3. GEOLOGICAL INVESTIGATION

3. GEOLOGICAL INVESTIGATION

3.1 General

A geomorphological map is shown in Fig. 3.1-1 and a geological map of northern Bolivia is shown in Fig. 3.1-2.
Northern Bolivia is divided into six geological province stretching in belts from the north west to the south east.
These provinces are titled;

a) West Andes
b) High Plateau (Altiplano)
c) East Andes
d) Sub-Andes Zone
e) Amazonian Lowland
f) Brazilian Shield

The West Andes are basically made up of Palaeozoic sedimentary rock with the north east slope covered with Tertiary and Quaternary volcanic rock.
These volcanic rocks consist of andesite, dacite, tuff and ignimbrite.
The reaches of the West Andes to the north and south contain many active volcanoes. But in this particular no active volcanoes present.
The West Andes attained its elevation through mountain-building processes during the Palaeozoic and Mesozoic Era.

Following the orography, these mountains can be seen to have been subjected to long-continued subaerial erosion. After entering the Tertiary Period, rapid upheaval of this mountains took place due to volcanic activity. This activity is still continuing up to today.

The Puna Surface consists of Tertiary sedimentary rock. The sedimentary surface remains as eroded monad rock in several places at the present time. The lower erosional surface was filled by a lake, glacial fan deposits and volcanic products from the end of the Tertiary Period into the Quaternary. As a result, the wide, flat surfaces of the Altiplano was created.

The East Andes principally consist of Ordovician sand

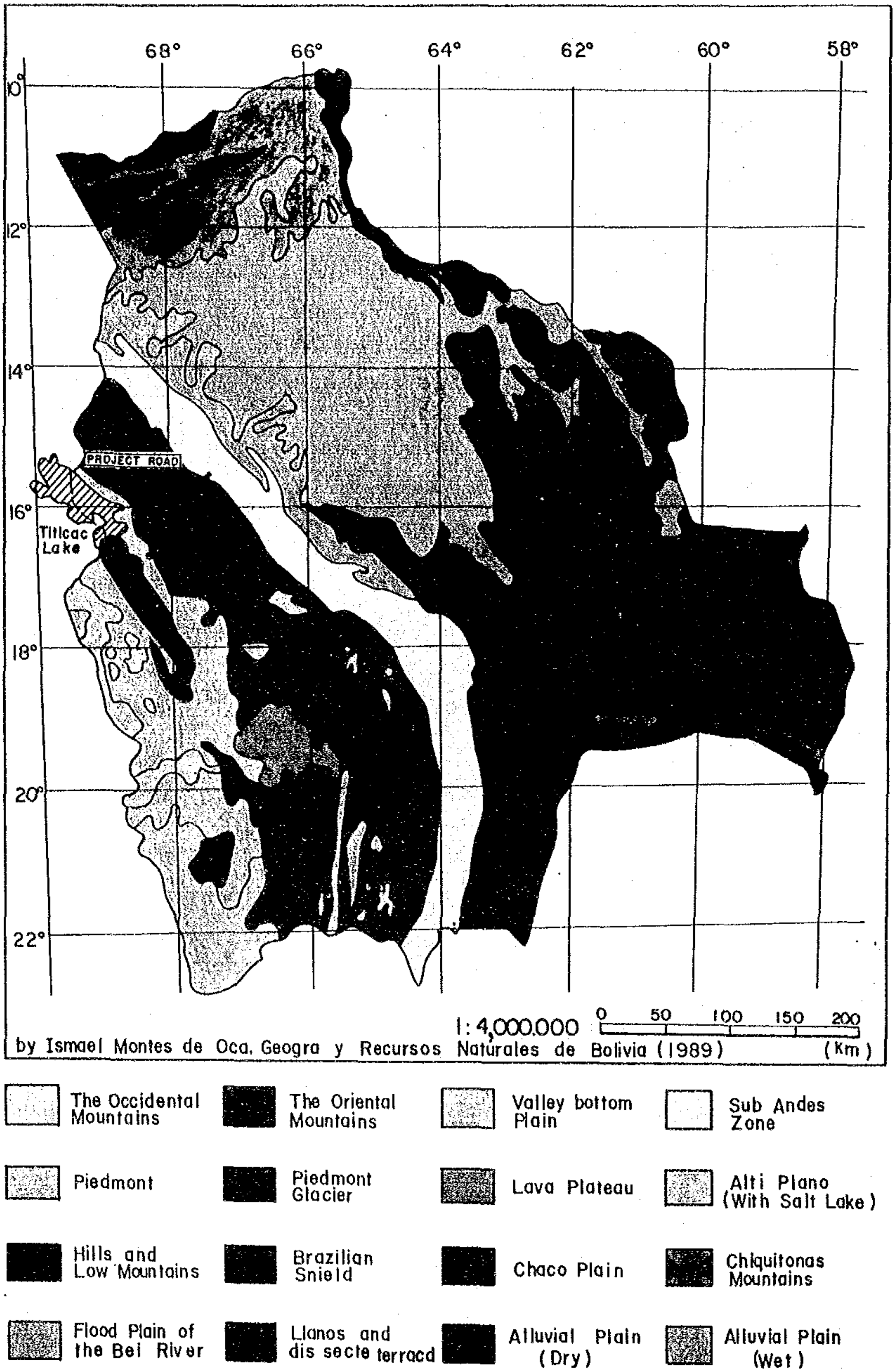


Fig 3. 1—1 Geomorphological Map of the Bolivia

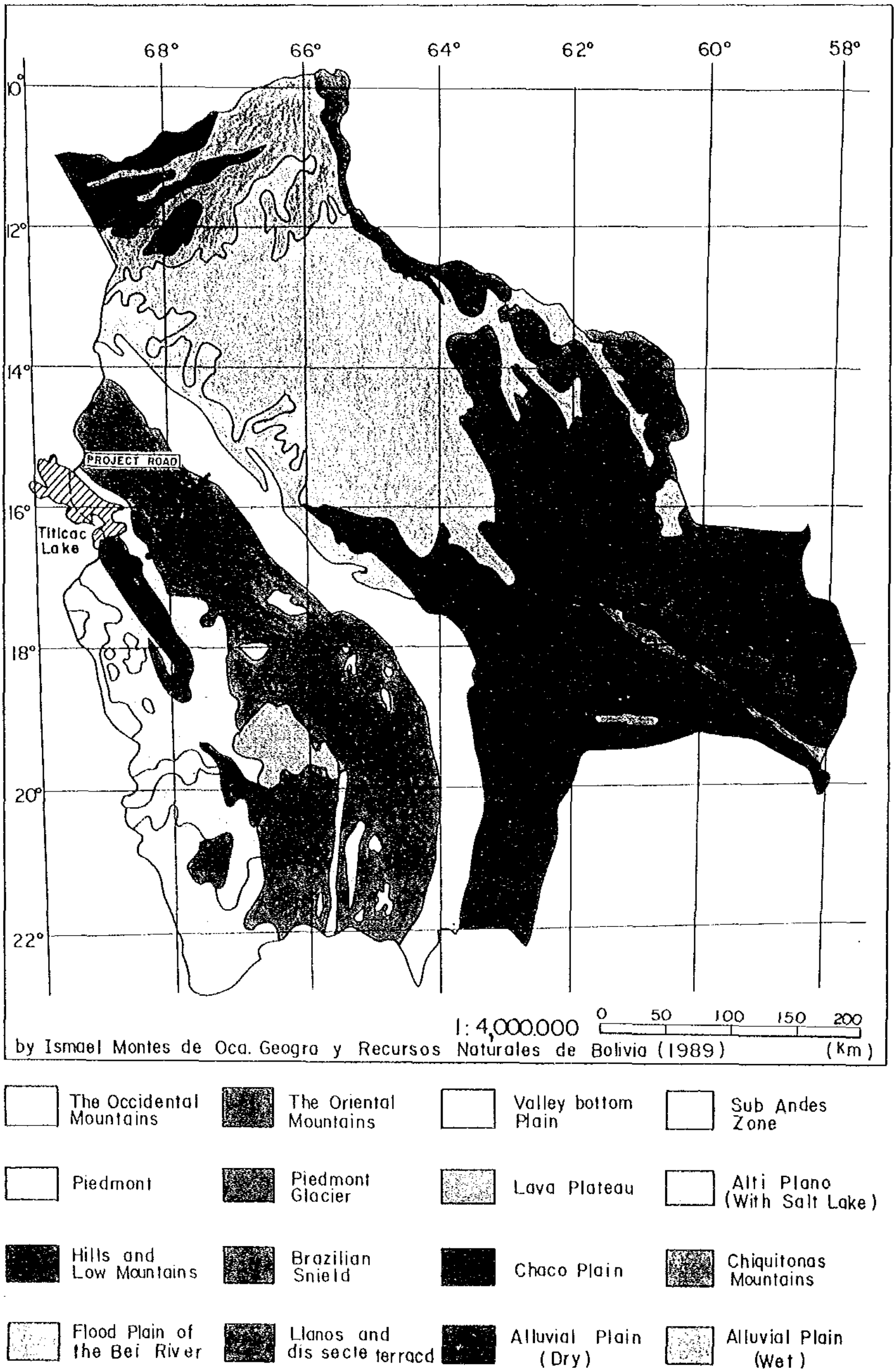


Fig 3. 1-1 Geomorphological Map of the Bolivia

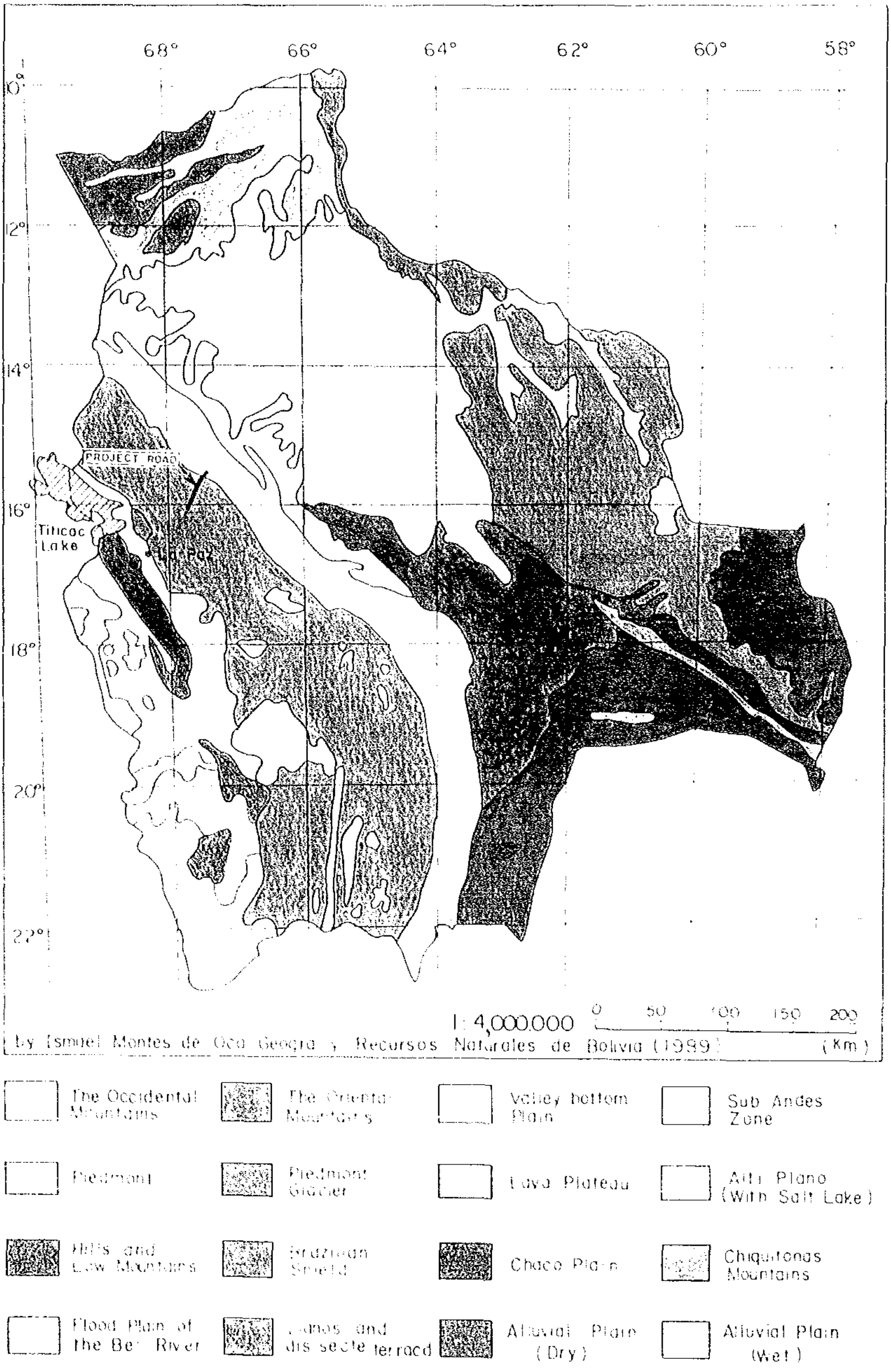
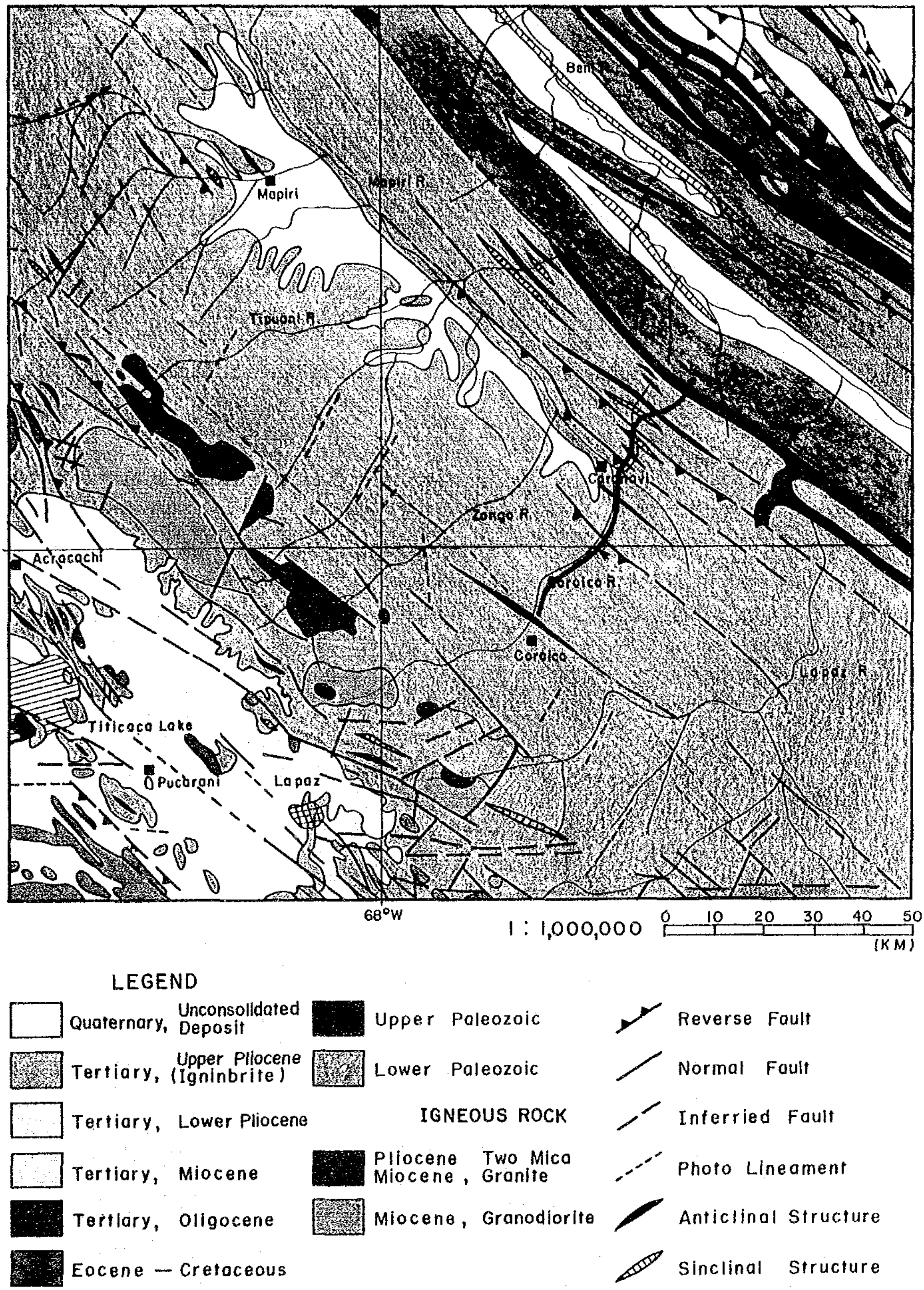
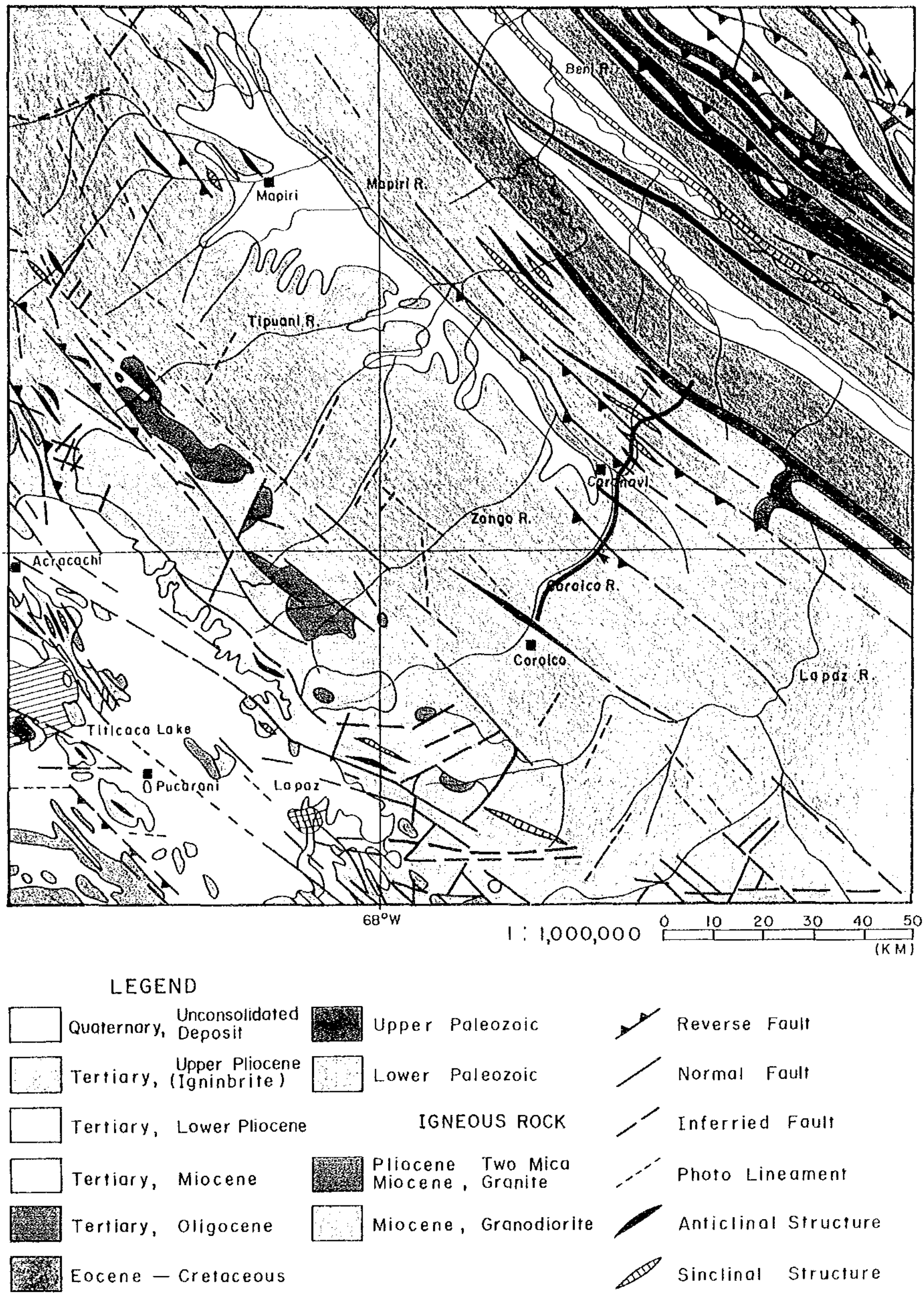


Fig 3 1-1 Geomorphological Map of the Bolivia



By C. Martinez, P. Tomasi, 「Carte Strucutrale Des Andes Septentrionales De Bolivie」 ORSTOM, Servico Geológico de Bolivia (Paris 1978)

Fig. 3. 1-2 Geological Map of the Northern-Bolivia



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ORSTOM, Servico Geológico de Bolivia (Paris 1978)

Fig. 3.1-2 Geological Map of the Northern-Bolivia

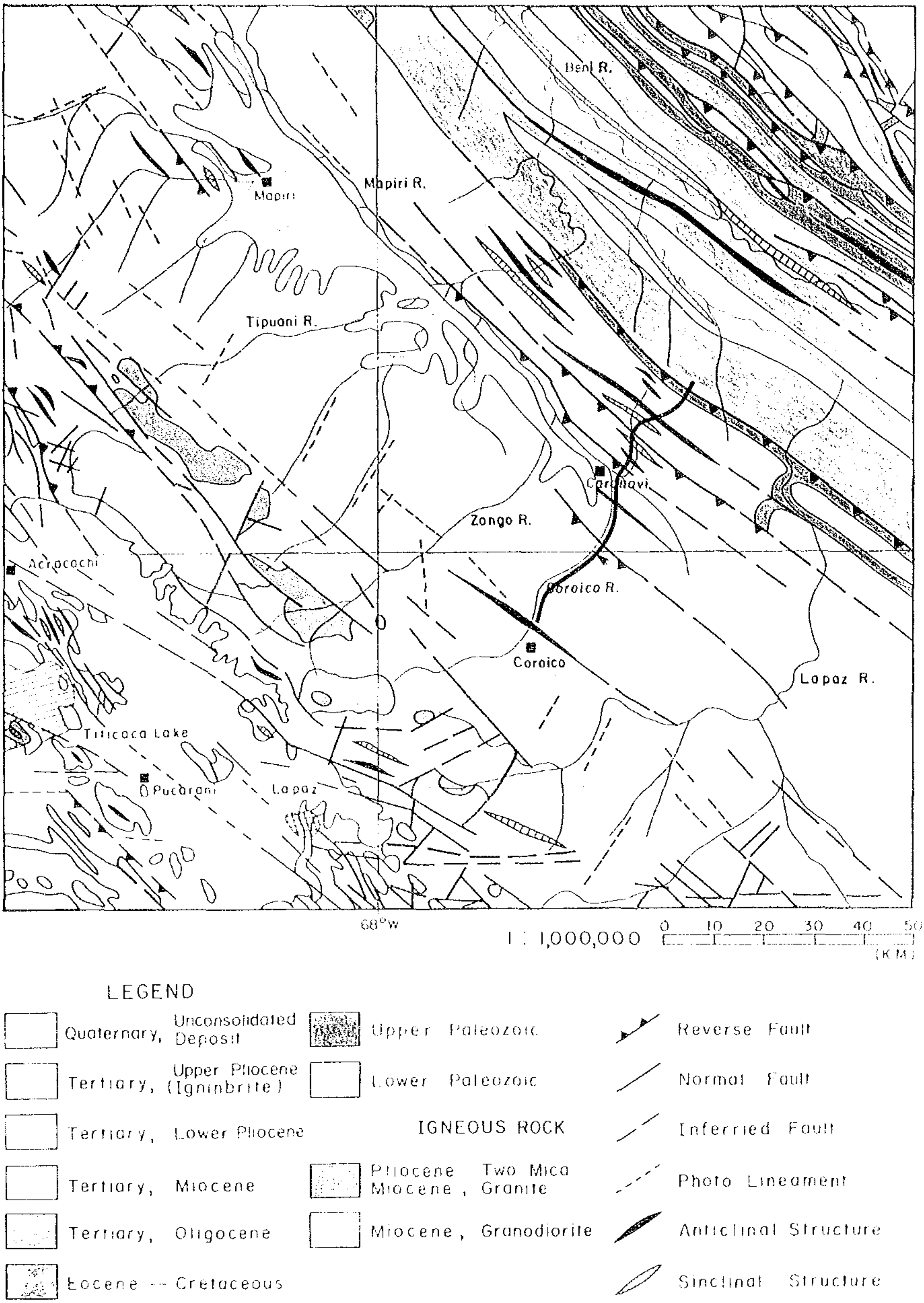

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Fig.3 1-2 Geological Map of the Northern Bolivia stone, slate, shale and mudstone. Among these, slate has the widest distribution. The Cretaceous sandstone and limestone lie narrowly along a fault Plain created in conjunct on Ordovician layer.

