	×	○	○
A150	34.2	30~56	55~48	RC	RC	85	87	ⅡA/ⅠB	65~45	×	—	○
B160	37	10~23	70~58	RC	RC	71	71	ⅡB	65~55	△	—	○

Based on the above results, calculation conditions used for rock fall and stability analyses are presented in Table 17.3.8.

Table 17.3.8 Rock Fall and Stability Calculation Conditions List

Design conditions list

	ID	Distance	Current slope gradient	Design slope gradient	Rock fall analysis point: Height (m)	Gradient (°)	Rock size	Rock type	Density (t/m3)	Stability analysis point: γ (t/m3)	C (t/m2)	φ(°)	Groundwater level
NIC-1	A290	60.9	45～52	○	19,38	50,52	1×1×0.8 0.5×0.5×0.5	Andesite Ⅱ B	2.5	-			
	A280	73.2	45～75	40			-			1.8	1.5	Reverse analysis	At normal and rain
	A240	168.4	45～57	○	18	50,52	1×1×0.8 0.5×0.5×0.5	Andesite Ⅱ B	2.5	-			
	B230	168.6	40～65	○	15,33	50,65	2.0×1.5×0.5 1×1×0.8 0.5×0.5×0.5						
	B170	171.3	42～70	60	The above results can be applied .					-			
	B150	175.0	50～90	70									
	B120	176.2	50～70	55									
NIC-3	B400	6.9	33～90	60	18	33, 45, 53	2.0×1.5×0.5 1×1×0.8 0.5×0.5×0.5	Tuff Ⅱ B	1.7	-			
	B370	7.4	45～90	60									
	B320	22.1	48～75	45	-					-			
	C230	32.9	48～60	55	-					1.8	1.0	Reverse analysis	At normal and rain
	E170	35.2	45～62	55	There are aplied The Result of The Above.					-			
	C150	38.9	45～65	45	-					1.8	1.0	Reverse analysis	At normal and rain
	C140	39.4	45～60	45									
NIC-5	A010	24.6	41～48	35	20,35	35,40,45	2.0×1.5×0.5 1×1×0.8 0.5×0.5×0.5	Andesite Ⅱ B	2.5	2.0	1.5	Reverse analysis	At normal and rain
NIC-26	A060	24.7	53～63	○	14	56	1×1×0.8 0.5×0.5×0.5	Tuff Ⅱ B	1.7	-			
	B140	34.0	50～60	40,55	14.39	40.55	2.0×1.5×0.5 1×1×0.8 0.5×0.5×0.5			1.9	1.5	Reverse analysis	At normal and rain
	A150	34.2	48～70	55	34,56	48,55	2.0×1.5×0.5 1×1×0.8 0.5×0.5×0.5			-			
	B160	37.0	53～70	○	22	48.60	1×1×0.8 0.5×0.5×0.5			-			

17.3.7 Stability Analysis

In general, slope collapse in a surveyed area artificially occurs, where a slope slides in a chair shape as a whole block. In order to confirm the existence of landslide terrains causing slope collapse, an aerial photograph was interpreted. There were gently inclined grounds forming talus in an about 4 km-hinterland across 160- C140 of NIC.3, but movement traces of Tertiary typical slide scarps and blades could not be continuously observed. For this reason, this area cannot be said to be a landslide-prone zone. However, it consists of alteration-based deteriorated tuff rocks, and shallow landslide occurred when their weathered layer fluidized at heavy rain, can be seen. Since landslide depth could be almost estimated from collapse terrains appearing on a survey cross-section, cohesion was presumed from landslide depth. The following cohesion is estimated from landslide depth in general use.

Failure surface depth (m)	Cohesion (t/m2)
5	0.5
10	1.0
15	1.5
20	2.0
25	2.5
30	3.0

Furthermore, if a landslide type is identified from local topographical and geological features and if it is determined on the basis of a field survey whether landslide is active or not, lands with current safety factor shown in **Table17.3.9** can be roughly identified.