In the south west of the East Andes, Ordovician sedimentary rock contains granitic intrusions from the Miocene Period. The East Andes underwent greater tectonic movement in comparison with the West Andes.
As a result, the mountains have been divided into numerous fault blocks which have formed a complex topography.

The direction of the fault and fold axes is generally in accord with direction of the geologic provinces, However, some faults cross these borders.

The Sub-Andes Zone is a gathering of hills ranging in altitude from 400 m to 2,000 m above sea level. This zone consists of a Tertiary conglomerate, sandstone, mudstone and alternating layers of these.
These sedimentary layers make contact with cretaceous strata forming a fault line, accompanied with a basal conglomerate.
The sub-Andes zone was acted on by strong tectonic movement and is crisscrossed with faults in the same way as the East Andes. The rocks in this zone show lithofaces of molasse which was uplifted at the end of the Tertiary Period.

At the north eastern end of the Sub-Andes Zone, a wide lowland spreads out at an altitude of less than 400 m. This lowland is the flood plain of the Beni River and is part of the Amazon Basin.

At the north eastern section of the flood plain of the Beni River, the Brazilian Shield widely stretches out. This shield consists of the oldest crust on the continent of South America with the Guiana Shield at the northern end of the Amazon River.
The absolute age of these rocks are estimated to be from about 2,000 to 3,000 Ma. These rocks consist of sedimentary, Igneous and metamorphic strata from the Precambrian Era.

1)Andes Orogenic Movement

The Andes range is principally made up of Palaeozoic and Mesozoic sedimentary rocks and attained its elevation through mountain-building processes during the Cretaceous and Tertiary Period.
During the Cretaceous period, a geocyncline was formed at the east side of the West Andes which had already been subject to uplifting.
At a later period, these geocyncline deposits rose with faulting, folding and volcanic actions. At the end of Mesozoic Era, the whole of the Andes appeared above sea level. Then, granitic intrusions took place in the West Andes.

After entering the Tertiary stage, tectonic movement with volcanic action occurred along the eastern parts of the West Andes and granitic intrusions occurred along the East Andes. Tectonic movement with faulting and folding was limited in the East Andes region.
After that, the East and West Andes rose up until the present day.

During the Tertiary Period, the region between the West Andes and the East Andes was submerged beneath the sea, and Tertiary sedimentary rocks were deposited there. At the end of the Tertiary period, this region was elevated to form the Puna Surface.
Finally, the Sub-Andes Zone was elevated.

At the present time, the Andes Orogenic Movement is explained as a collision between the South American and the Pacific Plate undergoing tectonic movement.
The time when this orogenic movement began, is in accordance with the time that the Atlantic Ocean began to spread.

3.2 Description of Geological Formation

The geological investigation area is located in the East Andes Range and the Sub-Andes Zone.
In this area, sedimentary rocks of the palaeozoic, Mesozoic and Cenozoic Era are distributed. And Igneous rocks are not exposed here. A geological map and geological profile are shown in the "Geological Drawings" attached to this report.

3.2.1 Paleozoic Rocks

These rocks consist of sandstone, mudstone, shale, slate and alternating sandstone-mudstone conglomerate created during the Ordovician period. These rocks occupy 90 % of the area in the investigation zone.

Slate has the widest distribution of all, and is mainly distributed between Santa Bårbara and Santa Ana (Point "L"). This rock is colored black and is very hard but forms notable joints.
In every place, where rock falls and slope failures occur, this rock is present.

Shale is mainly distributed with sandstone and mudstone near Caranavi and Carrasco (Point "Q"). This rock is also black and hard and forms notable joints the same as slate, but is brittle.

Mudstone is mainly exposed in Santa Bårbara, Caranavi and Carrasco. This rock is dark gray and more easily broken than shale and slate, although it has few joints.
This rock often alternates with sandstone and also changes in quality to clay when subject to weathering. For that reason, this rock creates small slope failure like the landslide near Carrasco.

Sandstone is widely exposed and stratified with other rock types. A Small rock formation is distributed near Chojña (Point "J") and Caranavi.This fine or medium sandstone is gray and silicious, and is the hardest of all rocks in the area.

3.2.2 Mesozoic Rocks

These rocks consist of sandstone and limestone formed during the Cretaceous Period. They make contact with the Palaeozoic rocks and Tertiary Rocks along faults, and the area in which they are distributed is very limited.
This medium sandstone is a greenish gray and very hard, but the weathered form is breakable, having only a few joints.

The limestone is a grayish black color and is very hard, partially looking like fine sandy limestone.

3.2.3 Tertiary Rocks

Tertiary rocks are exposed in Bella Vista. The distribution of these rocks is in a hilly area of the Sub-Andes Zone.
These rocks consist of sandstone, mudstone, conglomerate and alternating strata of them from the Miocene age. Tertiary rocks contact with the Mesozoic rocks along the faults.

Conglomerate is a basal rock of the Tertiary Period and, distributed along faults. This rock is grayish brown, and well consolidated, and contains many rounded gravel aggregates whose diameters are from 5 to 10 mm. These gravels result from Mesozoic and Paleozoic sedimentary rocks .
The matrix of the conglomerate is composed of coarse and medium sand.

The sandstone is grayish and very hard. Blocks of this rock often cause rock falls because of the open joints which exist, every 2 to 3 meters. At a hill side slope, many blocks of sandstones are found weathered here and there.

The mudstone is reddish brown and severely weathered. It changes easily in quality to a clay when subjected to ground water. This is the weakest and softest among all the rocks encountered in the study area.
In the vicinity of Bella Vista, landslides sometimes occur due to the alternate strata of sandstone and mudstone.

3.2.4 Quaternary Rocks

Quaternary rocks consist of terrace deposits from the Pleistcene era and, talus, debris, landslide, and river deposits from the Holocene era.

Terrace and river deposits are distributed along the river channels. Others are encountered along mountain slopes. Terrace deposits are narrowly exposed along the Rio San Silverio, and widely exposed along the Rio Yara around Caranavi City.
Terrace deposits consist of rounded gravel whose diameters are from one to fifteen centimeters. The matrix is composed of brown medium or coarse sand.
The deposits are very stable, and the thickness of them is about 5 m at San Silverio and about 10 m at Caranavi.

Talus deposits are widely distributed over all of the investigation area. Thickness of the deposits varies from one to three meters on the mountain slope and partially from three to ten meters on the lower part of the valleys.
These deposits consist of angular gravels whose diameters are from five to ten centimeters. The matrix is composed of fine sand or silt.

Debris flow deposits are distributed near Santa Ana, Carrasco and Bella Vista. The debris flow deposits at Santa Ana are on a large scale, around one kilometer in width. The flat landscape at Santa Ana has been formed by the many debris flow in the past.
These deposits consist of angular gravel whose diameters are from 15 to 20 cm. The matrix is composed fine sand and silt. Therefore, it may be better to classify this as an earth flow instead of a debris flow.
At the present time, it seems that these deposits at Santa Ana have stabilized already, but those in the vicinity of Carrasco are still a little active and the existing road is sometimes damaged by earth flows.
Debris flow deposits in the Bella Vista area contain many sandstone boulder whose diameters are from one to two meters. These deposits are exposed along the bottom of the valleys, and it appears to be dangerous in that a section of these deposits seem ready to flow down again.

Landslide deposits are distributed around Bella Vista. These deposits result from the alternation of Tertiary rocks and consist of clay, fragments of weathered mudstone and boulders of sandstone.
At the present time, movement of these deposits exert a dangerous influence upon a security of the road at Bella Vista.

3.3 Geomorphological and Geological Problems for the Improvement of the Road

Along the investigation road, various slope failures were encountered. Those failures have been classified into several types for the geological investigation executed in the Study. The classification was identified in Fig. 2.3-1 in Section "2.3.2 Result of Site Reconnaissance". However, it has been sub-classified into more detail from a geological viewpoint as shown in Fig. 3.3-1.
The locations and characteristics of the failures investigated are shown in the tables from Table 3.3-1(1) to 3.3-1(9).
The locations and quantities of ground water run-off observed along the road are also shown in Table 3.3-2.

The types of slope failures classified in Fig. 2.3-1 in Chapter 2 are as follows;

a) Slope failure (cut slope or natural slope)
b) Embankment failure
c) Rock falls
d) Landslide
e) Debris flow or earth flow
f) Fractured zone along fault line

Under geological investigation, these six groups were furthermore classified into twelve categories.

The Slope failure (Type a) is divided into surface failure of soil and slope failure of rock. The Embankment failure (Type b) is divided into four types, each by the cause of failure, i.e.; lateral erosion by streams, displacement of embankment by rock material, displacement of filled soil and erosion by water from the road drainage. Rock falls are sub-divided into three categories; water erosion of soil material between boulders, control of open cracks and differential erosion speed among strata on slopes.

However, the actual failures at each site have two causes in reality, so that the exact classification of each failure can be very complicated. Hence, the description in Table 3.3-1 is based on the classification of six types only.

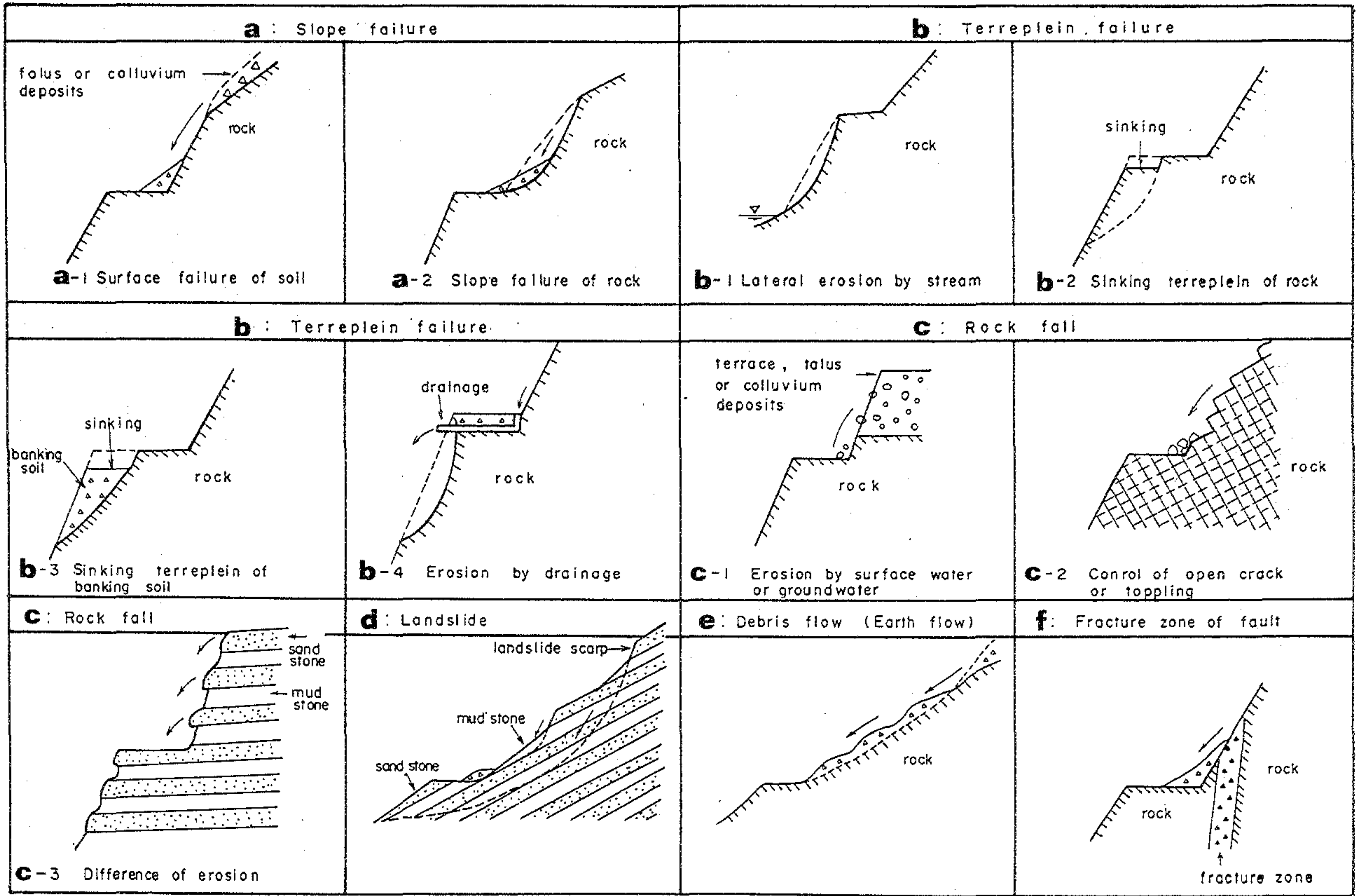


Fig 3.3-1 Type of Slope Failure

Table3. 3-1 (1) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure (m) Width	Hight	Length	Inclination of Slope (°) Sedi-mental	Arti-ficial	Natu-ral	Rock and Deposit Max φ=Maximum Diameter (cm) M φ=Mean Diameter (cm)	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
3	0.80	Surface failure	a	50	15	20	55	60	45	rock :slate, soil rubble:Max φ=40, M φ=5 matrix:clay	·N60W/65NE (2/m) N20E/70SE (2/m) N80E/50S (1/m)	—	II	
5	2.30	Surface failure	a	40	15	20	50	60	45	rock :mudstone, soil rubble:Max φ=40, M φ=5 matrix:clay	·N40W/40N (5/m) N35E/75SE (2/m) N60W/55N (5/m)	—	II	
6	2.50 A (2.70)	debris flow	e	160 (max)	80 (max)	160 (max)	45	-	30	rock :mudstone, soil deposit:nothing	·N55W/80NE (10/m) N50E/80E (2/m)	—	III	
8	3.10	fracture zone of fault (F-A)	f	20	10	-	-	60	40	rock :mudstone rubble:Max φ=20, M φ=5 matrix:nothing	impossible to measure	—	I	
14	5.00	fracture zone of fault (F-B) and surface failure	a·f	70	20	30	35	45	40	rock :mudstone rubble:Max φ=20, M φ=5 matrix:nothing	·N60W/45N (5/m) N20E/82NW (10/m) E-W/65S (3/m)	—	II	
20	6.50	old slope failure	a	20	20	28	40	50	35	rock :fine sandstone rubble:Max φ=200, M φ=10 matrix:clay	—	—	I	
21	6.54	disturbed zone of fault (F-C)	f	20 fault (3)	6	-	-	45	35	rock :fine sandstone deposit:nothing	impossible to measure	—	I	
25	B (7.30) 3.06	slope failure and gully at terreplein	a	20	25	30	45	60	45	rock :slate rubble:Max φ=50, M φ=10 matrix:clay	—	—	I	
27	8.46	slope failure	a	20	20	25	45	60	45	rock :slate with sandstone rubble:Max φ=90, M φ=10 matrix:clay	—	—	I	
30	9.20	active slope failure	a	40	10	-	70	80	60	rock :slate rubble:Max φ=100, M φ=10 matrix:clay	—	—	III	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance (Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II = Unstable when it Rains, III= Unstable

Table3. 3-1 (2) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure (m)			Inclination of Slope (°)			Rock and Deposit Max φ=Maximum Diameter (cm) M φ=Mean Diameter (cm)	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
				Width	Hight	Length	Sedi-mental	Arti-ficial	Natu-ral					
36	11.18	old surface failure	a	30	30	35	45	60	45	rock :slate rubble:Max φ=40, M φ=5 matrix:clay	—	—	I	
38	(C11.60) 12.00	two slope failure	a	40	20	20	45	70	40	rock :slate rubble:Max φ=200, M φ=10 matrix:clay	—	—	III	
41	challa 14.1	active rock fall and fractured zone of fault (F-D)	c·f	100	15	15	-	-	60	rock :slate rubble:Max φ=100, M φ=10 matrix:nothing	·N35W/60SW (1/m) N40E/60SE (2/m) N15W/15E (3/m)	—	III	
42	14.2	slope failure with rock fall	a·c	50	25	30	55	70	60	rock :slate rubble:Max φ=100, M φ=20 matrix:nothing	impossible to measure	—	III	
64	20.10	old slope failure	a	15	10	12	55	60	45	rock :slate rubble:Max φ=60, M φ=10 matrix:clay	—	—	I	
72	23.20	slope failure	a	90	10	-	-	-	-	rock :sandstone	—	—	II	
77	26.40 F (27.00)	fracfure zone of fault (F-F)	f	-	10	-	-	60	40	rock :slate (filite) deposit:nothing	·N60W/55SN (3/m) N75E/65S (1/m) N70W/40NE (10/m)	—	I	
82	27.90 choro	slope failure	a	15	15	-	55	-	-	rock :sandstone rubble:Max φ=300, M φ=10 matrix:nothing	—	—	I	
92	31.20 G (31.90)	slope failure and fracture zone of fault (F-G)	a·f	70	15	25	40	55	35	rock :slate rubble:Max φ=100, M φ=20 matrix:fine sand	·N50W/90 (fault)	—	III	
93	31.30	slope failure	a	80	-	-	-	-	-	rock :slate	·N55W/55S (20/m) N30E/90 (3/m) NS/65SW (3/m)	—	II	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance (Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II= Unstable when it Rains, III= Unstable

Table3. 3-1 (3) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure (m)			Inclination of Slope (°)			Rock and Deposit Max φ=Maximum Diameter (cm) M φ=Mean Diameter (cm)	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
				Width	Hight	Length	Sedi-mental	Arti-ficial	Natu-ral					
105	H(34.90) 35.90	slope failure with rock fall	a·c	40	20	25	45	60	-	rock :slate rubble:Max φ=40, M φ=20 matrix:clay	·N53W/30SW	75.0	II	
108	37.20	rock fall	c	15	-	15	-	70	-	rock :slate rubble:Max φ=150, M φ=10 matrix:nothing	—	—	II	
118	I(37.80) 39.10	Surface failure	a	15	15	15	50	90	-	rock :slate, Soil rubble:Max φ=150, M φ=20 matrix:clay	·N4E/49SE(5/m) N45E/49NE(2/m) N77W/84NE(3/m)	—	I	
120	40.80	Surface failure	a	40	-	25	45	60	30	rock :slate, Soil rubble:Max φ=60, M φ=10 matrix:clay	—	—	II	
122	41.10 J(41.70)	Surface failure	a	70	-	15	30	55	40	rock :slate rubble:Max φ=50, M φ=10 matrix:nothing	·E-W/38N(6/m) N22E/90(3/m) N78W/67SW(2/m)	—	III	
125	42.00	disturbed zone of small folding	f	-	-	-	-	82	-	rock :slate rubble:nothing matrix:nothing	—	—	I	
126	42.30	slope failure with earth flow, terreplein failure	a·b e	60	8	-	-	90	60	rock :slate rubble:Max, M φ=10 matrix:clay	impossible to measure	—	III	
133	44.60	Surface failure	a	50	-	20	45	60	-	rock :slate, Soil rubble:Max, M φ=10 matrix:clay	—	—	I	
135	44.80	fracture zone of fault (F-H) and crack of terreplein	b·f	30	15	-	-	55	40	rock :slate deposit:nothing	impossible to measure	—	II	
137	46.60	slope failure and fracture zone	a·f	50	15	18	-	55	60	rock :slate deposit:nothing	·N60W/45S(5/m) N70W/90(5/m) N25E/ 9C(5/m)	—	II	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance(Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II = Unstable when it Rains, III= Unstable

Table3. 3-1 (4) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure(m)			Inclination of Slope (°)			Rock and Deposit	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
				Width	Hight	Length	Sedi-mental	Arti-ficial	Natu-ral	Max φ=Maximum Diameter(cm) M φ=Mean Diameter(cm)				
138	47.30	fracture zone of fault(F-I)	f	20	10	-	-	50	60	rock :slate deposit:nothing	·N60W/80S(10/m)	—	I	
140	47.60	slope failure and fracture zone of fault(F-J)	a·f	120 fault (1m)	10	-	-	55	60	rock :slate deposit:nothing	·N70W/90(20/m)	—	II	
141	47.90 K(49.10)	fracture zone	f	20	10	-	-	55	60	deposit:nothing	N60W/35N(20/m) N40E/65SE(10m)	—	I	
145	49.90	slope failure	a	50	15	20	-	55	40	rock :slate deposit:nothing	·N35W/55E(20/m) N25W/35SW(10/m) N55E/80NW(2m)	—	II	
148 | 150	51.30 | 51.40	debris flow	e	500	300	1200	-	55	15	rock :slate rubble:Max φ=70, M φ=20 matrix:clay	—	7.5	II	
152	52.20	fracfure zone of fault(F-K)	f	10	10	15	-	60	40	rock :slate lack of data	impossible to measure	—	I	
154	L(52.40) 52.40	slope failure with debris flow	a·e	80	10	100	-	45	50	rock :slate rubble:Max φ=250, M φ=10 matrix:fine sand	—	—	II	
156	53.40	slope failure with debris flow	a·e	300	-	100	-	60	15	rock :slate rubble:Max φ=80, M φ=20 matrix:clay	·N-S/20E (10/m) N40W/70W(3/m) N55E/90(2/m)	—	I	
157	54.50	slope failure and debris flow	a·e	12	9	100	45	60	45	rock :slate lack of data	·N-S/20W(20/m) E-W/90 (20/m) N-S/90 (5/m)	—	II	
158	55.40	slope failure	a	10	8	10	45	60	20	rock :slate lack of data	—	—	II	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance(Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II= Unstable when it Rains, III= Unstable

Table3. 3-1 (5) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure(m)			Inclination of Slope (°)			Rock and Deposit	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
				Width	Hight	Length	Sedi-mental	Arti-ficial	Natu-ral	Max ϕ=Maximum Diameter(cm) M ϕ=Mean Diameter(cm)				
160	55.80	slope failure and fractue zone of fault(F-L)	a·f	8	5	7	45	60	20	rock :slate deposit:nothing	·N40W/20E (20/m) N45W/90(20/m) N55E/85N(3/m)	—	I	
161	57.60	debris flow	e	500	-	200	-	70	20	rock :slate lack of data	—	—	I	
165	58.50 M(59.70)	Slope failure	a	50	12	-	35	50	45	rock :sandstone rubble:Max ϕ=100,M ϕ=10 matrix:fine sand	—	—	II	
172	60.90	Surfase failure with terreplein failure	a·b	50	-	25	30	50	40	rock :mudstone,soil rubble:Max ϕ=100,M ϕ=10 matrix:clay	—	—	II	
174	61.30	Surface failure with crack at slope	a	40	10	40	20	-	20	rock :mudstone,soil rubble:Max ϕ=10, M ϕ=5 matrix:clay	—	—	III	
176	61.70 Caranavi (63.90)	Surface failure and rock fall	a·c	50	6	20	-	30	20	rock :mudstone,soil rubble:lack of data matrix:clay	—	—	I	
179	66.30	Surface failure	a	-	-	-	-	-	-	rock :mudstone,soil rubble:lack of data matrix:clay	—	2.4	I	
191	N(69.20) 70.60	slope failure	a	40	15	20	40	60	30	rock :fine sandstone rubble:Max ϕ=100,M ϕ=15 matrix:fine sand	·N45E/30SE(5/m) N50W/80SW(2/m) N70E/70N(2/m)	—	II	
194	71.20	slope failure with gully	a	40	15	20	40	60	30	rock :shale rubble:Max,M ϕ=1∼2 matrix:silt	—	—	II	
198	72.20	big slope failure	a	150	20	30	45	60	25	rock :shale lack of data	·N60W/40N(2/m) N15W/60SW(5/m) N45E/60S(10/m)	—	III	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance(Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II= Unstable when it Rains, III= Unstable

Table3. 3-1 (6) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure (m)			Inclination of Slope (°)			Rock and Deposit Max ϕ=Maximum Diameter (cm) M ϕ=Mean Diameter (cm)	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
				Width	Hight	Length	Sedi-mental	Arti-ficial	Natu-ral					
199 | 200	72.60 72.90	debris flow	e	300	8	400	-	70	25	rock :slate rubble:Max ϕ=5, M ϕ=2 matrix:silt	—	7.3	II	
202	73.40	slope failure	a	50	5	-	-	-	-	rock :shale	·N65W/40NE (5/m) N50E/40 (10/m) N80W/60S (5/m)	—	II	
203	73.90	slope failure	a	75	-	-	-	65	-	rock :sandstone, mudstone	·N60E/50NE (5/m) N30E/60E (5/m) N60W/10SW (5/m)	—	II	
207 | 208	77.20 78.00	debris flow by photo interpretation	e	300	-	300	-	-	-	rock :mudstone lack of data	—	—	II	
210 | 211	78.30 78.70	debris flow and slope failure	a·e	200 (15)	- (6)	200 (6)	-	- (90)	-	rock :mudstone rubble:Max ϕ=60, M ϕ=30 matrix:silt	·N54W/23E (3/m) N58W/52SW (7/m) N47E/90 (1m)	—	I	
211 | 212	78.50 79.30	debris flow by photo interpretation	e	150	-	300	-	-	-	rock :mudstone lack of data	—	—	II	
213 | 214	79.90 80.20	debris flow by photo interpretation	a·e	30	-	-	-	-	-	rock :mudstone lack of data	—	—	II	
215	P (80.10) 80.30	surface failure and terreplein failure	a·b	40	-	40	-	60	50	rock :mudstone, soil lack of data	·N25W/13SW (7/m) N14W/45NE (7/m) N38W/85SW (5m)	Surface water	I	
217	80.70	rock fall	c	4	6	6	-	90	-	rock :mudstone rubble:Max ϕ, M ϕ=30 matrix:nothing	·N59W/17SW (7/m) N23E/81SE (2/m) N52W/81SW (1m)	—	I	
219	81.00	surface failure	a	110	-	40	40	63	-	rock :mudstone, sandstone rubble:Max ϕ=15, M ϕ=3 matrix:clay	·N49W/17SW (5/m) N23E/81SE (2/m) N77W/72NE (4m)	—	III	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance (Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II= Unstable when it Rains, III= Unstable

Table3. 3-1 (7) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure (m)			Inclination of Slope (°)			Rock and Deposit Max ϕ=Maximum Diameter (cm) M ϕ=Mean Diameter (cm)	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
				Width	Hight	Length	Sedi-mental	Arti-ficial	Natu-ral					
223	81.80	surface failure	a	100	5	5	-	-	60	rock :sandstone, soil lack of data	—	—	II	
226	83.70	surface failure	a	150	6	-	-	-	15	rock :shale, soil rubble:Max ϕ, M ϕ=5～10 matrix:clay	—	0.2	III	
227	83.80	surface failure	a	30	5	-	-	50	20	rock :shale, soil rubble:lack of data matrix:clay	—	—	II	
228	84.10	surface failure	a	250	6	80	-	40	20	rock :shale, soil rubble:lack of data matrix:clay	—	—	III	
230	85.00	Surface failure with crack at slope	a	50	-	40	-	45	35	rock :shale, sandstone, soil rubble:lack of data matrix:clay	—	—	II	
231	Q(85.10) carrasco	debris flow	e	20	15	-	45	60	40	rock :shale, sandstone rubble:Max ϕ=20 ～30 matrix:clay	·N40W/35E(10/m) N55W/50S(5/m) N55E/55N(3/m)	serface water	II	
234	85.70 R(87.70)	slope failure	a	150	10	15	45	60	40	rock :shale rubble:lack of data matrix:nothing	—	—	III	
243	88.00	two slope failure	a	40	20	25	45	60	40	rock :shale lack of data	·N20W/60E(20/m) E-W/60N(20/m) N80E/20N(10/m)	—	II	
245	88.50	surface failure and fracture zone of fault (F-M)	a·f	10	15	25	45	60	40	rock :shale lack of data	—	—	I	
248	89.20	Terreplen failure and fracture zone of fault (F-N)	b·f	10	10	-	45	60	45	rock :shale lack of data	·N15E/20E(10/m) N25E/75W(10/m) N30W/80NE(10/m)	serface water	III	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance (Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II= Unstable when it Rains, III= Unstable

Table3. 3−1 (8) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure (m)			Inclination of Slope (°)			Rock and Deposit Max φ=Maximum Diameter (cm) M φ=Mean Diameter (cm)	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
				Width	Hight	Length	Sedi-mental	Arti-ficial	Natu-ral					
249	89.50	slope failure	a	150	5	-	-	-	-	rock :slate	·N60W/65N (20/m) N30E/65W (3/m) N55W/35SW (3/m)	—	II	
254	90.50	surface failure	a	-	-	-	45	80	50	rock :shale, soil rubble:Max φ=60, M φ=3 matrix:clay	—	—	II	
258	90.65	terreplein failure	b	10	-	-	-	-	60	rock :shale, soil lack of data	·N32W/49NE	—	I	
259	91.00	surface failure	a	-	-	-	-	-	-	rock :shale, soil lack of data	—	seepage of water	I	
260	91.05	surface failure with earth flow	a	20	-	-	-	-	-	rock :shale, soil lack of data	—	—	I	
261	91.30	surface failure with earth flow	a	30	-	-	37	62	32	rock :shale, soil lack of data	—	—	I	
263	91.45	surface failure and terreplein failure	a·b	5	-	40	-	64	32	rock :shale, soil lack of data	—	—	I	
264	91.60	surface failure	a	20	-	30	-	-	-	rock :shale, soil lack of data	—	seepage water	I	
278	S(93.40) 93.70	rock fall	c	40	-	20	-	-	-	rock :sandstone rubble:Max φ=100, M φ=20 matrix:fine sand	·N77W/13NE N77W/77SW E-W/67N	—	I	
280	95.80	surfase failure	a	76	-	30	-	-	-	rock :mudstone, soil rubble:Max φ, M φ=5 matrix:clay	—	—	I	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance (Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II= Unstable when it Rains, III= Unstable

Table3. 1-1 (9) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure(m)			Inclination of Slope (°)			Rock and Deposit Max φ=Maximum Diameter(cm) M φ=Mean Diameter(cm)	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
				Width	Hight	Length	Sedi-mental	Arti-ficial	Natu-ral					
284	96.30 T(96.40)	terreplein failure	b	20	-	-	-	-	70	rock :mudstone lack of data	—	0.2	II	
288	97.6	slope failure	a	50	-	-	-	-	-	rock :mudstone	—	0.2	II	
289	97.80	surface failure	a	40	5	-	-	60	35	rock :mudstone, soil rubble:Max φ, M φ=10 matrix:clay	—	—	II	
294	98.90	rock fall	c	50	-	15	-	80	60	rock :mudsotne rubble:Max φ=80, M φ=15 matrix:clay	·N27W/75NE(7/m) N54E/85NW(5/m) N56E/76NW(1/m)	—	II	
296	99.10	rock fall	c	60	-	12	35	80	85	rock :mudstone rubble:Max φ, M φ=5 matrix:clay	·N6W/31NE(8/m) N13E/90(3/m)	—	I	
301	100.30	rock fall	c	44	-	12	40	90	45	rock :mudstone rubble:Max φ, M φ=15 matrix:clay	—	—	I	
304	101.7	slope failure	a	21	-	25	45	70	55	rock :mudstone	·N5W/90(8/m)	90	II	
310	102.60	surface failure	a	30	-	20	-	-	70	rock :mudstone, soil rubble:Max φ=30, M φ=10 matrix:clay	·N72W/39SW N16E/36NW N50W/63NE	60.0	II	
317	U(104.30) 104.35	surface failure	a	30	-	90	-	75	45	rock :mudstone, soil rubble:Max φ, M φ=5 matrix:clay	·N54W/40NE N49E/67NW N22E/22SE	6.0	I	
320	104.65	fracture zone of fault(F-0)	f	10	-	-	-	80	-	rock :sandstone and breccia	—	—	I	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance(Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II= Unstable when it Rains, III= Unstable

Table3. 1－1 (10) Dangerous Zone to Construction of Highway

NO	Distance (Km)	Type of Slope Failure		Scale of Slope Failure (m)			Inclination of Slope (°)			Rock and Deposit Max ϕ=Maximum Diameter (cm) M ϕ=Mean Diameter (cm)	Dip and Strike of Bed and Joint Stratum (Number/m)	Ground-water (ℓ/min)	Grade of Stability	Remarks
				Width	Hight	Length	Sedi-mental	Arti-ficial	Natu-ral					
330	107.10 V (107.70)	rock fall and fructure zone of fault (F-Q)	c·f	49	-	15	30	70	30	rock :mudstone rubble:Max ϕ=200, M ϕ=50 matrix:clay	·N68W/30SW N94E/90 N50W/75NE	—	I	
332	107.90	slope failure and fracture zone of fault (F-R)	a·f	60	-	-	-	40	-	rock :sandstone and breccia	—	—	II	
339	110.50	debris flow	e	200	-	-	-	-	20	rock :sandstone, gravel rubble:Max ϕ, M ϕ=100～200 matrix:nothing	—	—	II	
340	W (111.30) 111.30	landslide	d	100	5	200	20	45	10	rock :mudstone , sandstone rubble:Max ϕ=200, M ϕ=100 matrix:clay	—	—	III	
341	111.60	landslide	d	30	5	200	20	45	10	rock :mudstone , sandstone rubble:Max ϕ=200, M ϕ=100 matrix:clay	·N65W/15E (3/m) N65E/90 (3/m) N-S/90 (3/m)	—	III	
344	112.50	landslide	d	30	7	400	20	45	10	rock :mudstone , sandstone rubble:Max ϕ=200, M ϕ=100 matrix:clay	—	serface water	III	
349	114.30 Bella vista	landslide	d	30	2	300	15	-	15	rock :mudstone , sandstone rubble:Max ϕ=200, M ϕ=100 matrix:clay	·N60W/25N (10/m) N80E/80S (10/m) N25W/80W (2/m)	7.5	III	

NOTE: ·Number of above "No. " is indicated in the Geoligical Plan.
·Distance (Km) is the accumlated distance from Santa Barbara.
·Type of Slope Failure: a =Slope Failure, b= Terreplein Failure, c=Rock Fall, d=Landslide, e= Debris Flow, f= Fracture Zone of Fault
·Grade of Stability : I =Stable , II= Unstable when it Rains, III= Unstable

Table 3.3-2 (1) POINTS OF GROUND WATER RUNOFF

NO.	LOCATION AND DISTANCE (km)	ROCK AND DEPOSIT	AMOUNT OF RUNOFF (l/min)
10	point A (2.70) 4.20	talus deposit of mudstone	1.2
11	Padilla (4.40) 4.50	talus deposit of mudstone	49.5
12	4.60	talus deposit of mudstone	90.0
13	4.76	talus deposit of mudstone	23.0
16	5.10	talus deposit of slate	0.5
29	point B (7.30) 8.72	between slate and its talus	60.0
31	9.38	talus deposit of slate	3.0
34	10.18	sandstone	3.0
35	10.38	sandstone	15.0
37	point C (11.60) 11.38	talus deposit of slate	13.8
44	Challa (12.08) 12.98	between slate and its talus	1.8
46	Villa Espada 15.40	between slate and its talus	13.8
48	point D (15.50) 15.70	talus deposit of slate	2.0
49	16.40	talus deposit of slate	0.5
50	10.45	talus deposit of slate	1.0
52	17.15	talus deposit of slate	7.2
53	17.35	talus deposit of slate	6.6
54	17.40	talus deposit of slate	14.4
55	17.50	talus deposit of slate	5.4
57	18.30	slate	40.0
58	18.34	talus deposit of slate	33.0
59	18.60	slate	30.0
60	19.70	slate	17.0
62	19.80	slate	60.0
65	20.20	slate	60.0
66	20.36	slate	15.0
67	20.72	slate	20.0
69	point E 22.10	slate	1.0
72	22.70	sandstone	0.3
79	Alto choro (27.10) 27.40	sandstone	2.0
80	27.50	between sandstone and its talus	28.0
81	27.80	between sandstone and its talus	22.5
84	Choro (28.35) 28.40	between sandstone and its talus	9.0
87	29.50	between slate and its talus	0.6
96	San Pedro (31.90) 32.30	between slate and its talus	15.0
98	32.90	between slate and its talus	0.3
101	Pto. Leon (34.90) 133.8	between slate and its talus	6.0
105	35.50	talus deposit of slate	75.0
111	point I (37.80) 37.70	slate	1.0
129	18 de mayo (43.50) 43.10	talus deposit of slate	0.8
142	San silverio (49.20) 49.20	talus deposit of slate	1.0
150	51.20	talus deposit of slate	7.5
162	point L (52.40) 57.80	talus deposit of slate	2.4
163	point M (59.70) 58.10	talus deposit of slate	0.3
177	Caranavi (63.90) 64.90	talus deposit of sandstone	1.5
178	66.00	talus deposit of sandstone	0.7
179	66.30	talus deposit of sandstone	2.4
180	66.40	talus deposit of sandstone	3.0
182	67.20	talus deposit of sandstone	3.0
184	67.60	talus deposit of sandstone	0.2

the term of investion : from 13/9/89 to 28/9/89

Table 3.3-2 (2) POINTS OF GROUND WATER RUNOFF

NO.	LOCATION AND DISTANCE (km)	ROCK AND DEPOSIT	AMOUNT OF RUNOFF (l/min)
200	point N (69.20) 72.90	debris flow deposit of shale	7.3
205	point O 76.80	shale	0.2
224	82.60	talus deposit of sandstone	0.2
226	83.60	talus deposit of shale	0.2
253	Carrasco (85.10) 90.20	talus deposit of shale	0.2
255	90.35	shale	0.2
256	90.50	talus deposit of shale	1.7
259	91.00	shale	seepage
264	91.60	shale	seepage
266	91.70	talus deposit of shale	0.3
267	91.90	talus deposit of shale	0.4
270	92.50	sandstone	seepage
271	92.65	talus deposit of sandstone	1.0
272	92.75	mudstone	1.9
273	92.90	shale	seepage
274	93.05	talus deposit of sandstone	1.0
275	point S (93.40) 93.30	talus deposit of sandstone	6.0
277	93.65	talus deposit of sandstone	6.0
279	94.50	talus deposit of sandstone	6.0
281	95.75	talus deposit of mudstone	7.5
282	95.80	between mudstone and its talus	7.2
283	95.90	shale	0.2
284	point T (96.40) 96.30	talus deposit of mudstone	0.2
286	96.50	mudstone	0.2
288	96.90	talus deposit of mudstone	0.2
292	98.70	talus deposit of mudstone	0.2
295	99.05	talus deposit of mudstone	7.2
299	99.70	talus deposit of mudstone	9.4
300	100.10	talus deposit of mudstone	0.6
302	100.60	talus deposit of mudstone	5.4
303	100.80	talus deposit of mudstone	4.5
304	101.40	slope failure of sandstone	90.0
306	102.00	between sandstone and its talus	3.0
308	102.40	mudstone	1.0
310	102.60	talus deposit of shale	60.0
312	102.90	talus deposit of mudstone	1.0
315	104.20	mudstone	0.8
316	point U (104.30) 104.25	talus deposit of mudstone	3.0
317	104.35	between mudstone and its talus	6.0
318	104.50	talus deposit of mudstone	2.0
321	104.70	between mudstone and its talus	45.0
338	point V (107.70) 109.90	between sandstone and its talus	0.6
347	Bella Vista (114.00) 114.00	deposit of landslide	7.5

the term of investion : from 13/9/89 to 28/9/89

The stability of slopes, as a result of observation, is also shown in Table 3.3-1 using the following classification:

Grade I stable
Grade II unstable when it rains
Grade III unstable

The places which are classified as Grade I are considered to have been already stabilized. A serious disaster will hardly be expected in those places.

The grouping of Grade II is a slope that has a fresh surface of failure and shows traces of slope sliding. This group has the possibility of undergoing slope failure when subject to ordinary rain.

Grade III is active slope failure. In this group, movement of materials is always found.

The number of slopes investigated by failure type and stability grade are shown in Table 3.3-3.

Table 3.3-3 Number of Failures by Type and Stability Grade

Type	a	b	c	d	e	f	Total	
Grade								
I	22(20)	3(1)	6(5)	0(0)	3(3)	11(8)	45(37)	
II	31(28)	3(2)	3(2)	0(0)	9(9)	5(0)	51(41)	┐*
III	12(12)	2(1)	2(1)	4(4)	2(1)	3(0)	25(19)	┘(60)
Total	65(60)	8(4)	11(8)	4(4)	14(13)	19(8)	121(97)	

Note: The slope failures in some spots out of the 97 locations were considered to be complex having more than two types of failure. The number of failures counted them separately is shown as a figure without a parenthesies, and that in a parenthesis indicates the real number of location. That is, 121 failures have been observed in 97 spots in total. The number with an asterisk, 60, coincides with that in Table 2.3-12 in Capter 2.

3.3.1 Slope Failure (Type a)

Slope failure can occur on a slope of soil or rock. Sometimes a slope consists of part soil and part rock, and in such case a slope failure on both parts often occurs at the same time. A slope failure accompanied with a small landslide may also be found.
Slope failures account for nearly 50 % of all dangerous and potential disaster spots in the study area.
The slope stability by rock type is as follows:

Stability	SS	MS	SH	SL	Total
I	2	6	6	8	22
II	6	8	6	11	31
III	0	2	4	6	12
Total	8	16	16	25	65

Note: 1) SS = sandstone, MS = mudstone,
SH = shale, SL = slate
(All rocks are Paleozoic rocks.)
2) The total number shown in the table above is not in accordance with that in Table 3.3-3. This is because a combination of different rocks fell in the same location.

Slope failures occur with Paleozoic rocks and are not generally found in Mesozoic or Tertiary rocks.
However, it does not mean that in Mesozoic and Tertiary rocks, slope failure does not occur. It is because the distribution of those rocks is very limited in the study area. Among Paleozoic rocks, slate has a highest proportion of slope failure. This is due to the notable forming of joints in the strata.

The frequency of occurrence for slope failure has an origin in the difference of rock and geological structure. But difference in topography is one of the most important factors.
The following cases have a high incidence of slope failure:

a) Long slopes
b) Convex slopes
c) Steep slopes
d) Abundant ground water

e) Lack of vegetation
f) Rich precipitation

Since the cause of a slope failure is complicated, it is difficult to make an accurate prediction for future slope failure. However, places subject to failure are likely to undergo re-occurrence and extension. At these places, especially for stability Grade III, it is desirable to take proper measures.