Table 17.3.9. Current Factor of Safety

	Bedrock landslide	Weathered rock landslide	Colluvial deposit landslide	Clayer soil landslide
Unmoving	1.10	1.05～1.10	1.03～1.05	1.0～1.03
Sliding	0.99	0.95～0.99	0.93～0.95	0.9～0.93

As is seen in this table, current factor of safety tends to decrease in an order of bedrock landslide, weathered rock landslide, colluvial deposit landslide and clay soil landslide. Current factor of safety in active landslide is 0.9 times higher than that in unmoving landslide.

As unit weight (γ) for soil blocks used in this survey, $\gamma=1.8\sim1.9$ (t/m^3) was employed, and γ of soil blocks closer to rocks was made higher. ($\gamma=2.0$ t/m^3 : NIC.5)
Shear strength can be determined by calculating $\tan\phi$ from the following expression, cohesion C and current factor of safety. Namely, when calculating reverse analysis-based shear strength, either of C or ϕ must be determined. Conversely, if either of them is calculated, other will be automatically determined. Which should be determined first is now uncertain.

$$C=(\Sigma W\cos\theta-\Sigma U\cdot l)/\Sigma l \times \tan\phi + F\cdot\Sigma\sin\theta/\Sigma l$$

The above equation represents a linear relation of C and $\tan\phi$. Based on the above calculation process, a method for improving current factor of safety by removal of soil and reduction of groundwater level was selected as measures against slope surfaces, using software "ARC." These results were summarized in Table 17.3.10(1)- (3).

However, a groundwater level is an estimated value based on Phase 2 survey's hearing and field investigation, and concerning to a groundwater level at heavy rain, maximum level was estimated based on boring results and collapse starting points. For a groundwater level at downpour, super hurricanes damaging drainage functions were assumed. Stability analysis for circular failure is conducted, generally using diakoptics assuming a circular sliding surface shown in Figure 17.3.4. According to this method, a soil block on the sliding surface is divided into some slices, and each slice's shearing force and resistance is determined. Then, summing up each shearing force and resistance, factor of safety is calculated from each ratio. As shown in the following expression, the number of slices shall be generally more than 6~7.

For more information, there is an expression assuming a complex sliding surface including direct lines instead of a circular sliding surface.

Calculate formula

$$Fs = \Sigma \mid C \cdot L + (W - u \cdot b) \cos \alpha \cdot \tan \Phi \mid / \Sigma W \cdot \sin \alpha$$

F	:	Factor of Safety
C	:	Cohesion
ϕ	:	Internal friction angle (°)
l	:	Sliding surface length cut by a slice (m)
W	:	Total weight of slices
U	:	Pore water pressure
b	:	Slice width
α	:	Angle between a line-linking a middle point and a center of a sliding surface and a plumb line

The methods for stability calculation include total stress and effective stress methods. Experience shows that the two methods are currently used appropriately according to drainage environments such as ground materials, structure and target grounds due to easy selection of a strength constant and a pore water pressure.

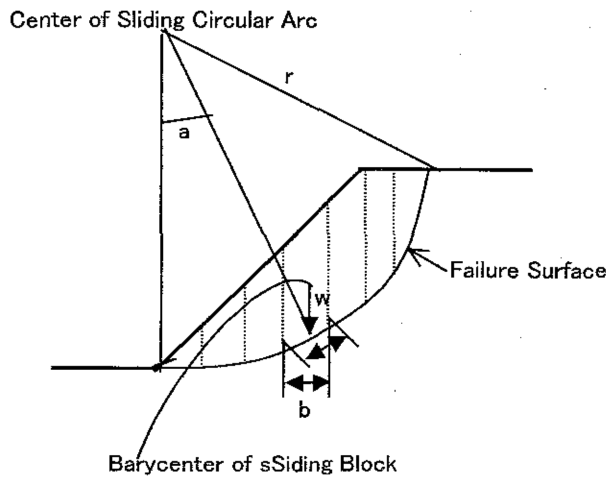


Figure 17.3.4 Stability Analysis by Sliding Circular Arc Method at Non-earthquake Condition