Slope failures are found throughout the existing road. Between Santa Bårbara and Challa (Point C) and near Carrasco, more slope failures than in any other sub-sections are found.

3.3.2 Embankment Failure (Type b)

The number of failures observed of this type was only 8 and that it is 7 % of the total number.
Although this failure type has been sub-divided into four categories, from Type B-1 to B-4, in Fig. 3.3-1, most of failures for this group in the study area belong to Type B-2 or B-3, which are failures due to displacement of the embankment itself. The difference of Type B-2 and B-3 is from fill material; i.e., Type B-2 is for the case that the fill material is soil, and Type B-3 if for the case of rock. In almost all of these cases, a slide of fill material is the surface of the natural slope beneath the embankment. In other words, failures occurred because the embankment was constructed directly on a steep natural slope without any preparation work such as bench-cutting.

It was confirmed that all places except for two could be classified as this type and have been stabilized already or are considered to have no influence over the improved road.

Stability of this type of failure categorized by kind of rock fill or soil fill parent rock is shown below :

Stability	SS	MS	SH	SL	Total
I	0	1	2	0	3
II	0	2	0	1	3
III	0	0	1	1	2
Total	0	3	3	2	8

Note : SS = sandstone, MS = mudstone,
SH = shale, SL = slate
(All rocks are Paleozoic rocks.)

The location of two places, which have been categorized as stability Grade III in the above table and seem to require attention in the Study, are at Chojña (Point J, Location number in Table 3.3-1 = 126) and near Carrasco (Location number = 248). The former is a combination of slope failure and embankment failure and when some measures for slope failure are made, no consideration of embankment failure will be required. In the latter case, a failure occurred and is continuing due to bad maintenance of the drainage pipe installed beneath the existing road.

3.3.3 Rock Falls (Type c)

The number of places investigated and classified as this type of failure are shown below:

Stability	SS	MS	SH	SL	Total
I	1	5	0	0	6
II	0	1	0	2	3
III	0	0	0	2	2
Total	1	6	0	4	11

Note : SS = sandstone, MS = mudstone,
SH = shale, SL = slate
(All rocks are Paleozoic rocks.)

Rock falls caused by open cracks in rocks (Type C-2 in Fig. 3.3-1) are predominant. On the other hand, Type C-1 occurs only in an area where terrace and talus deposits exist and can hardly be found in the study area at all. Type C-3 failures are encountered only in the Bella Vista region.

At Location No. 41 in Table 3.3-1 near Challa (Point C), blocks of slate fall down very frequently on the road. This dangerous section continues for 250 m along the road.

3.3.4 Land Slide (Type d)

Land slide occurs only in Tertiary rock zone near Bella Vista. This Tertiary rock consists of hard sandstone and weathered fragile mudstone.

Near Bella Vista , the topography is gentle but appears to be a place where landslide could occur easily. Four active land slides with relatively a small scale were found in the investigation. They are all rotational slip failures on clear scarps and sedimentary flat planes of land slide deposits. The ends of these land slide reach the existing road, so it is quite within the bounds of possibility that these land slides could damage the road in the future.

Furthermore, big lineament was found by aerial photo investigation at the hill slope about 100 m higher than the road elevation. It is 1.6 km in length parallel to the road.
By field investigation, it was confirmed that the lineament was a land slide scarp (from 3 to 5 m in height) with clear cracks and that the movement of this land slide did not reach the road and has now been stabilized.

In any event, careful attention must be paid to the planning of the improved road in this region.

3.3.5 Debris Flow and Earth Flow (Type e)

Debris flow is a rapid rush of unstable earth and sand along a channel like landscape. The source of rushing material is upstream, and it flow down with water in many cases.

Number of debris flow potential places is 14 as shown below:

Stability	SS	MS	SH	SL	TSS	Total
I	0	1	0	2	0	3
II	1	3	1	4	0	9
III	0	1	0	1	0	2
Total	1	5	1	7	0	14

Note : SS = sandstone, MS = mudstone,
SH = shale, SL = slate
(All rocks above are Paleozoic rocks.)
TSS = Tertiary sandstone

Material of Paleozoic rock origin consist of small rubble, sand and silt, so that the flow may be called an earth flow rather than a debris flow. The diameter of the rubble is from 15 to 20 cm.

At location No.6 in Table 3.3-1 (near Santa Bårbara), a natural slope which has been damaged by failures and has many cracks on it was notably prominent.
The deposits of loose earth, sand and gravel piled up by a slope failure have the potential to flow down as a debris flow. Once this debris flow occurs, the road would surely be severely damaged, so careful attention must be paid and a appropriate countermeasures must be taken into account, here.

Two places, where some large scale debris flows has surely occurred in the past, exist at Santa Ana (Point (L), Location No. in Fig. 3.3-1 = 154) and between Location No. 199 and 212. At Santa Ana, a fault, which runs parallel to a ridge there, is considered to be a source of material for the flows. However, in both places, stability Grade II has been given because they are not so active now.

At Location No. 339 in the Bella Vista region, many boulders of Tertiary sandstone were found. These unstable boulders have enormous destructive power when they begin to flow down a water course. Even if they seems to be stable at present, some preventive countermeasures must be prepared in the Study.

3.3.6 Fractured Zone of Fault (Type f)

The potential disaster places from Type a to Type e which were previously described until here have been classified mainly by the type and form of failure. However, Type f is a little different from them and is defined as a group of places which are in a fractured zone.

Nineteen fractured zones were found in the study area, but at one place out of the nineteen no fault could be confirmed by the field reconnaissance team. The surface of the fault line at this place is considered to have been covered by talus materials.
In other words, the existence of eighteen faults, which are named Fault-A to Fault-R on the Geological Maps attached to this report, were confirmed in the region, and all those faults are on a small scale having a fault line width of from 10 to 30 cm, which consist of clay. The width of the fractured zone along the fault line is in between 10 and 30 m.

At a fractured zone, rocks are fractured into rubble that are several tens of centimeters in diameter. But the matrix of the fractured zone is well consolidated.
Therefore, the slopes are generally stable and stability Grade I occupies 60 % of all fractured zones.

The fractured zones accompanied with slope failure, embankment failure and rock falls are found in eight locations. Among them, three locations have been classified as stability Grade III.
They are at;

a) Location No. 41 (Fault-D)
b) Location No. 92 (Fault-G)
c) Location No.248 (Fault-N)

As a fractured zone generally is not as stable as it looks, careful attention must be paid for such a zone during the course of the Study.

The number of places in this class are shown in the following table.

Stability	SS	MS	SH	SL	TSS	Total
I	0	2	1	6	2	11
II	1	1	0	3	0	5
III	0	0	1	2	0	3
Total	1	3	2	11	2	19

Note : SS = sandstone, MS = mudstone,
SH = shale, SL = slate
(All rocks above are Paleozoic rocks.)
TSS = Tertiary sandstone

3.3.7 Ground Water

Ninety three locations where spring water was observed are shown in Table 3.3-2. The number of locations where spring water is flowing out from a rock bed, from between rock and talus deposits, and from talus deposits is 25, 15 and 53 places out of 93 locations, respectively.
The amount of water flowing out at each place is generally a little; less than 10 liters/min. at 70 places.
A maximum amount of nearly 90 liters/min. of water was observed at Location No. 12 and No. 304.

Spring water often occurs along a fault. For example, Location No. 11 and No. 13 are along Fault-B same as No. 57 and No. 59 are along Fault-H.
Locations having spring water are concentrated in a region where cracked bed rock is widely distributed such as between Santa Bårbara and Challa, between Puerto Leon and Chojña, and near Carrasco.

Treatment of this spring water must be taken into account in the Study.

3.4 Drilling Survey

Drilling surveys and laboratory tests on rocks obtained were carried out at six locations in the study area.

The total length of drilling performed was 87.2 m. Standard Penetration Tests (SPT) were conducted for the soft ground of talus deposits and very weathered Tertiary rock. Core samples of hard rock obtained by drilling were transported to the SNC Laboratory to undergo Unconfined Compression Test (UCT).

Quantity of survey and tests are tabulated in Table 3.4-1.

Table 3.4-1 Summary of Drilling and Tests

Boring No.	Location (Km)	Geological Type	Length of Drilling (m)	Standard Penetration Test (Times)	Unconfined Compression Test (Piece)	Specific Gravity Test (Piece)
P1	0.9	Talus (Quaternary)	10.6	10	-	-
P2	8.9	Slate (Paleozoic)	15.0	-	2	2
P3	37.8	Slate (Paleozoic)	15.5	-	26	26
P4	81.8	Weathered Mudstone (Paleozoic)	15.5	-	-	-
P5	105.2	Sandstone (Mesozoic)	15.5	-	5	5
P6	112.2	Weathered Mudstone (Tertiary)	15.1	15	-	-
Total	-	-	87.2	25	33	33

3.4.1 Description of Materials by Bore Hole

(1) P1 (L = 10.6 m)

This place is located near Santa Bàrbara where sedimentary talus deposits exist, so SPTs were also performed when drilling.
Drilling was done as far as 10.6 m vertically down from the ground surface, but it did not reach a bed rock.

Materials of the sub-soil was confirmed to contain rubble and silty fine sand.
The result of SPT, N-value, fluctuated greatly, i.e., in the sub-surface upto 3 m deep it was between 67 and 179, and it was between 23 to 49 at a depth of 3 to 10 m.
This was surely due to the existence of boulders among the deposits.

(2) P2 (L = 15.0 m)

Drilling at this place was done into Paleozoic slate strata being black and hard with many joints. So that, only two specimens for Unconfined Compression Test (UCT) could be obtained from cored rock drilling here.

(3) P3 (L = 15.5 m)

This place also is in a zone of Paleozoic slate between Puerto Leon and Chojña. The rock was gray colored and very hard with few joints.
26 specimens for UCT were obtained from this hole.

(4) P4 (L = 15.5 m)

This place is located at near Carrasco, where Paleozoic mudstone exists.
This mudstone is black or gray in color and hard, but considerably weathered and fragile. It forms many fine cracks, so that it is easily broken when hit by a hand hammer.
No sample for UCT was obtained here.

(5) P5 (L = 15.5 m)

The place is between Carrasco and Bella Vista.
Drilling was carried out completely upto Mesozoic sandstone with a yellowish gray or light gray color.
This sandstone was very hard, and many joints were observed in the upper part of the hole. In the lower part, it changed to be silicious with mudstone strata with a thickness of 30 to 40 cm was found in between this sandstone.
Only five samples for UCT were obtained.

(6) P6 (L = 15.1 m)

This drilling was to obtain characteristics of the Tertiary mudstone which exists in the Bella Vista region.
This rock was found to be severely weathered and to contain hard sandy mudstone.
Color of the rock was a reddish brown.
As the rock was so soft, a SPT was performed here. N-values showed 60 to 80 below 4 m from the surface.

3.4.2 Unconfined Compression Test

The results of the tests performed are shown in Table 3.4-2.

UCT (Unconfined Compression Test) of weathered slate from P2, fresh slate from P3 and Mesozoic sandstone from P5 were carried out.
The average UCT strength of each kind of rock are shown below:

Rock	Average UCT strength (kg/cm^2)	Samples (pieces)
Weathered slate	250	2
Fresh slate	530	26
Mesozoic sandstone	640	5

It seems that these values are reasonable, but it is necessary to consider that rocks change their dynamic strength under the influence of weathering.

Cohesion and the internal friction angle of the rock material is described in Fig. 3.2-2.
These values for talus deposits at P1 and weathered mudstone at P6 were calculated based on the N-values.
The physical features or coefficient of rocks is shown in Table 3.4-3.

Table 3.4-2 Result of Rock Test

Boring No			P1	P2	P3	P4	P5	P6
Geological Type			Talus (Quaternary)	Slate (Paleozoic)	Slate (Paleozoic)	Weathered Mudstone (Paleozoic)	Sandstone (Mesozoic)	Weathered Sandstone (Tertiary)
Number of Test			10	2	26	0	5	15
Standard Penetration Test	N-Value (Times)	Max.	179	-	-	-	-	115
		Min.	23	-	-	-	-	48
		Ave.	53	-	-	-	-	80
	Fatigue (Kg/cm2)	Max.	4.8	-	-	-	-	3.6
		Min.	3.4	-	-	-	-	2.7
		Ave.	3.9	-	-	-	-	3.1
Unconfined Compression Test	Breaking Strength (Kg/cm2)	Max.	-	327	882	-	847	-
		Min.	-	175	290	-	457	-
		Ave.	-	251	531	-	637	-
	Static Modulus of Elasticity (Kg/cm2) (x 10,000)	Max.	-	2.52	29.94	-	8.19	-
		Min.	-	2.73	4.68	-	4.39	-
		Ave.	-	2.63	12.55	-	6.53	-
Specific Gravity Test	Apparent Specific Gravity (G)	Max.	-	2.82	2.86	-	2.65	-
		Min.	-	2.81	2.81	-	2.58	-
		Ave.	-	2.82	2.84	-	2.55	-
	Bulk Density (Kg/cm3)	Max.	-	2.80	2.87	-	2.44	-
		Min.	-	2.78	2.80	-	2.36	-
		Ave.	-	2.79	2.84	-	2.41	-
Assumed Design Constant	Internal Friction Angle	Deg.	43	30 - 45	55 - 65	20 - 30	40 - 55	0 - 15
	Cohesion (C)	Kg/cm2	0 - 5	10 - 20	40 - 50	10 - 20	20 - 40	5.3

Note:
Deg.(P1)= (15*N)**(1/2) + 15
C(P6)= 1/15 * N

Table 3.4-3 Physical Constant Estimated from Grade of Rocks

Grade of Rocks	Modulus of Deformation (Kg/cm2)	Cohesion (Kg/cm2)	Internal Angle of Friction (Degree)	Seismic Velocity P(Kg/sec)	Rebound by Schmidt Hammer
A - B	More Than 5.0 x 10000	More Than 40	55 - 65	More Than 3.7	More Than 36
C(H)	5.0 x 10000 to 2.0 x 10000	40 - 20	40 - 55	3.7 - 3.0	36 - 27
C(M)	2.0 x 10000 to 0.5 x 10000	20 - 10	30 - 45	3.0 - 1.5	27 - 15
C(L)	0.5 x 10000 to 0.2 x 10000	10 - 15	15 - 40	1.5 - 1.0	Less than 15
D (Sandy Soil)	0.2 x 10000 to 0.01 x 10000	0 - 5	15 - 40	1.5 - 1.0	-
D (Clayey Soil)	Less than 0.01 x 10000	5 - 10	0 - 15	Less than 1.0	-

From: "Rock Mass Classification", (by Kikuchi), 1989

4. BASIC STUDY ON ROAD IMPROVEMENT

4. BASIC STUDY ON ROAD IMPROVEMENT

4.1 Fundamental Problems with the Existing Road

From the aforementioned descriptions detailed in this report, the fundamental problems with the existing road can be summarized, and the following four problems areas can be classified as being the most critical points;

1) Excessive transportation costs.

Due to poor road conditions the practical running speed of vehicles is fairly slow, and hence, it takes a lot of time to pass along this section. Coupled with the fact that the rate of wear and tear on vehicles here is higher than usual (due to the irregular, rough road surface), the result is that this road is considerably more costly as a transportation mode than other roads.

2) Driver exhaustion.

The presence of dilapidated bridges, narrow road with a small horizontal curvature, combined with short sight distances make it physically and mentally exhausting for drivers who use this road.

3) Frequent road closure.

Often disasters such as slope failure, rock fall and shoulder displacement make it necessary to close the road. Due to the lack of an information system, vehicles in many cases of occurrence of road closure caused by a such disaster, have to wait for the completion of repair works of disaster at the site after coming down there without a previous notice of that road closure.
System of repair and maintenance of the disasters sometimes does not function well and prolong the closed period.

4) Frequent severe traffic accidents.

Not only the frequency but severeness of accidents are serious, along this section.

These phenomena are caused by a number of various reasons, listed up below. It is just and proper that the Project aims to eliminate or at least improve these factors.

a) Poor geometric (horizontal and vertical) alignment
b) Insufficient cross sectional width and composition
c) Poor road surface treatment
d) Existence of dangerous structures
e) Insufficient disaster prevention and drainage facilities
f) Absence of traffic safety facilities and sign boards
g) Poor repair/maintenance and information system

Regarding factors a), b) and c) out of the above mentioned seven, there is an apparent need to study and compare several methods of improvement, in order to determine the best since there are a few alternatives to be considered for each factor.
The results of this study, which are described in this Chapter, influence the evaluation of the feasibility of the Project directly.

The study on pavement, factor c), as there are not many optional ideas, will be mentioned in the following Chapter. In respect of the factor d), it will be studied together with improvement of factors a), b) and c).

4.2 Design Standard and Criteria

4.2.1 Basic Policy for Road Improvement

The following basic policy for the Study is confirmed by SNC:

a) The road must be improved in accordance with the Bolivian Standard (Manual y Normas para el Diseño Geomèrico de Carreteras-SNC: hereinafter referred to as "Norma".),

b) The improved road must be a two-lane paved road,

c) In order to decrease the required funds for the improvement, the Study aims at maximum utilization of the existing road, and

d) The result of the Study must be balanced and conform(from an engineering viewpoint) with the improvement program for adjacent sections of the same road.

4.2.2 "Norma" and Road Classification

"Norma" specifies various engineering criteria for roads except for streets in Bolivian the cities. The roads are classified by "Norma" into six Classes depending on the expected traffic volume for the road, as shown in Table 4.2-1.

Table 4.2-1 Classification of Roads given in "Norma"

Class	Traffic Volume	Number of Lanes
O	15,000<ADT	4=<
I.A	5,000<ADT<15,000	4
I.B	1,500<ADT< 5,000	2
II	700<ADT< 1,500	2
III	300<ADT< 700	2
IV.A	200<ADT< 300	2
IV.B	ADT< 200	2

Note: 1) ADT is "Average Daily Traffic" at ten years ter the date of completion of construction.

2) Number of Lanes is in both directions.

The principal geometic design criteria stipulated in "Norma" are tabulated in Table 4.2-2.

Table 4.2-2 Geometric Standard of Road (SNC)

Class	Topo-graphy	Design speed (km/h)	Sight distance (m) Stop	Sight distance (m) Passing	Min.radius H. curves (m)	Super-elevation (%)	Vertical grade (%)	K-value Convex	K-value Concave	One lane width (m)	Shoulder width (m)	Min.median width (m)	Desirable pave-ment
0	flat land	120	300	800-(525)	665	6	3-(5)	219-112	77-53	3.50	3.5-3.0	10-18	Rigid
	hilly	100	210	680-(425)	415	6	3-(5)	107- 58	52-36	3.50	3.5-3.0	10-18	Rigid
	mountain	80	140	560-(325)	250	6	3-(5)	48- 30	32-24	3.50	3.5-3.0	3-10	Rigid
	steep mount	60	85	420-(240)	135	6	4-(6)	18- 14	17-15	3.50	3.0-2.5	3-10	Rigid
I.A	flat land	100	210	680-(425)	415-(375)	6-(8)	4-(5)	107- 58	52-36	3.50	3.5-2.5	3-10	Flex.
and	hilly	80	140	560-(325)	250-(210)	6-(10)	4-(6)	48- 30	32-24	3.50	3.5-2.5	-	Flex.
I.B	mountain	60	85	420-(240)	135-(115)	6-(10)	5-(7)	18- 14	17-15	3.50	3.5-2.5	-	Flex.
	steep mount	40	45	270-(160)	55-(45)	6-(10)	6-(8)	10- 9	12-11	3.50	3.0-2.5	-	Flex.
II	flat land	100	210	680-(425)	415-(375)	6-(8)	4-(5)	107- 58	52-36	3.65-3.35	3.0-2.0	-	Asph.Mac.
	hilly	70	110	490-(280)	185-(155)	6-(10)	5-(6)	29- 20	24-19	3.65-3.35	3.0-2.0	-	Asph.Mac.
	mountain	50	65	350-(200)	90-(75)	6-(10)	6-(7)	10- 9	12-11	3.65-3.35	3.0-2.0	-	Asph.Mac.
	steep mount	30	30	180-(120)	30-(25)	6-(10)	7-(8)	2	4	3.65-3.35	3.0-2.0	-	Asph.Mac.
III	flat land	80	140	560-(325)	250-(230)	6-(8)	6-(7)	48- 30	32-24	3.50-3.00	3.0-1.0	-	Gravel
	hilly	60	85	420-(240)	135-(115)	6-(10)	7-(8)	18- 14	17-15	3.50-3.00	3.0-1.0	-	Gravel
	mountain	40	45	270-(160)	55-(45)	6-(10)	7-(8)	10- 9	12-11	3.50-3.00	3.0-1.0	-	Gravel
	steep mount	30	30	180-(120)	30-(25)	6-(10)	7-(8)	1	2	3.50-3.00	3.0-1.0	-	Gravel
IV.A	flat land	60	85	420-(240)	135-(115)	6-(10)	7-(8)	18- 14	17-15	3.35-3.00	3.0-0.5	-	Stab. Soil
	hilly	40	45	270-(160)	55-(45)	6-(10)	8-(9)	10- 9	12-11	3.35-3.00	3.0-0.5	-	Stab. Soil
	mountain	30	30	180-(120)	30-(25)	6-(10)	9-(10)	2	4	3.35-3.00	3.0-0.5	-	Stab. Soil
	steep mount	20	20		12	6-(10)	10	1	2	3.35-3.00	3.0-0.5	-	Stab. Soil
IV.B	flat land	60	85	420-(240)	135-(115)	6-(10)	7-(8)	18- 14	17-15	3.35-3.00	3.0-0.5	-	Stab. Soil
	hilly	40	45	270-(160)	55-(45)	6-(10)	8-(9)	10- 9	12-11	3.35-3.00	3.0-0.5	-	Stab. Soil
	mountain	30	30	180-(120)	30-(25)	6-(10)	9-(10)	2	4	3.35-3.00	3.0-0.5	-	Stab. Soil
	steep mount	20	20		12	6-(10)	10	1	2	2.75	3.0-0.5	-	Stab. Soil

Note 1) Values in parentheses are allowable values against desirable ones.
2) Flex., Asph. Mac. and Stab. Soil mean flexible, asphalt macadam and stabilized soil surface pavement.

4.2.3 Classification and Design Criteria of the Project Road

(1) Classification

Assuming that the road improvement will be completed by the year 2000, the projected future traffic volume (ADT) in 2010 (the year is taken as ten years after the completion of the improvement) is examined by determining the class of road according to "Norma". That is 1,516 vehicles for the section between Santa Bárbara and Caranavi and 623 vehicles for between Caranavi and Bella Vista. This is described in Chapter 5 of this report. If the project section was not divided into two different regions, the ADT could be considered to be between 1,500 and 5,000 vehicles/day.
The topography in the study area is apparently very steep and mountainous.

From this, the Project Road must be assigned to Class I.B, and all values in the row labeled Class I.B,steep mountain on Table 4.2-2 should be adopted for the Study.

(2) Geometric design criteria for the Project Road

Adding items which are not included in Table 4.2-2, all values of geometric design criteria for the Study are listed in Table 4.2-3.

(3) Items from "Norma"

Several items in Table 4.2-3 are outside the scope of "Norma". This is firstly due to the topography in the study area, and secondly, in maintaining conformity with the improvement plans for other sections of the same road (i.e., such as the Cotapata Section - Santa Bárbara and Section Bella Vista - Yucumo).

1) Comparison with other sections
A Comparison of some items of the geometric design criteria is shown in Table 4.2-4.

Referring to the values in Table 4.2-4 and Table 4.2-2, it can be easily understood that some criteria had to be set outside the score of "Norma" in the case of improvement planning for other sections, where topography is hard, and it was required for decreasing the requred improvement funds.

Table 4.2-3 Geometric Design Criteria for Study

Road classification		Class I.B, very mountainous	
Design vehicle		semi-trailer truck (WB-40)	*1
Design speed		40 km/h	
Stopping sight distance		45 m	
Passing sight distance		160 m	
Radius of horizontal alignment	desirable :	> 50 m	*2
	minimum :	= 45 m	
Superelevation rates	desirable :	< 8 %	*2
	maximum :	= 10 %	
Minimum radius for 2% superelevation of	(minimum)	300 m	*3
Minimum radius without superelevation		1400 m	
Grades for vertical alignment	desirable :	< 6 %	
	maximum :	= 8 %	*4
K-value : concave vertical curves	desirable :	12	
	minimum	11	
: convex vertical curves	desirable :	10	
	minimum :	9	
Normal cross slope		2 %	
Lane widths		3.50 m	
Widening on curves for two lanes	250 m<R<300 m :	0.4 m	*2
(R = Radius of horizontal curves)	145 <R<250 :	0.7 m	
	100 <R<145 :	1.0 m	
	80 <R<100 :	1.3 m	
	65 <R< 80 :	1.6 m	
	55 <R< 65 :	1.9 m	
	45 <R< 55 :	2.2 m	
Shoulder width	normal :	1.0 m	*2
	exceptional :	0.6 m	*2
Total width of cross section		>10.4 m	*2
Width of side ditch		1.0 m	
Clearance Height		>5.5 m	

Note: *1 "Norma" has a category of SR (Semi-trailer), which is equivalent to WB-50 in AASHTO Specification, but one equal to WB-40 in AASHTO's. "(3) Items out of "Norma"" in this section.
*2 See, "(3) Items out of "Norma"".
*3 Minimum superelevation 2 % coincides to normal cross slope.
*4 A continuous length of road with a7-8% grade must be less than 400 m.

Table 4.2-4 Comparison of Geometric Design Criteria for each Section of Road No. 3

Section	Projected ADT (vehi./d)	Topography	Design vehicle	Design speed (km/h)	Minimum radius (m)	Vert. grade (%)	Shoulder width (m)
La Paz-Cotapata	>1500	mountainous	SR	50	80	7.0	1.0
Cotapata-S/Barbara	>1500	steep mountain	CO	40-(15)	55-(25)	8.0	0.5
S/Barbara-B/Vista	>1500	steep mountain	WB-40	40	50-(45)	8-(10)	1-(0.6)
B/Vista-Quiquibey	?	mountainous	CO	50	80-(65)	7.0	1.0
Quiquibey-Yucumo	?	?	?	35	35	7.5	1.0
Yucumo-S/Borja	?	flat	SR	100	350	5.0	1.0
S/Borja-Trinidad	> 700	flat	SR	100	415	4.0	1.0

Note : The values in parenthesis are for exceptional use.
?: They are not recorded clearly in the SNC documents.

2) Shoulder width

Considering the topography in the study area, it is impractical to comply with the minimum shoulder width of 2.5 - 3.0 m as stipulated by "Norma". It would be quite the same in other sections with a similar topography.

Referring to the criteria for other sections, it was confirmed that the standard shoulder width in the Study was 1.0 m. (See, Table 4.2-4.)

3) Design vehicle and widening on curves

The widening width for the on curves of the carriage way is fully dependent on the type of Design Vehicle. "Norma" specifies the following four categories regarding Design Vehicle classification;

a) VT : passenger car,
b) CO : conventional truck,
c) O : big size bus, and
d) SR : semitrailer (equivalent to AASHTO's WB-50).

The required widening widths on curves in the case of a design speed of 40 km/h calculated according to the formulas in "Norma", AASHTO and the Japanese Standard, are tabulated in Table 4.2-5.

When adopting SR as the Design Vehicle, the required widening width would be 3.26 m, that is, the total road width would be nearly equal to that for three carriage ways.
Considering that the road length of the curved sections along the study road would be fairly long even after improvement, it would surely result in a larger Project cost.

On the other hand, although the present traffic on the study road does not include semitrailers, it must take into consideration that semitrailers voill eventually use the improved road, as has happened with other newly paved roads in Bolivia.

Table 4.2-5 Required Widening for Turning Curve (for 2 lanes, design speed 40 km/h)

Radius of curve	"Norma" (SR)	AASHTO (WB-50)	AASHTO (WB-40)	Japanese (s/trailer)	"Norma" (O)	"Norma" (CO)
45 m	3.26 m	3.17 m	2.26 m	2.42 m	2.29 m	1.61 m
50	2.96	2.86	2.05	2.18	2.08	1.47
100	1.59	1.47	1.07	1.09	1.16	0.85
150	1.12	1.00	0.73	0.73	0.83	0.63
250	0.73	0.60	0.44	0.44	0.56	0.43

Note : See, Appendix 4-1.

Subsequently, the following were confirmed suitable for the Study:

a) To adopt, as Design Vehicle, WB-40 which is a little smaller than SR or WB-50,
b) To decrease the width of the road shoulder in propor tion to the required widening at places with a curve radius of less than 80 m. As a result, the total road width will be less than 10.4 m as shown in Fig. 4.2-1 and Fig. 4.2-2 for the next section, and
c) To install appropriate traffic sign boards for regulatory and warning signs on curves with less shoulder width.

The values for widening by adjusting the horizontal curve radius shown in Table 4.2-3 were practically, modified values of those given in Table 4.2-4 (WE-40).

(4) Superelevation and the radii of horizontal alignments

The desirable values for superelevation and radii of horizontal curves in Table 4.2-2 are 6 % and 55 m, respectively. However, there is a stipulation in "Norma" to modify the desirable value of superelevation when the objective area is not so cold that the road surface frosts over. According to this stipulation, a value of 8 % can be adopted as the superelevation for the Study. In this case, the desirable value for the radius for a horizontal curve can also be reduced to 50 m.

(5) Design criteria for structures

Regarding structures such as bridges, the following points were confirmed for the Study;

a) live load on the structure : HS-20 in AASHTO Spec.
b) allowable concrete strength: 350 kg/cm^2 for pre-stressed concrete
: 210 kg/cm^2 for others
c) other items and factors : according to the AASHTO Spec.

4.2.4 Illustrative Cross Section

Figuers 4.2-1, 4.2-2 and 4.2-3 illustrate typical cross sections for the improved road as a result of the considerations described in the previous section.

TYPICAL CROSS SECTION (I)

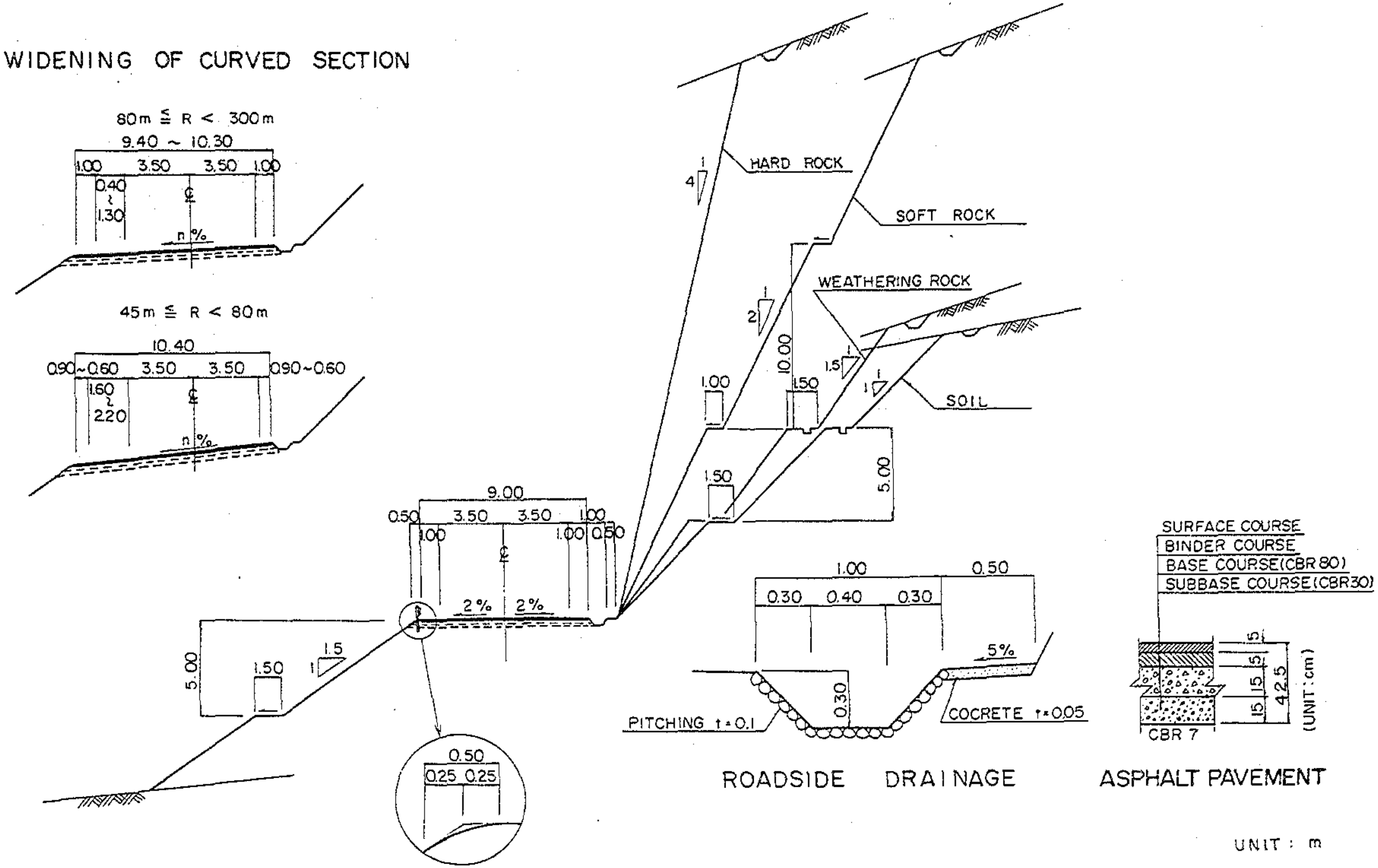


Fig. 4.2-1 Typical Cross Section

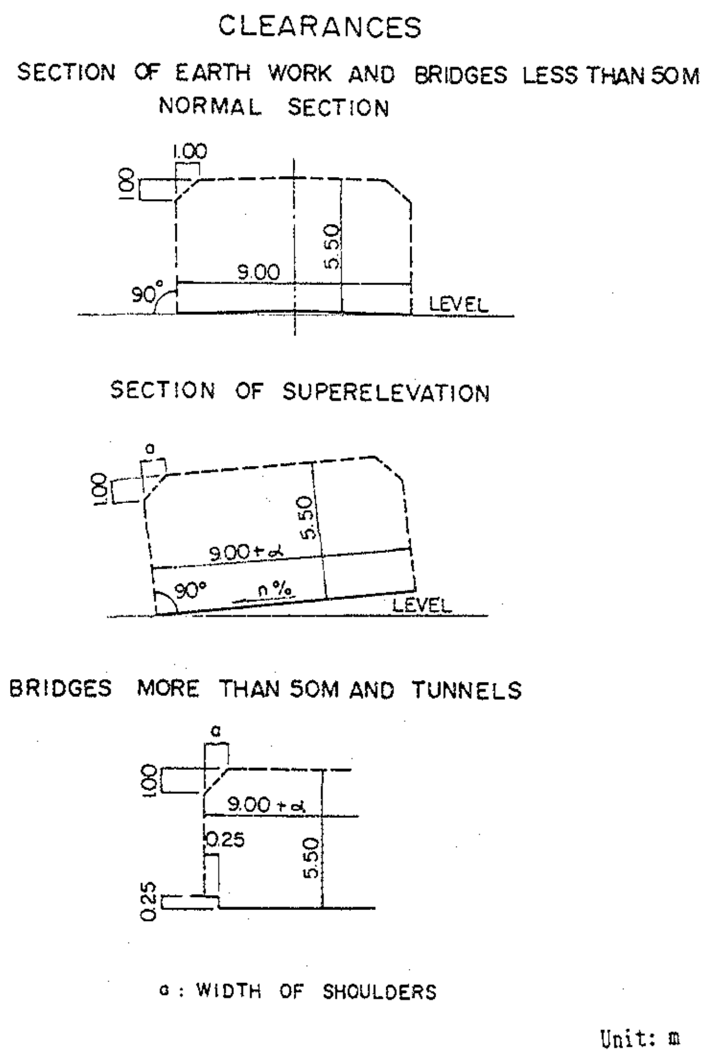


Fig. 4.2-2 Clearances

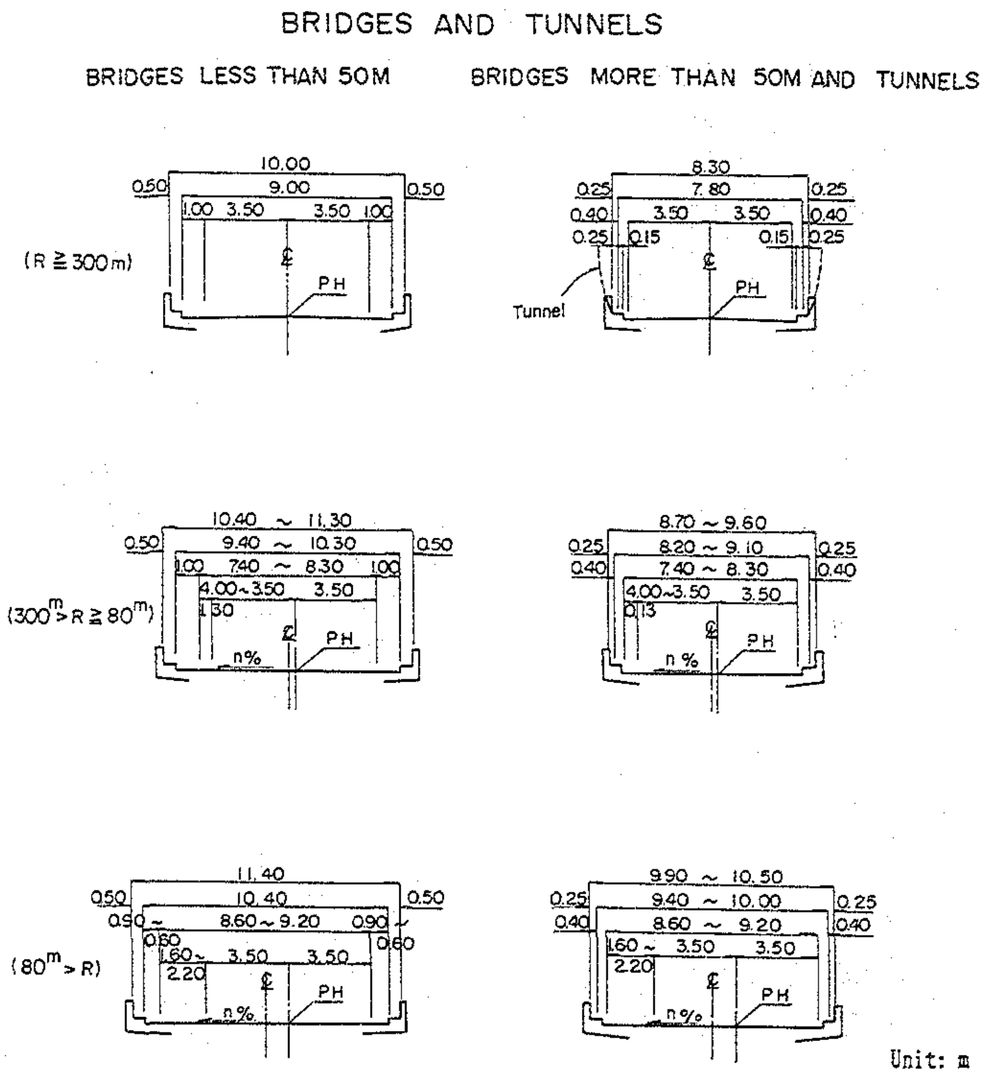


Fig. 4.2-3 Typical Cross Sections for Bridges

4.3 Geometrical Alignment Alternatives

4.3.1 General

It can be said that the existing road alignment roughly follows the shortest route between Santa Bárbara and Bella Vista. Viewing the surrounding topography, it is impossible to find a more advantageous route to construct a new alignment, than where the existing road currently runs.
Therefore, the alignment for improvement must basically be similar to that of the existing road.

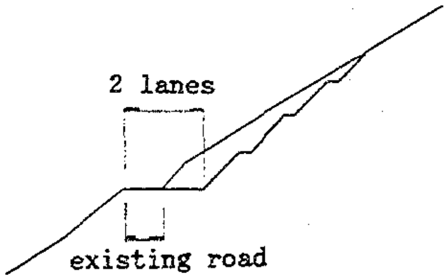
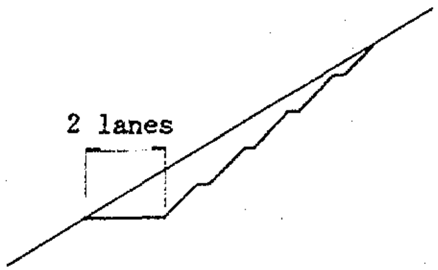
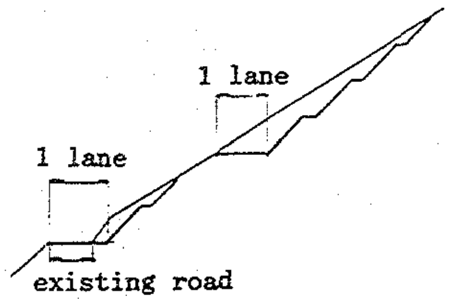
For road improvement in this case, the widening of the road up to two lanes is the primary study item.
Table 4.3-1 shows the result of comparing a few ideas on how to widen the road in a typical landscape.

It is apparent from the table that widening of the existing road up to two lanes of carriage way, is the most favorable, especially from an economic viewpoint.
Based on this understanding, an alternative study for geometrical improvement is made mainly for limited subsections where; 1) the horizontal or vertical alignment of the existing road does not comply with the aim criteria described in "4.2 Design Standard and Criteria", and 2) a disaster is likely to occur in the future and it is considered better to change the road alignment so as to avoid damage at such a time.

In the following sections from 4.3.2 to 4.3.7, the results of the study aiming to find the most appropriate road alignment are described by sub-section from Santa Bárbara to Bella Vista.

With regard to bridges, the sites and the required dimensions are also studied together in the road alignment study, and the result is described in the section 4.4.

Table 4.3-1 Comparison Table For Method of Road Widening

Alternatives		Widening of Existing Road	New Alignment	Dual Carrage Way Road
Sketch				
Approx. Const. Cost		US$ 1,277,000 /km	US$ 1,950,000 /km	US$ 1,635,000 /km
Comparison	Economic Effect	Cost is the lowest	50% higher than the lowest	30% higher than the lowest
	Obstruction of Traffic Flow	Reasonable	Reasonable	Inconvenience of going in and out the neighbouring area.
	Ease of Construction	Construction is possible conducting time limit to traffic movement.	Construction is possible without obstruction of traffic movements.	Construction is possible if traffic flow is converted into the new alignment.
	Traffic safety	Fair	Fair	Traffic movement is very safe due to the oneway
Complehensive Evaluation		The alternative of "Improvement of Existing Road" is significantly supiorior to other alternatives from the view point of not only the cheapest cost but also the overall evaluation. Therefore, the alternative "Improvement of Existing Road" isadopted.		

4.3.2 From Santa Bårbara to Point F

As described in "2.3 Existing Project Road", the existing road in this sub-section runs in an undulating fashion along the Rio Coroico. The positioning of the road can be said to be adequate. That is to say, that while the natural slope gradient, in general, is gentler at a higher part of the mountain than at a lower part in this section, the road was constructed approximately at the point of varying gradient, as shown in Photo-3.
The road runs connecting the places as it descends, when a big valley is encountered, so as to cross the valley.

In other words, if the road was constructed at a higher level of natural slope, the road length and vertical alignment would have been longer and more sever than those encountered on the existing road. On the contrary, if con structed at a lower level, the construction cost would have been far greater, due to the more irregular and steep topography.

Under such a situation, it was concluded that the required geometrical improvement for this sub-section must certainly be a small change in horizontal alignment at various ,ponts and also a widening of the existing road. Hence, no alternative study is needed, except for the cases described below.

At Point A, B, C and F, new bridges must be constructed, and the best location of the bridge site must be selected considering the required bridge length and cost for each of the candidate sites.
In particular, a large scale debris flow could possibly occur at Point A, so a new bridge is considered necessary to detour that dangerous section.

The result of alternative studies on these places is described later in "4.4 Alternatives for Bridge Structures" of this Chapter.

4.3.3 From Point F to Point K

(1) Existing Conditions and Method of Improvement

Between Point F and J+2.4 km, the Study Road is located alongside the Coroico River and the maximum horizontal distance from the river bank to the road is less than 50 m.