Table 17.3.10 (1) NIC.1 Stability Analysis Results

NIC.1 A280	Cross Section	Fs	Notes
Back Analysis of stability	NIC1 A280 Normal groundwater level (+ EL.424m)	1.05	Rainy season
	NIC1 A280 High groundwater level (+ EL.430m)	0.97	At heavy rain in rainy season

Table 17.3.10 (2) NIC.3 Stability Analysis Results

NIC.3 C230	Cross Section	Fs	Notes
Back Analysis of stability	NIC.3 C230 Back analysis Normal groundwater + EL.1011m	1.06	Rainy season
Cutting Slope	NIC.3 C230 Medium groundwater + EL.1022m	1.01	At heavy rain in rainy season
	NIC.3 C230 High groundwater + EL.1032m	0.79	At downpour in rainy season
Large slope failure including the road	NIC.3 C230 Normal groundwater + EL.1011m	1.44	Rainy season
	NIC.3 C230 Medium groundwater + EL.1022m	1.17	At heavy rain in rainy season
	NIC.3 C230 High groundwater + EL.1032m	1.02	At downpour in rainy season
Back Analysis of stability of cut and fill	NIC.3 C230 Shoulder back analysis Normal groundwater + EL.1011m	1.00	Rainy season
Top weight of shoulder	NIC.3 C230 Shoulder counter weight Normal groundwater + EL.1011m	1.44	Rainy season
	NIC.3 C230 Shoulder counter weight High groundwater + EL.1022m	1.00	At heavy rain in rainy season

NIC.3 C150	Cross Section	Fs	Notes
Back Analysis of stability	NIC.3 C150 Back analysis Normal groundwater + EL.1366m	1.02	Rainy season
Cutting Slope	NIC.3 C230 High groundwater + EL.1379m	0.94	At heavy rain in rainy season
Large scale slope slide including the road	NIC.3 C150 Normal groundwater + EL.1366m	1.14	Rainy season
	NIC.3 C150 High groundwater + EL.1379m	0.99	At heavy rain in rainy season
Back Analysis of stability of cut and fill	NIC.3 C150 Shoulder back analysis Normal groundwater + EL.1011m	1.02	Rainy season
Top weight of shoulder	NIC.3 C150 Shoulder counter weight Normal groundwater + EL.1011m	1.14	Rainy season
	NIC.3 C150 Shoulder counter weight High groundwater + EL.1022m	1.01	At heavy rain in rainy season

NIC.3 C140	Cross Section	Fs	Notes
Back Analysis of stability	NIC.3 C14 Back analysis High groundwater + EL.1411m	0.91	At heavy rain in rainy season
Large scale slope failure including the road	NIC.3 C14 Normal groundwater + EL.1404m	1.40	Rainy season
	NIC.3 C14 general failure High groundwater + EL.1379m	0.99	At heavy rain in rainy season
Back Analysis of cut and fill	NIC.3 C14 Shoulder back analysis High groundwater + EL.1404m	0.90	At heavy rain in rainy season
Top weight of shoulder	NIC.3 C14 counter weight Normal groundwater + EL.1404m	1.15	Rainy season
	NIC.3 C14 counter weight High groundwater + EL.1379m	0.99	At heavy rain in rainy season