A few horizontal curves with a radius of less than 50m are encountered in the existing road of this sub-section. The number of such small curves on average is 0.3 per km here, and comparing with the average of 0.9 per 1 km over the whole extension of the Study Road, the horizontal alignment of the existing road in this sub-section is said to be the most satisfactory. In respect to the vertical alignment, the road descends gently, following the river bank, and no probles where encountered in this sub-section.

The aspect from Point H+2.5 km to I+0.35 km, from the cross sectional viewpoint, shows one of the most severe conditions on the whole Study Road, consisting of very steep cliffs on the mountain side of the road with an extension of 750 m as described in " 2.3.2, Result of Site Reconnaissance ". The width of the existing road is so narrow that a large construction cost will be required to widen it. Therefore, alternative ideas are presented here for the improvement.

From Point J+2.4 Km to K the road passes near to the point of change in geographical gradient for the mountain slope, as does the extension from Santa Bårbara to Point F.

As a result, complete extension of this sub-section except from Point H+2.5 km to I+0.35 km is planned for improvement by partially adjusting the realignment and widening the existing road.

(2) Alternative Ideas

As alternative ideas proposed for the section from Point H+2.5 to I+0.35, a "Tunnel", a "Semi-Tunnel" and a "Divided Two-lane Road" (consisting of one-lane tunnel and the other lane by steg), in addition to the fundamental idea of the "Widening of Existing Road" are considered first.

The idea of widening the existing road by utilizing earth works only, the idea of "Semi-Tunnel" and the idea of "Divided Two-lane Road" are applied to the route of the existing road so as to partially incorporate the existing road.
In these cases, a change in the horizontal alignment should be considered only at the place where the road crosses the Cajones River to eliminate the existing small curve. (See, Fig. 4.3-1.)

On the contrary, the "Tunnel" would be constructed along a new alignment. Considering driver's sight distance in a tunnel without any lighting facilities, not only must the horizontal alignment of the tunnel be straight but also the vertical gradient must be constant.
In the case where two tunnels and a bridge are planned for this sub-section, as shown in Figs. 4.3-1 and 4.3-2, the abutment of the bridge on the Cajones River can be held at a practical height of about 6 m and a vertical gradient of 4 %, if this is the case it is said to be within the allowable range from the viewpoint of the quantity of traffic exhaust fumes.

(3) Comparison of Alternative Ideas

Comparison of the alternative ideas has been performed with following items borne in mind;

- Economic effect
- Trafficability
- Ease of construction
- Disaster prevention

Consequently the result of the comparison (as shown in Table 4.3-2) shows that the idea of adopting a tunnel is the optimum choice among the alternatives. In particular, this choke is preferable for disaster prevention.

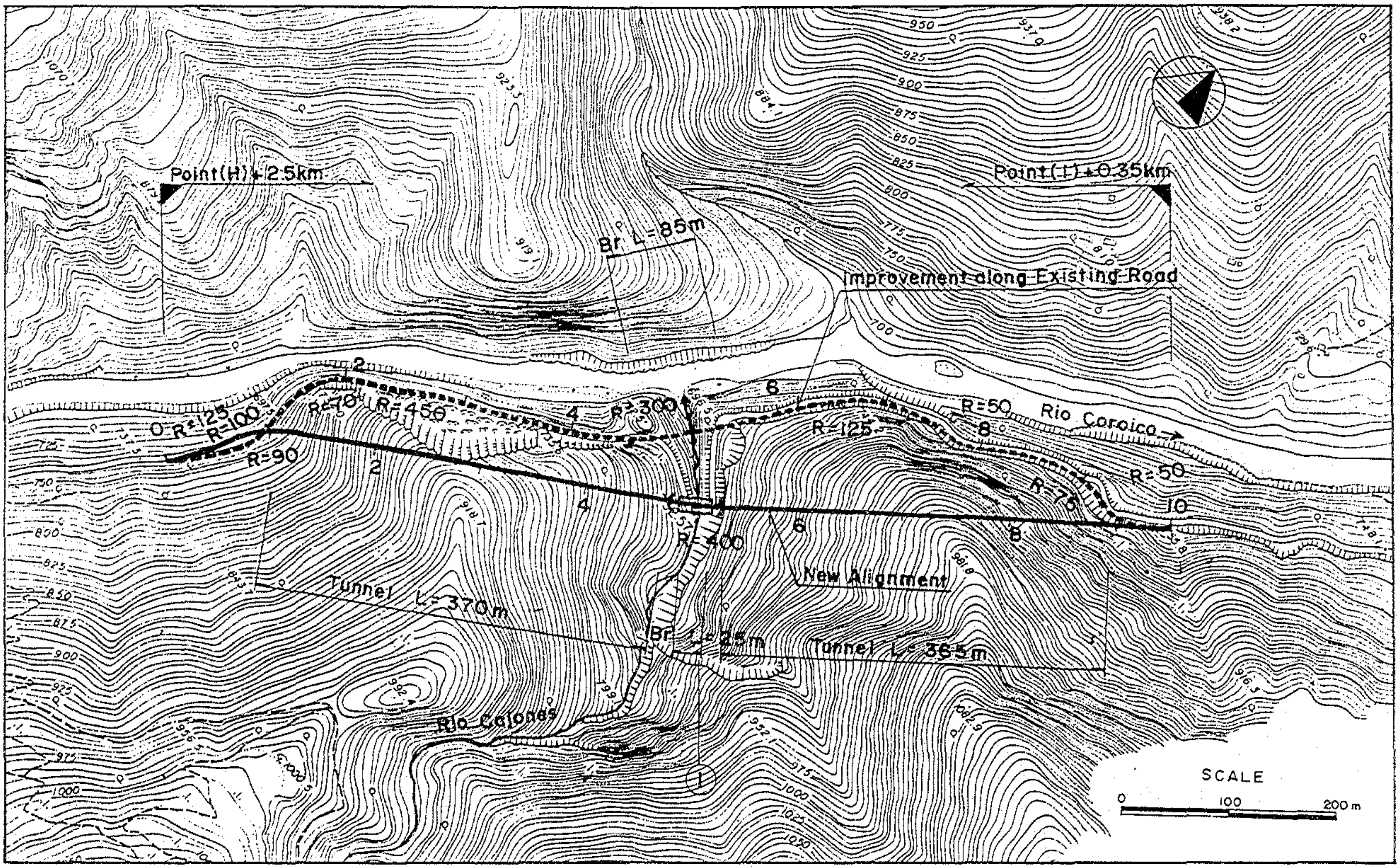


Fig. 4.3-1 Comparison of Alternatives between Point (H)+2.5 km and Point (I)+0.35 km

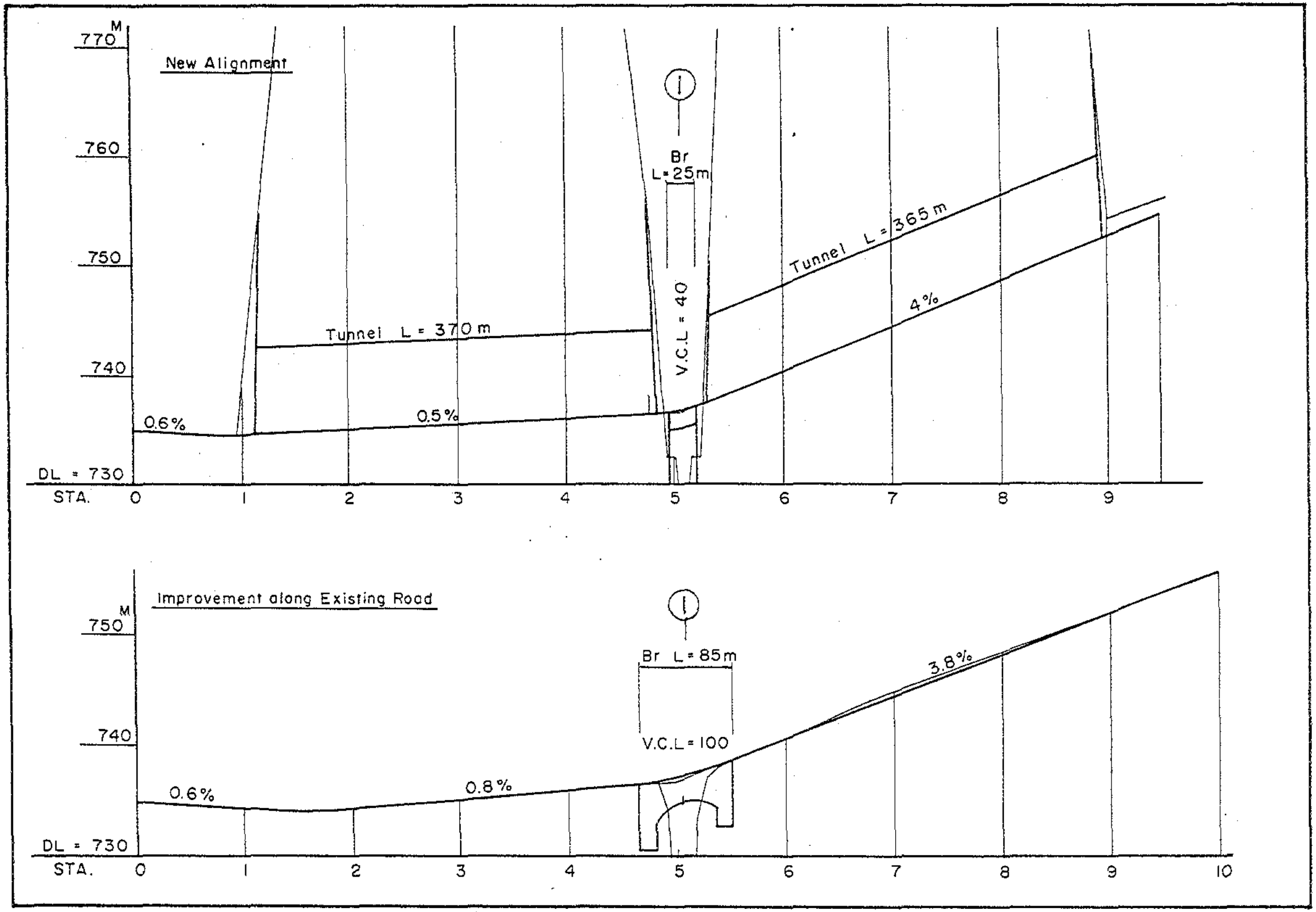
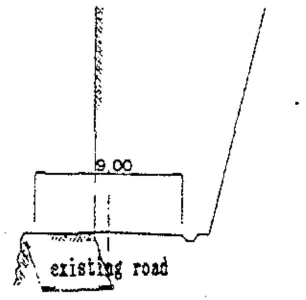
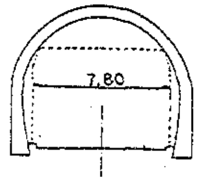
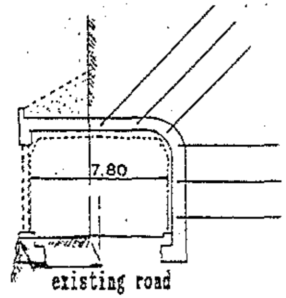
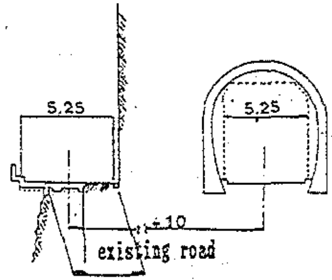


Fig. 4.3-2 Profile of Comparison for the Alternative Ideas between Point (H) + 2.5^{km} and (I) + 0.35^{km}

Table 4.3-2 Comparison of alternatives between Point (H) + 2.5 km and Point (I) + 0.35 km

		Widening of Existing Road (earth-work)	New Alignment (tunnel)	Improvement of Existing Road (semi-tunnel)	Dual Carriage Way Road (tunnel + steg)
Road Length		1,000 m	950 m	1,000 m	southward = 1,000 m : northward = 950 m
Alignment	Horizontal Vertical	Curvature = 294 deg./km : Nos. of Curves = 7 Steepest Grade = 3.8 % , length = 500 m Average Vertical Grade = 2.3 %	Curvature = 54 deg./km : Nos. of Curves = 3 Steepest Grade = 4.0 %, length = 445 m Average Vertical Grade = 2.2 %	Curvature = 294 deg./km : Nos. of Curves = 7 Steepest Grade = 3.8 %, length = 500 m Average Vertical Grade = 2.3 %	Curvature = 179 deg./km : Nos. of Curves = 5 Steepest Grade = 4.0 %, length = 445 m Average Vertical Grade = 2.3 %
Croquis of Each Idea					
Approx. Const. Cost		12,511,000 US$	4,980,000 US$	5,792,000 US$	5,521,000 US$
Comparison	Const. Cost	Highest (Much rock excavation would be required.)	Lowest	16 % higher than the Lowest	10 % higher than the Lowest
	H. Alignment	Worse	Very Good (H. and V. alignment : straight.)	Worse	southward = Worse : northward = Very Good
	Construction	Possible but difficult (considering traffic)	No problem	Possible but difficult (considering traffic)	No problem but requires long time
	Bridge	Required length = 85 m	Required length = 25 m	Required length = 85 m	Requires two narrow bridges (85 m and 25 m)
	Traffic Safety	Possible disaster such as rock fall	Safe	Damage to road by disaster is avoidable	southward = Dangerous : northward = Safe Safer than others due to dual carriage way
Evaluation and Recommendation		"New Alignment with tunnels" is sellected as the most recommendable idea. This idea has more advantages in construction cost, construction easiness, traffic safety, road alignment and total road length.			

4.3.4 From Point K to Caranavi

(1) Existing Conditions and chosen Method of Improvement

Topographical features around the existing road along this section are said to be moderately gentle except in the sub-section from Point L+5.1 Km to M+1.3 Km.

Between Point L+5.1 km and M+1.3 km, the road runs along a steep slope with many pleats downward to the Coroico River.

The horizontal alignment of the existing road is satisfactory as a whole, complying with the geometric criteria established in this Study, except in those limited locations having a large curvature.
With respect to the sub-section from Point L+5.1 km to M+1.3 km mentioned above, as it is impossible to find a better alternative route, the road should be improved along the existing road alignment.

The vertical alignment of the existing road is said to be satisfactory too except for the sub-section from Point K+2.5 km to L+2.1 km, where the road has an excessively steep gradient of 8 % together with an unnecessary up and down in spite of the vicinity topography being moderate, as described in Section 2.3.2 (3), 1). Moreover, there are three zones afflicted by landslides in the past, and they are still subject to danger.

Therefore, in order to decrease the vertical gradient of the road and also to avoid passing potential landslide areas, a new alignment for the sub-section between Point K+2.5 km and L+2.1 km has been presented and evaluated. The result is shown in the following section.

Widening the existing road is recommended as a means of improvement for the whole extension except this sub-section.
Incidentally, the existing road from Point M+1.3 km to Caranavi has already been upgraded to a gravel road with a width of about 9 m.

(2) Alternative Alignment from Point K+2.5 km to L+2.1 km

The horizontal and vertical alignment of the existing road and a alternative route are shown in Fig. 4.3-4.
The new alignment passes at a safe distance from potential disaster places, and its vertical gradient is less than 5.5 %. Furthermore, the total length of the road along the new alignment is 600 m shorter than that of the existing road.

Table 4.3-3 shows a comparison of two proposals for road improvement, construction along the new alignment and widening of the existing road to the required width.
In the latter case, the vertical gradient of the existing road will not be improved and will have a maximum of 8 % as it does now.

Evaluating all the items on the table, it is apparent that construction along the new alignment is indeed preferable and should be recommended as the improvement method.

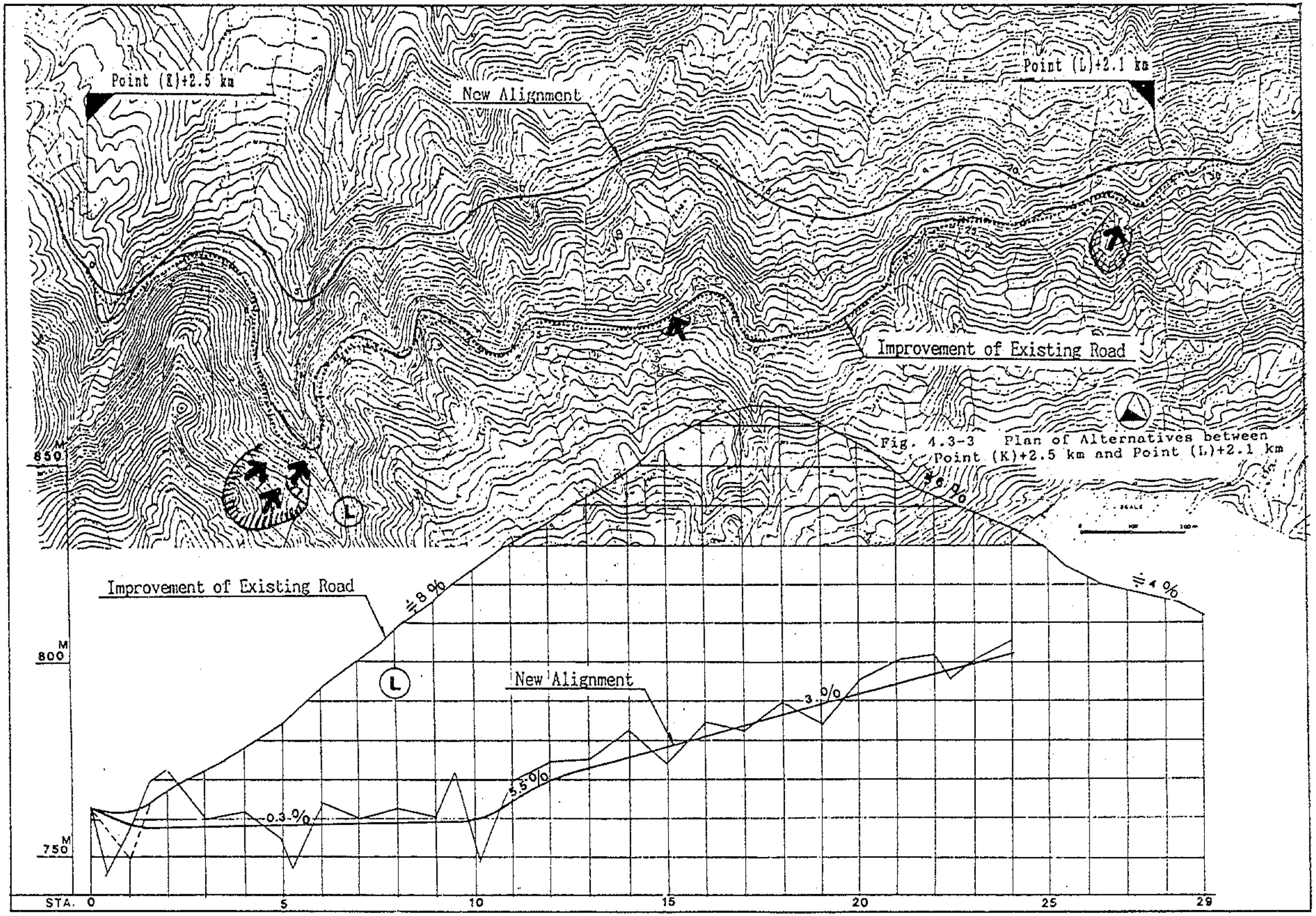


Fig. 4.3-3 Plan of Alternatives between Point (K)+2.5 km and Point (L)+2.1 km

Fig. 4.3-4 Profile of Alternatives between Point (K)+2.5 km and Point (L)+2.1 km

Table 4.3-3 Comparison of Alternatives at Point (L), Santa Ana

		Improvement of Existing Road	New Alignment
Road Length		2,900 m	2,300 m
Alignment	Horizontal	Curvature = 539 deg./km : Nos. of Curves = 19	Curvature = 426 deg./km : Nos. of Curves = 15
	Vertical	Steepest Grade = 8.0 %, length = 500 m Length with more than 5 % Grade = 1,900 m with less than 5 % Grade = 1,000 m Average Vertical Grade = 5.9 %	Steepest Grade = 5.5 %, length = 190 m Length with more than 5 % Grade = 200 m with less than 5 % Grade = 2,100 m Average Vertical Grade = 1.9 %
Rough Const. Cost		3,183,000 US$	2,776,000 US$
Comparison	Const. Cost	14 % higher than "New Alignment"	Cheaper than "Improvement of Existing Road"
	V. Alignment	500 m with 8 % does not comply with "Norma"	Very Good
	Construction	Worse	Better as is a new construction
	Disaster	Worse. 4 disaster spots exist along road.	Better. Detouring problematic disaster zone.
Evaluation and Recommendation		"New Alignment" is recommendable because it is better in all comparison items.	

4.3.5 From Caranavi to Point Q (Carrasco)

(1) Existing Road Conditions and Method of Improvement

This section is located alongside the river. The alignment and width of the existing road are relatively satisfactory compared with other sections covered along the Study Road, and there is less of a problem for widening the existing road.

However, two locations should be noted as being problem spots as described in Section 2.3.2,(3) and (4). One is at around Point O+1.8 km with an extension of 250 m, where the vertical gradient of the existing road is nearly 11 %. It seems to be necessary to construct a detour along a new alignment instead of the idea of widening the existing road in order to improve the vertical alignment of the existing road.

The other location is near Point P+1.7 km, and there the existing road width is 3.2 m. However this condition continues only 150 m along the existing road. The surrounding geography is very hard, that is to say, there exists almost vertical cliffs of hard rock on both the hillsides and valley sides of the existing road. Meanwhile, it is difficult to find an appropriate and advantageous alternative alignment for a detour along this extension due to the difficult geography.
Considering this situation and the fact that the extension of this sub-section is not so long, widening of the existing road is planned and recommended as being the most suitable method of improvement for this sub-section.

(2) Alternative Alignment around Point O+ 1.8 km

In order to improve the vertical alignment in this sub-section, improvement along the existing road and construction of a detour with a new alignment have been compared and evaluated.

These are shown in Figs. 4.3-5, 4.3-6 and Table 4.3-4.
By improving the existing road, the vertical gradient will possibly be less than 7 % but it requires a considerable amount of earthwork.

The new alignment passes through an area which has been subject to landslides in the past, however, the area appears to have stabilized and it is not considered necessary to pay any special attention to this other than installing ordinary drainage facilities. Durintg the course of site reconnaissance in the vicinity, it was observed that the existing road, (which had been constructed after the occurrence of a landslide) possesses subsoil and surface water from upper side and it makes the ground unstable, especially in the lower part of the existing road where the new alignment passes.

As a result of the comparison, it has been confirmed that the new construction of a detour along the new alignment is much more advantageous than improvement along the existing road from the various viewpoint as indicated in Table 4.3-4.

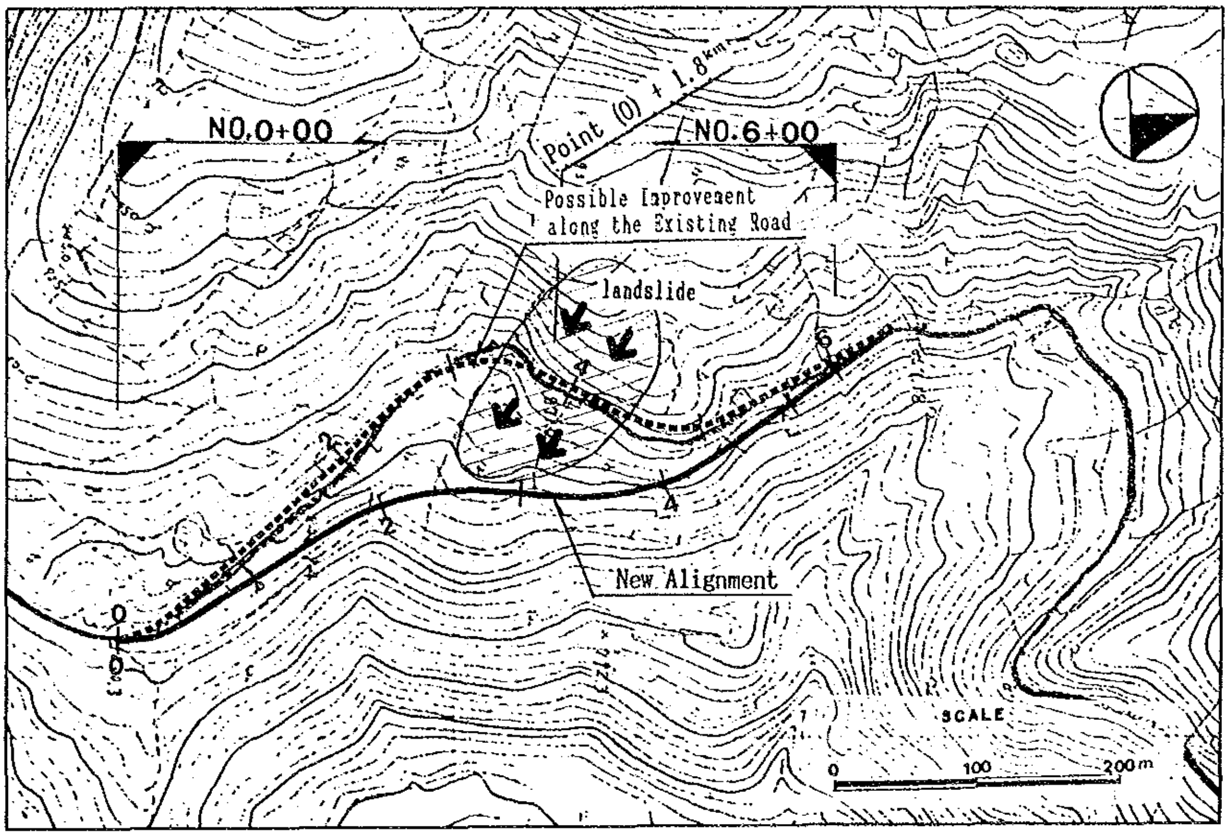


Fig.4.3-5 Plan of Comparison for the Alternative Ideas around Point (0) + 1.8^{km}

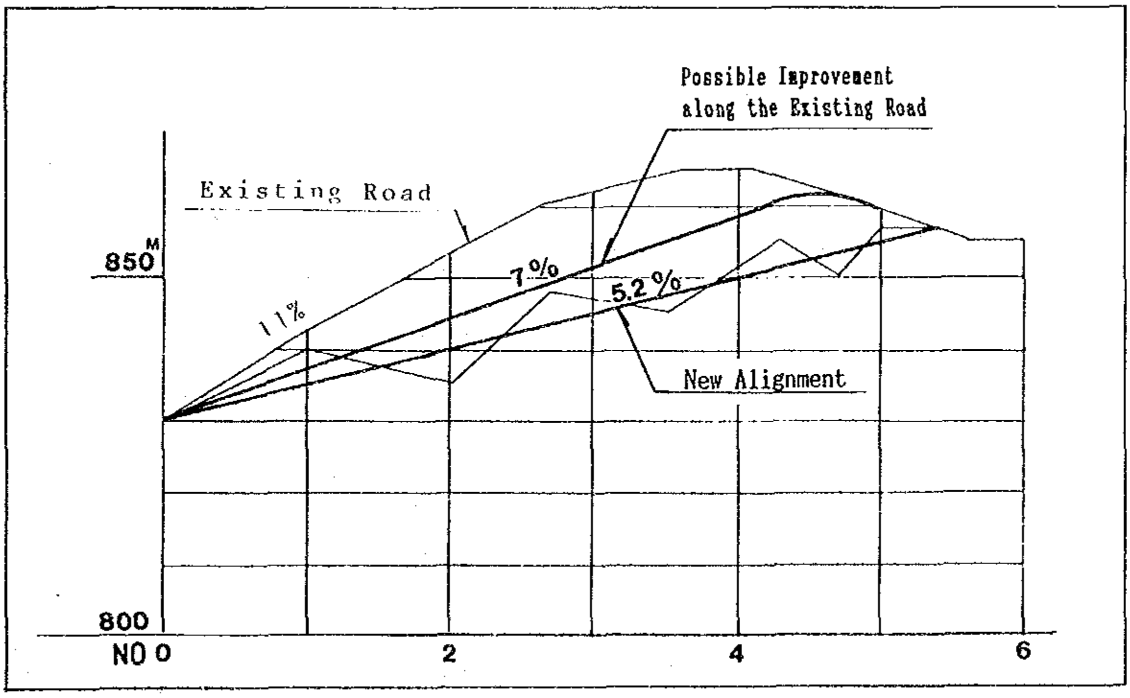


Fig.4.3-6 Profile of Comparison for the Alternative Ideas around Point (0) + 1.8^{km}

Table 4.3-4 Comparison of Alternatives at Point (O) + 1.8 km

		Improvement of Existing Road	New Alignment
Road Length		600 m	540 m
Alignment	Horizontal	Curvature = 653 deg./km : Nos. of Curves = 3	Curvature = 600 deg./km : Nos. of Curves = 3
	Vertical	Steepest Grade = 7.0 %, length = 460 m Length with more than 5 % Grade = 560 m with less than 5 % Grade = 40 m Average Vertical Grade = 6.6 %	Steepest Grade = 5.2 %, length = 540 m Length with more than 5 % Grade = 540 m with less than 5 % Grade = 0 m Average Vertical Grade = 5.2 %
Rough Const. Cost		1,027,000 US$	495,000 US$
Comparison	Const. Cost	More than double of "New Alignment". Improvement of the existing road's V. alignment (11%) requires much cost.	Cheaper than "Improvement of Existing Road"
	V. Alignment	Worse	Better
	Construction	Worse	Better as is a new construction
	Disaster	No problem	Passing land slide remains zone, but no problem. It has already been stabilized.
Evaluation and Recommendation		"New Alignment" is sellected as a better alternative, mainly due to less construction cost.	

4.3.6 From Point Q (Carrasco) to Point S

(1) Condition of the Existing Road

General topography from Point Q to Point S is relatively moderate. Widening of the existing road here, might not be so difficult. However, this section is characterized by a steep vertical gradient on the existing road. That is from Point Q with an altitude of 830 m above sea level, to Point S whose altitude is 1,340 m, the road climbs up successively with an average vertical gradient of 6.3 %.

Surveying around the vicinity, it is apparently impossible to find a new route for the road, for which the problem of the existing road described above could be drastically eliminated. Therefore, improvement of the existing road will basically be the most advantageous idea for this section.

(2) Alternative alignment from Point Q+ 5 km

Supposing that the existing road was to be improved, the 1.2 Km long sub-section from Point Q+5 km would be critical in attempting to condition to comply with the design criteria for the desired vertical gradient of the improved road. For this reason, an alternative idea for constraction a new alignment, as shown in Figs. 4.3-7 and 4.3-8, has been proposed and compared with the case for improveing of the existing road for this sub-section.

As shown in Table 4.3-5, it is confirmed that the construction of a road along the new alignment is more favorable not only because of its geometrical features but also from an economical viewpoint. Hence, it is recommended that the road along the new alignment between Point Q+ 5 km and Point Q+ 6.2 km be developed and to improve the existing road in the remaining sections from Point Q to Point S.

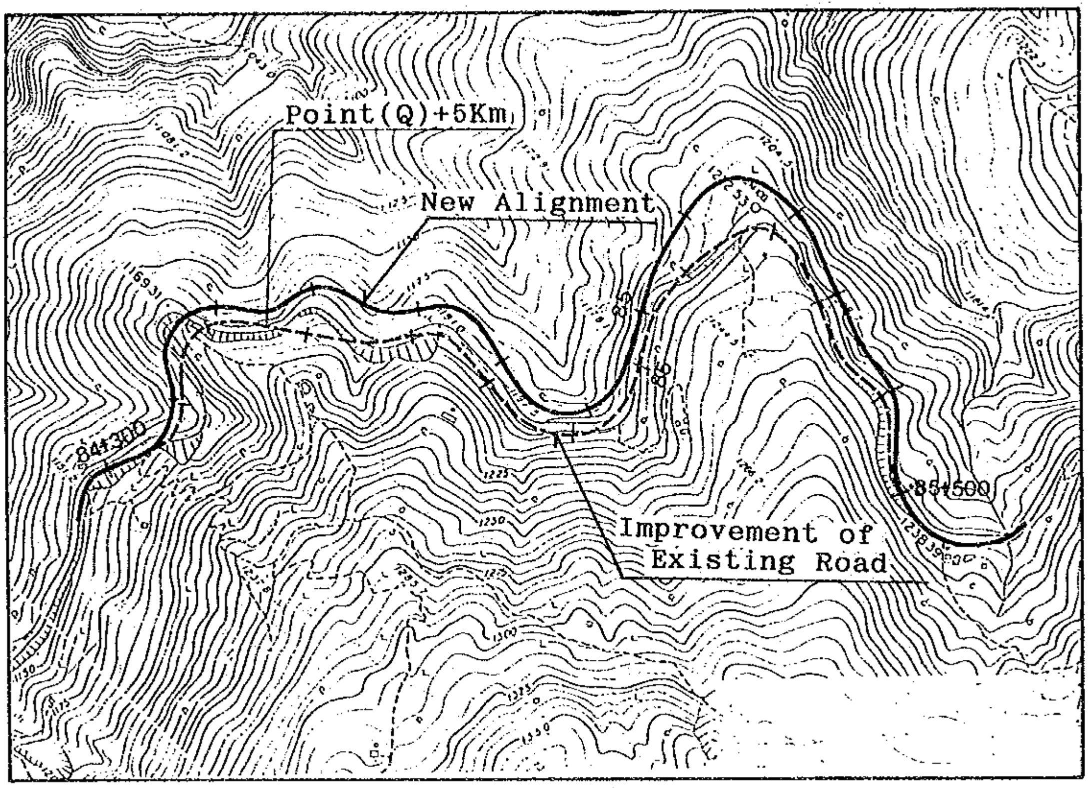


Fig. 4.3-7 Plan of Alternatives at Point(Q)+5Km

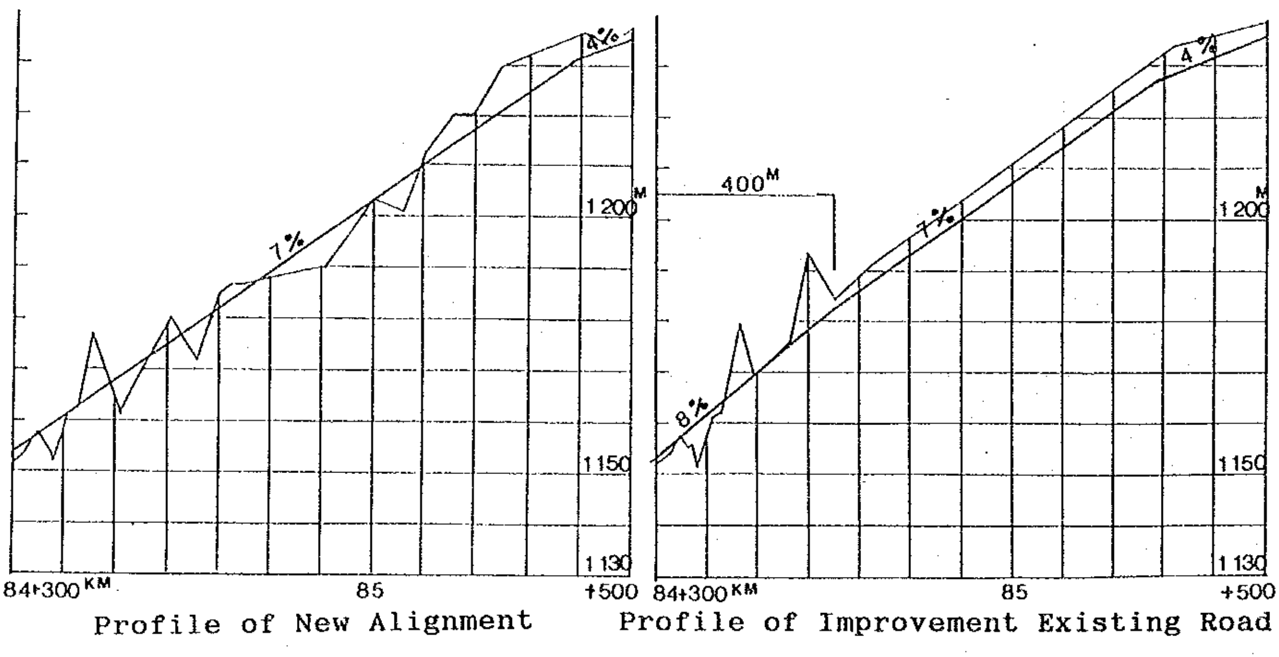


Fig. 4.3-8 Profile of Alternatives at Point(Q)+5Km

Table 4.3-5 Comparison of Alternatives at Point (Q)+5Km

		Improvement of Existing Road	New Alignment
Road Length		1,200 m	1,200 m
Alignment	Horizontal	Curvature = 573 deg./km : Nos. of Curves = 9	Curvature = 563 deg./km : Nos. of Curves = 11
	Vertical	Steepest Grade = 8.0 %, length = 400 m Steep Grade = 7.0 % Length = 500 m Average Vertical Grade = 6.8 %	Steepest Grade = 7.0 %, length = 1,100 m Average Vertical Grade = 6.8 %
Construction Cost		US$ 1,374,000-	US$ 968,000-
Compa-rison	Const. Cost	42 % higher than "New Alignment"	Cheaper than "Improvement of Existing Road"
	V. Alignment	400 m with 8 %, 500 m with 7 % and 200 m with 4 %	1,100 m with 7 % and 100 m with 4 %
	Construction	Worse because of control for existing traffic	good
Evaluation and Recommendation		Although there is not so much difference between 2 cases in horizontal alignment, "New Alignment" is superior in vertical alignment and economic aspects. So, "New Alignment" is recommendable.	

4.3.7 From Point S to Point V

(1) Condition of the Existing Road and Alternative Routes

The road must cross over two mountain ranges in this section. The lowest point of the first and the second mountain range in the vicinity of the existing road has an altitude of 1,500 m and 1,235 m, respectively.

From Point S, the existing road climbs and decends repeatedly with a considerably steep vertical gradient of more than 8 %. It passes over the first mountain range at a point with an altitude of 1,543 m and continues through to the second mountain range.
The road crosses the second mountain range at Point V, which has the lowest altitude(1,235 m) in the area.

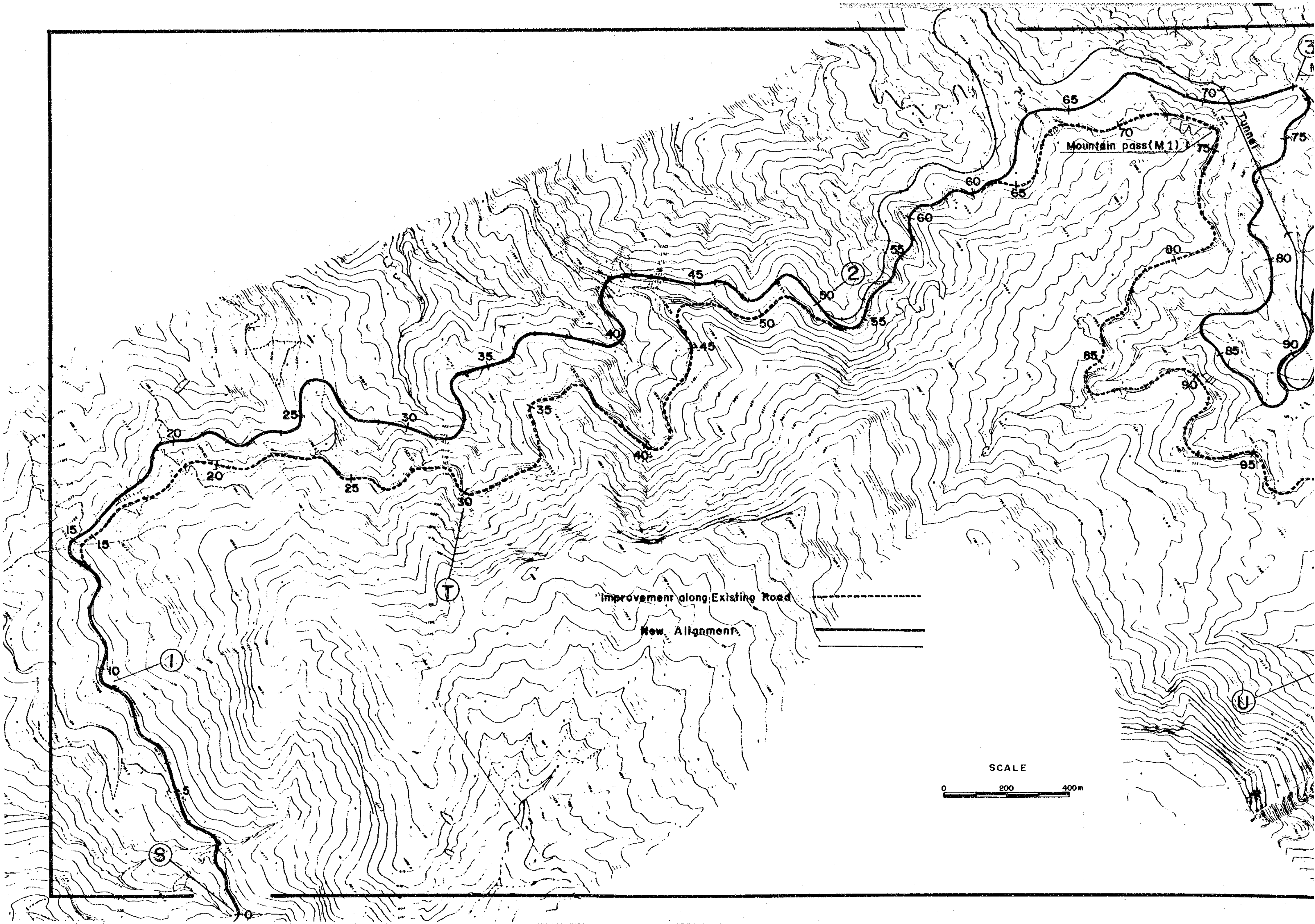
Although Point V seems to be the best place to go over the second mountain range, both the horizontal and vertical alignments of the existing road in this section,(especially from Point T+7.4 km to Point U+0.7 km) are poor already. Unfortunately it is apparently impossible to improve the existing road for this sub-section so as to make its geometrical features comply with the design criteria by using practical method, because of the precipitous surrounding geography.

Furthermore, four potential disaster spots can be identified along the existing road and the method of how to cope with them, so as to make the road avoid them or to construct some other countermeasures, will be another problem.

Therefore, various alternative routes for a detour are presented for this section and are shown in Fig. 4.3-9.

(2) Presentation of Alternative Routes

As previously described, the existing road passes through rugged terrain, involving an unsatisfactory vertical alignment. There are also several disaster zones in the same region. Hence, a new alignment is to be established and studied here.