Table 17.3.10 (3) NIC.5 Stability Analysis Results

NIC.5	Cross Section	Fs		Notes
Back Analysis (1)	Back Analysis	NIC.5 Back analysis High ground water (+ EL.558m)	1.00	At heavy rain in rainy season
Recutting (earth removal)	Cut to 40 deg	NIC.5 θ =40 deg Normal groundwater (+ EL.550m)	1.04	Rainy season
		NIC.5 θ =40 deg Normal groundwater (+ EL.558m)	1.00	At heavy rain in rainy season
	Cut to 35 deg	NIC.5 θ =35 deg Normal groundwater (+ EL.550m)	1.21	Rainy season
		NIC.5 θ =35 deg Normal groundwater (+ EL.558m)	1.12	At heavy rain in rainy season
Back Analysis (2)	Back Analysis	NIC.5 Back analysis High ground water (+ EL.558m)	1.01	At heavy rain in rainy season
Recutting (earth removal)	Cut to 40 deg	NIC.5 θ =40 deg Normal groundwater (+ EL.550m)	1.10	Rainy season
		NIC.5 θ =40 deg Normal groundwater (+558m)	1.02	At heavy rain in rainy season
	Cut to 35 deg	NIC.5 θ =35 deg Normally groundwater (+550m)	1.12	Rainy season
		NIC.5 θ =35 deg High groundwater (+558m)	1.10	At heavy rain in rainy season

Table 17.3.10 (4) NIC.26 Stability Analysis Results

NIC.26 B140	Cross Section	Fs	Notes
Back Analysis	NIC.26 N140 Back analysis High groundwater (+194m)	0.96	At heavy rain in rainy season
Stability of present conditions	NIC.26 N140 Back analysis Normal groundwater (+186m)	1.00	Rainy season
Stability after recutting	NIC.26 N140 Back analysis Normal groundwater (+186m)	1.27	Rainy season
	NIC.26 N140 Back analysis High groundwater (+194m)	1.02	At heavy rain in rainy season

17.3.8 Versatility of Slope Gradient in a Loosened Bedrock

The geological versatility level of slope gradient, is one of key points. Our survey scope is slant to the north in Nicaragua. Since bedrocks distributed here are bedrocks that were formed during the all Tertiary period as shown in the following table, the features of the bedrocks will be able to apply to those of bedrocks in west, east and north that were reportedly formed during almost the same era. Bedrocks from Oligocene to Eoceno in the former half of the Tertiary period, however, are significantly altered, while their loosened area tends to extend to deeper zones. For this reason, particular considerations should be paid to their stability. Table 17.3.11 shows a list of stratums to which these analysis results can be applied. In addition, these bedrocks spread two-dimensionally as shown in Figure 17.3.5. It was found that these results could be used very widely. But, it is uncertain whether the results can be applied to metamorphic rocks and abyssal rocks in the north.

Table 17.3.11 Slope Gradient Applicable Stratums

<table>
<tr><th colspan="2">Era</th><th colspan="2">West</th><th>Center</th><th>East</th><th>North East</th></tr>
<tr><td rowspan="2">Quaternary</td><td>Holocene</td><td colspan="2">Volcanic and Alluvium</td><td>Alluvium</td><td colspan="2" rowspan="2">Alluvial and Deposits Residuals</td></tr>
<tr><td>Pleistocene</td><td colspan="2">Group Las Sierras</td><td>Indistinct</td></tr>
<tr><td rowspan="5">Tertiary</td><td>Pliocene</td><td colspan="2">Fm.El Salto</td><td rowspan="2">Group Coyol</td><td rowspan="2">Fm.Bluefields</td><td rowspan="2">Group Coyol</td></tr>
<tr><td>Miocene</td><td>Fm. El Fraile</td><td>Fm. Tamarindo</td></tr>
<tr><td>Oligocene</td><td colspan="2">Fm.Masachopa</td><td rowspan="2">Group Matagalpa</td><td rowspan="2">Fm.Cukra</td><td rowspan="2">Group Matagalpa</td></tr>
<tr><td>Eocene</td><td colspan="2">Fm.Brito</td></tr>
<tr><td>Paleocene</td><td colspan="2" rowspan="2">Fm.Rivas</td><td colspan="3" rowspan="2">Group Pre−Matagalpa</td></tr>
<tr><td rowspan="2">Cretaceous</td><td>Superior</td></tr>
<tr><td>Inferior</td><td colspan="2">Completo Nicola en Costa Rica</td><td colspan="2"></td><td>Fm.Metapan</td></tr>
</table>