Mountain pass(M1)
Tunnel
Improvement along Existing Road
New Alignment
SCALE
0
200
400m

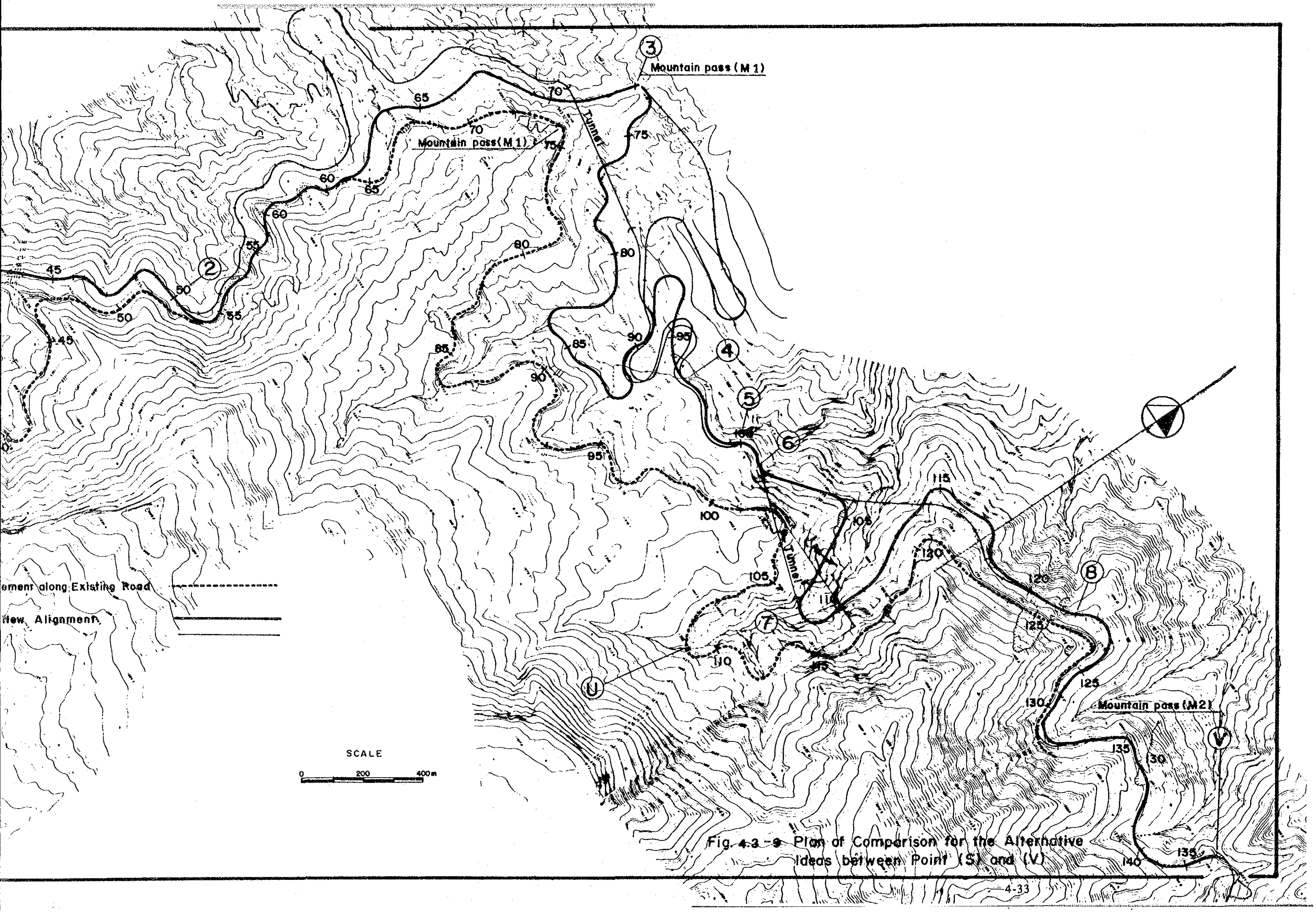


Fig. 4.3 -9 Plan of Comparison for the Alternative Ideas between Point (S) and (V)

a) Improvement of the Existing Road

The first case to be studied is the improvement of the existing road itself, to satisfy geometrical features such as, horizontal and vertical alignment and road width.

b) New Alignment

Based on the site reconnaissance results and the topographic survey as shown in Fig. 4.3-10, some of them are assumed the construction of a tunnel or a fairly big bridge.

Though the interrelation between each route seems to be a little complicated on the Figure, the following schematic diagram should be helpful to make it clearer.

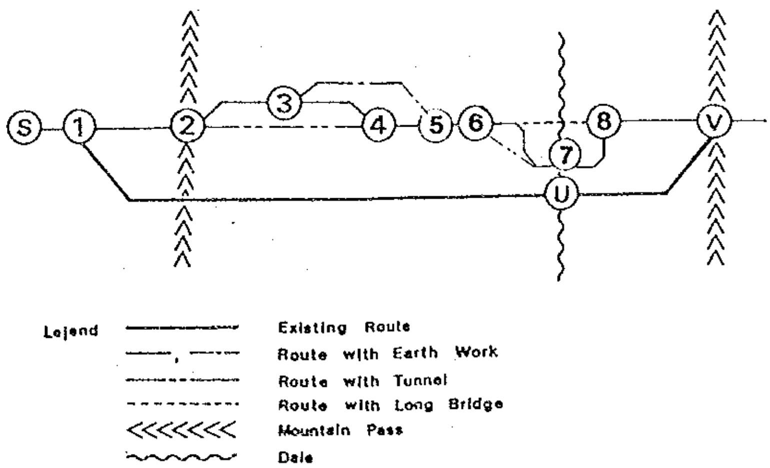


Fig. 4.3-10 Schematic Diagram of Alternative Routes

(3) Improvement of the Existing Road

In this case, the work required to improvement would principally be earthwork and there would be no place where the construction of tunnels or bridges would be warranted for the required purpose.
The result of the study, shown in Fig. 4.3-11 and Table 4.3-10, makes it clear that it would be impossible to improve the existing road in order to comply with the target limit for vertical gradient, i.e., 8 %.

(4) Comparison of Alternative Routes

To find the most satisfactory route from Point S to Point V, it is necessary to firstly compare partially each pair of competitive routes, as shown in Fig. 4.3-10.

1) From Point 3 to Point 5

The routes 3-4-5 and 3-5 are defined as Case 1 and Case 2, and both of these cases are compared with each other by the volume of earthworks required. Road length and each volume for embankment and cutting are summarized in Table 4.3-6.

Table 4.3-6 Volume of Earthwork from Point 3 to 5

	Case 1	Case 2
Road Length	2,800 m	2,800 m
Embankment	59,000 m^3	275,000 m^3
Cutting	194,000 m^3	452,000 m^3

Case 1 is apparently more economical than the other, with a lower volume for earthworks, moreover, this route has fewer hair-pin curves. Therefore, Case 1 (route 3-4-5) is considered to be the optimum option.

2) From Point 2 to Point 4

The routes 2-3-4 and 2-4 are defined as Case 3 and Case 4 which indicate a "proposal utilizing earthworks only" and "proposal utilizing earthworks and tunneling".

Road length and each construction cost is summarized in Table 4.3-7.

Table 4.3-7 Comparison of Construction Cost from Point 2 to 4

	Case 3	Case 4
Road Length	4,600 m	4,140 m (including 500 m of tunnel)
Approx. Const. Cost	US$ 7,705,000	US$9,774,000

Although road length could be considerably shortened in Case 4 with a 500m tunnel, the estimated construction cost is more than 25 % higher than that of Case 3.

Consequently Case 3 (route 2-3-4), which would require earthworks only, is considered to be a better selection.

3) From Point 5 to Point 7

Case 5 and Case 6 here represent the route 5-6-7 and route 5-7. The latter case proposes to construct a 450 m long tunnel.

Table 4.3-8 Comparison of Case 5 and Case 6 between Point 5 and Point 7

	Case 5	Case 6
Road Length	1,500 m	1,280 m (including 450 m of tunnel)
Horizontal Curvature	625 deg./km	360 deg./km
Vertical Gradient	6.5 % (max.) 5.6 % (ave.)	7 % (max.) 6.4 % (ave.) 5 % (in tunnel)
Approx. Const. Cost	4,181,000 US$	5,341,000 US$

The results from the comparative study are summarized in Table 4.3-8 above.

The vertical gradient of the road in Case 5 would be relatively moderate, however, the horizontal alignment would surely be worse due to the rugged terrain in the region. It has been confirmed in this comparative study that a con siderable quantity of structures such as retaining walls and slope protection would be required for this case. The value of horizontal curvature in the above Table reflects this condition to some extent.

On the other hand, noteworthy weak points in Case 6 are the vertical gradient in the tunnel and the estimated construction cost. It appears to be difficult to decrease the vertical gradient of the tunnel to less than 5 % because that for the sections before and after tunnel would already be sufficiently steep enough.
Considering a tunnel length of 450 m, the tunnel with vertical gradient of 5 % ,without any ventilation equip-

ment, might possibly cause problems for the traffic in air pollution or restricted visibility.
The estimated construction cost for this case is more than 25 % higher than that for Case 5.

Although Case 5 would involve unfavorable problems, it is considered that Case 5 (route 5-6-7) would be a more suitabl idea paying attention to less construction cost in this case.

4) From Point 6 to Point 8

There exists a deep concave valley between Point 6 and Point 8. Incidentally, Point 7 is located at the innermost position of the valley.
The alignments from Point 6 to Point 8 via Point 7, and connecting Point 6 directly with Point 8 by a bridge are named here Case 7 and Case 8, respectively.

The result of comparing of these alternative cases is summarized in Table 4.3-9.

Table 4.3-9 Comparison of Alternatives (Point 6 to 8)

	Case 7	Case 8
Road Length	1,800 m	960 m (including 250 m of a bridge)
Approx. Const. Cost	US$3,015,000	US$11,044,000

As there is an indisputable difference in construction cost between these two cases due to the fairly higher construction cost for the bridge, Case 7 (route 6-7-8) is selected as a better alternative in this sub-section.

(5) Result of Comparative Study on Improvement of the Existing Road and Development of the Road along the New Alignment

Putting in order the results described in the previous section, the most suitable route for recommendation from Point 1 to Point 8 among the various alternatives is route

1-2-3-4-5-6-7-8. The idea to construct a new detour along this route is hereafter called the "New Alignment".
The "New Alignment", as a result, would have no tunnel nor bridge. Only one potential disaster zone which is located along the existing road may influence the New Alignment, however, it could easily be solved by appropriate preventive countermeasures.

A profile of the comparison between the New Alignment and the Improvement of the Existing Road are shown in Fig. 4.3-11 and Table 4.3-10.

Only by observing the descriptions on construction cost and vertical alignment in Table 4.3-10, it is apparent that the New Alignment would be more advantageous to implement for this section from Point S to Point V.

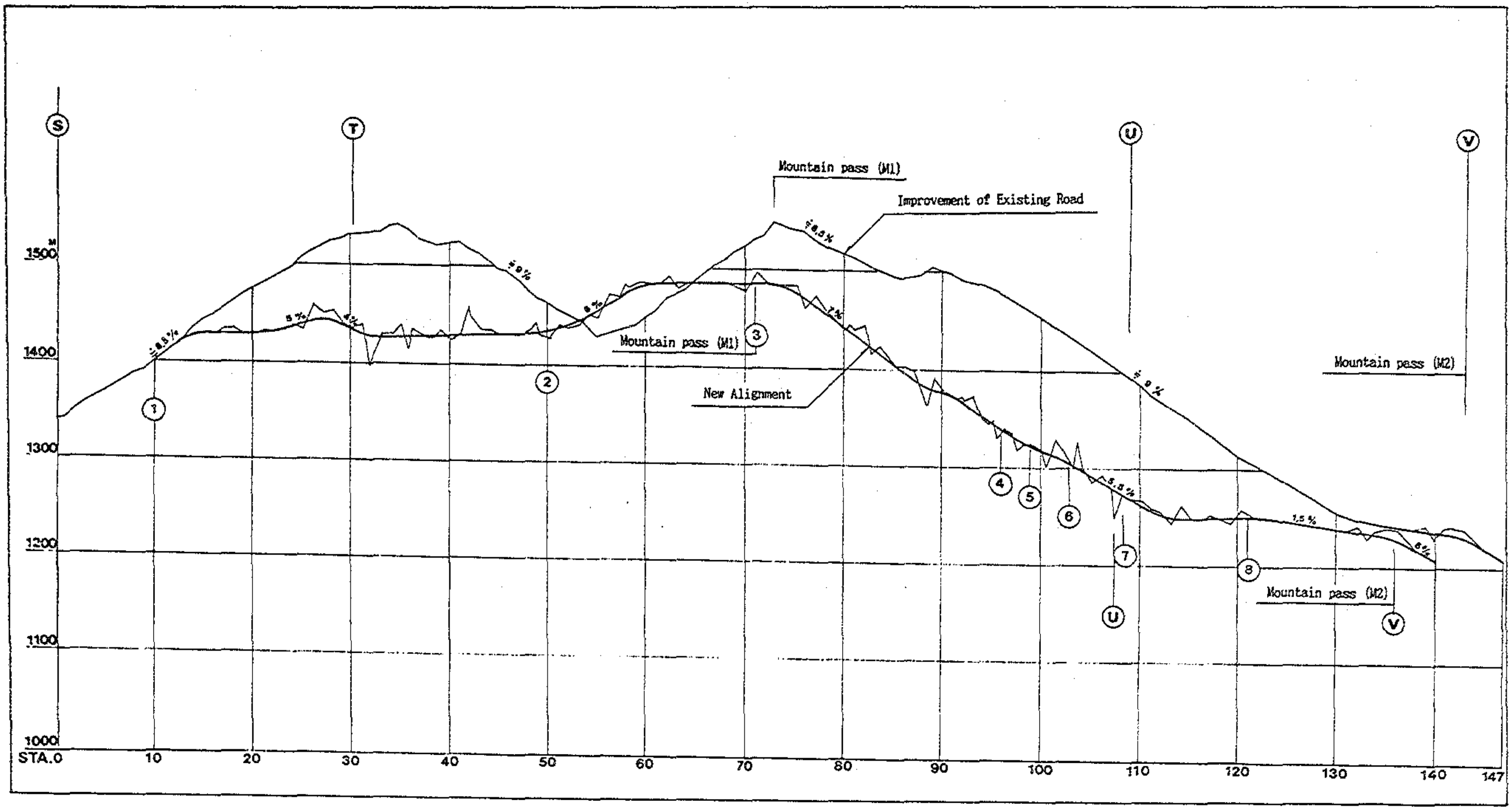


Fig.4.3-11 Plofile of Comparison for the Alternative Ideas between Point (S) and (V)

Table 4.3-10 Comparison of Alternatives between Point (S) and Point (V)

		Improvement of Existing Road	New Alignment
Road Length		14,700 m	14,000 m
Alignment	Horizontal	Curvature = 578 deg./km : Nos. of Curves = 90	Curvature = 548 deg./km : Nos. of Curves = 85
	Vertical	Steepest Grade = 9.0 %, length = 200+300 m Length with more than 5 % Grade = 9,500 m with less than 5 % Grade = 5,200 m Average Vertical Grade = 5.0 %	Steepest Grade = 7.0 %, length = 1,500 m Length with more than 5 % Grade = 5,450 m with less than 5 % Grade = 8,550 m Average Vertical Grade = 2.9 %
Construction Cost		US$ 24,994,000-	US$ 18,077,000-
Comparison	Const. Cost	38 % higher than "New Alignment" Widening at Point (U), where rocky cliff continues more than 1 km, costs very much.	Cheaper than "Improvement of Existing Road"
	V. Alignment	Almost impossible to comply with 8 % (Norma).	Very Good in average except 7 % with 1500 m
	H. Alignment	A little worse as a whole	A little better as a whole but it has two hair-pin curves.
	Construction	Worse	Better as is a new construction
	Disaster	Many disaster spots exist.	Impossible to detour a debris flow spot, but no problem with adequate countermeasure.
Evaluation and Recommendation		"New Alignment" has much advantage in various meaning against "Improvement of Existing Road", so construction of new road is recommended.	

4.3.8 From Point V to Bella Vista

(1) Condition of Existing Road

This section can be divided into two sub-sections, from Point V to Point W and From Point W to Bella Vista. The existing road in the former sub-section lies along sloping terrain with an average land surface gradient of 32 degrees, and with numerous small dales. The horizontal alignment of the road zigzags to avoid these dales.

Topographical and geographical features in the latter sub-section are fairly moderate with an average land surface gradient of 20 degrees and can be said to be different from the other sections. However, it should be noted that this sub-section consists of weak soil strata and is subject to heavy rainfall as much as 2500 mm annually and up to 160 mm maximum daily. This causes the area in the vicinity of this sub-section to be a potential landslide zone.

The vertical alignment of the existing road in this section descends gently from Point V towards Bella Vista.

(2) Improvement of the Existing Road

As a conclusion, improvement of the geometric characteristics of the existing road, such as a partial revision of the horizontal alignment and increasing the road width, would be the best improvement plan for this section. The reasons for this are; firstly, it is almost impossible to find a more favorable route for a detour in the vicinity, and secondly, as no active landslide has been found between Point W and Bella Vista during the site reconnaissance this time, it is considered that the improved road, along the existing road alignment, with some appropriate preventive countermeasures, could avoid critical damage from landslide.

4.4 Study on the Bridges

A study of bridge construction was carried out according to the flow chart given in Fig. 4.4-1. The study in this section aims to identify the places requiring bridge and to determine the exact location of its construction there.

Since the study on bridges is closely related to that on the geometric design of the road, both studies were carried out simultaneously.

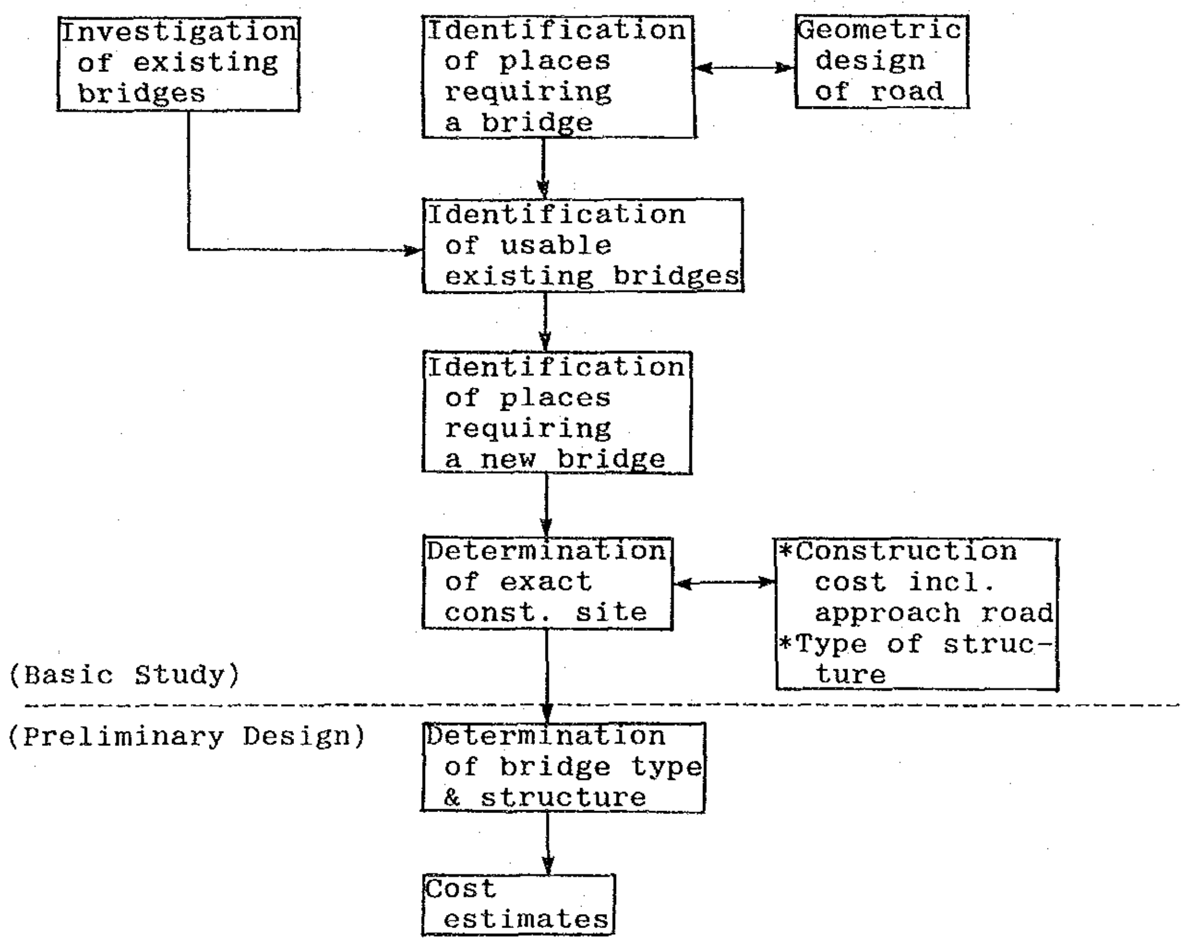


Fig. 4.4-1 Flow chart for the Study on Bridges

4.4.1 Identification of Places Requiring a New Bridge

(1) Places requiring a bridge

The following fourteen places are found to require a new bridge by the reconnaissance survey;

(1) Point A	(2) Patuni
(3) Challa	(4) Cascada
(5) Alto Choro-2	(6) Pto. Leon
(7) Cajones	(8) Chojña
(9) San Silverio	(10) Yara
(11) San Lorenzo	(12) Espiritu
(13) Carrasco	(14) Avaroa

The project road crosses the tributary of the Coroico River at all the above places except at Point A.
Since the water flow of this tributary is considerably large, a pipe or box culvert is not suitable to be served. Therefore, bridges are required at these thirteen places.
On the other hand, there is a narrow stream at Point A. However, there are numerous potential land slide zones near the Point A as shown in Table 4.4-2(1). In order to avoid the expected avalanches of soil, sand and rock, the route of the project road is desired to be shifted from the dangerous area. In this case the bridge construction is indispensable.

(2) Evaluation of the Existing Bridges

On the existing project road there are fourteen bridges as shown in Tables 2.3-9 and 2.3-10.
Among these fourteen bridges, thirteen bridges have been already constructed at places required for a bridge as mentioned in (1). The remaining one bridge (Alto Choro-1) is recommended to be changed into a concrete box culvert instead of a bridge. The reasons are as follows;

a) The bridge scale is smaller than that for the other bridges.
b) The depth of the valley is not great (2.5m to 3.7m).
c) There is no water flow during the dry season.

The other existing thirteen bridges must be evaluated from the viewpoint of whether they are usable in the future or not.
The following points are the minimum conditions required, for fully utilizing these bridges after improvement of the project road:

a) The horizontal alignment around the bridge, including the approach roads, should satisfy the geometric structure indicated in Section 4.2.

b) The effective bridge width should be 9.0m or more for a bridge length of less than 50m, and 7.3m or more for a bridge length of 50m or greater, respectively. (See Fig. 4.2-3).

c) Each bridge structures should be able to be utilized even in the future without significant deterioration.

Table 4.4-1 shows the evaluation of each existing bridge under the above conditions.

From Table 4.4-1, only the Yara bridge meets the aforementioned three conditions. Therefore, the existing Yara bridge should be utilized as it is even in the future. On the other hand, the San Lorenzo, Espiritu and Carrasco bridges can be utilized if the horizontal alignment of the approach road and the width of the bridges could be improved, otherwise, a new bridge should be constructed. The choice of whether to improve the existing bridges or to construct new bridges, depends on the cost of each operation. A cost comparison was performed for the Carrasco bridge located in a geographic area where horizontal alignment improvement and width expansion are the easiest out of the three bridges. (See, Table 4.4-2 (9))

As a result, it was confirmed that the cost for improvement would be 1.3 to 1.5 times as much as that required for new bridge construction. Since the San Lorenzo and Espiritu bridges are located in more difficult geographic areas than the Carrasco bridge , it can be assumed that the idea of horizontal alignment improvement and width expansion for both bridges would be less advantageous than the construc-