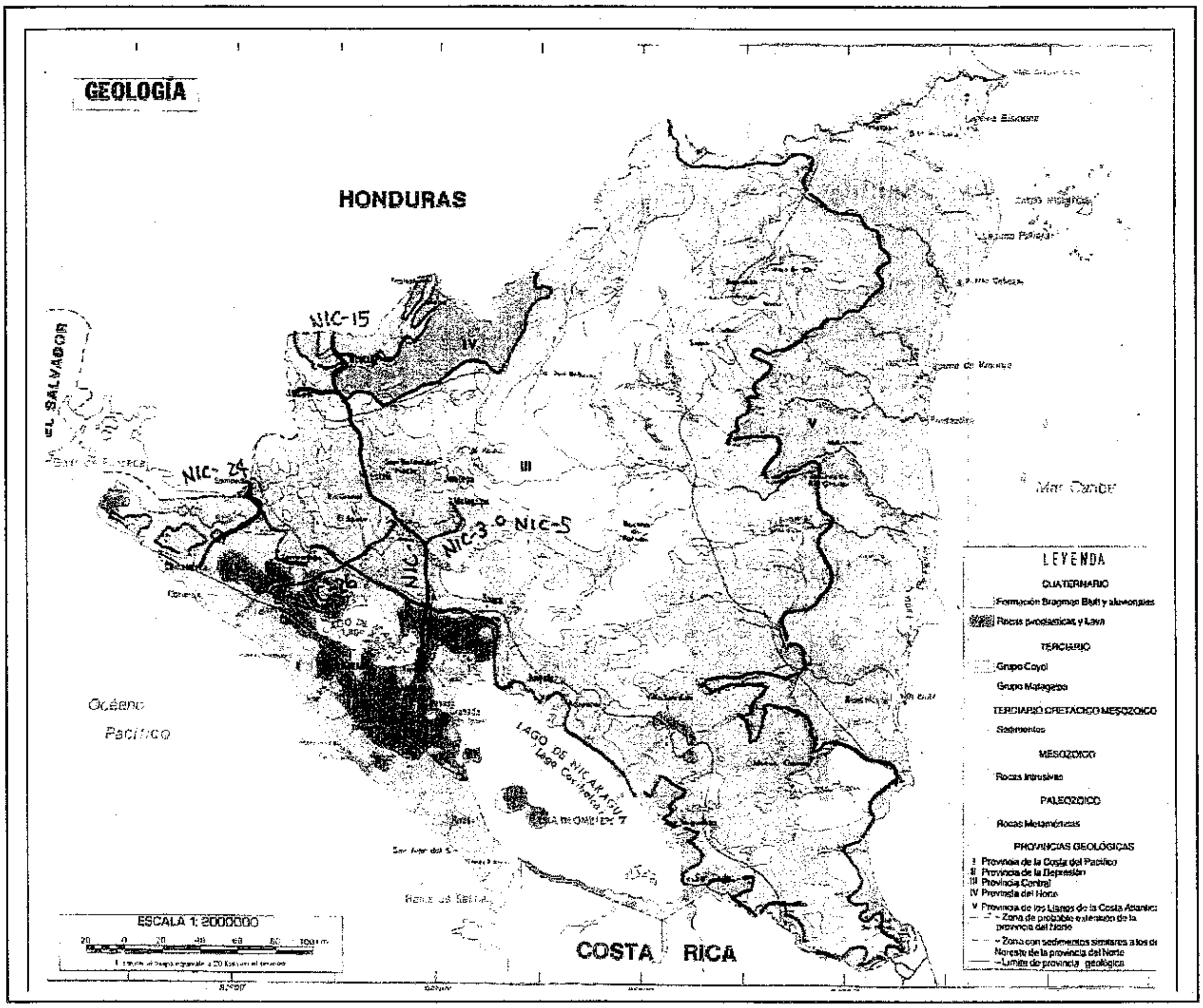


Figure 17.3.5 Geological Area