Geology in this area consists of diabase and diorite formation. The geological condition is fair except in the area between diabase and andesite group existing along Route 5.

4-2-6 *Geology along the Most Likely Route*

Geology of the selected route is shown in Fig. 4-2.3 (GS-4) in a geological map.

(1) Km. 203 + 000 — Km. 204 + 000

In this section, the proposed road will be passing through a lower or middle terrace place consisting of ricefield. The geology is then believed to consist of sand and gravel mixtures with some pebbles and boulders of different sizes, and as the thickness of sand and gravel decreases to 5.0 m or less,it is necessary to select the foundation of structure directly on the base rock.

(2) Km. 204 + 000 — Km. 204 + 400

The topography of this section shows a fault topography consisting of Kern corn and Kern bat. Thus, it is expected that geological condition in this section is poor. In cutting the western side of this section, landslide is expected. In view of the abovementioned possibility, it is recommended to lessen the cutting in this portion.

(3) Km. 204 + 400 — Km. 205 + 800

In this section, geology consists of dacitic andesite. The hardness of andesite is very strong and the interval of cracks is approximately 10-20 cm, rich in closed cracks and the geological condition in such case is considered fair or good.

However, at Km. 204 + 400, Km. 204 + 750, Km. 205 + 100, Km. 205 + 450 and Km. 205 + 600, shearing zone is believed to exist.

Geology near this portion is disturbed and it can obviously be seen that weathering and alternation is advanced. However, the relation between the new road alignment and direction of the shearing zone are crossing on a high angle approximately more than 50° and has a lesser possibility of landslide.

(4) Km. 205 + 800 — Km. 207 + 700 (Tunnel section)

This section is the tunnel area in which geotechnical investigation was conducted and the geological condition at both sides of the tunnel entrance and P wave velocity of the tunnel formation were confirmed. Based on these data, the condition of base rock at the tunnel formation is fair.

Geology of the tunnel area consists of andesite group, but near both sides of the tunnel entrance tuff breccia and andesite forms the distribution of the alternation. Facie change is remarkable. Although andesite is hard, with open cracks as a whole, tuff breccia is loose in case of weathering, broken easily and rich in porous.

At Km. 205+800, Km. 206+100 and between Km. 207+300 to 700 it is believed that shearing zone exists based on geological map. Shearing zone at Km. 205+800 and Km. 206+100 is crossing the proposed road alignment at about 90°, thus lessening the problem in the construction. On the other hand, shearing zone at Km. 207+300 is running along the proposed road alignment. If the shearing zone exists, the geological condition between Km. 207+300 to Km. 207+700 is expected to be poor.

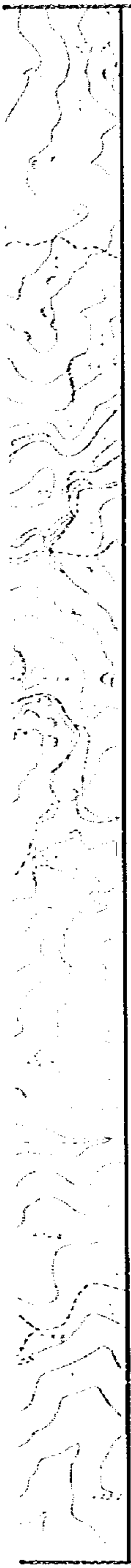
Based on seismic prospecting, low velocity zone exists between Km. 206 + 800 to Km. 207 + 100, which shows a poor geological condition.

(5) Km. 207 + 700 — Km. 208 + 350

The original topography in this section consists of a gentle slope like a landslide area; thus, it is expected that the topsoil consisting of talus is more than 10.0 m thick. These talus consist of breccia, mixed clay zone and gravel zone with thickness of 2.0m and rich in ground water. Therefore, in cutting and embankment during the construction, it is necessary to treat the ground water, and set the sandmat or drainage pipe in the embankment area.

(6) Km. 208 + 350 — Km. 210 + 000

The geology in this section consisted of granitic rock including diabase dike.
This formation is advanced in weathering, and sandy in nature.
In this case, slope surface consisting of granite and sand is eroded by surface running water during typhoon or heavy rain. Many failures occur in this section. Thus, slope protection must be carried out completely.



LEGEND

Dt	River Deposit (Include Talus, Terrece Deposit)
Lm	Limestone
Dn	Dacitic Andesite
Sh	Shale
Tf	White Tuff
Gt	Green Tuff
Tb	Tuff Breecia
An	Andesite
An(a)	Auto Clastic Andesite
Da	Diabase
Gr	Granite
	Shear Zone
	Bedding
	Joint
	Altered Zone

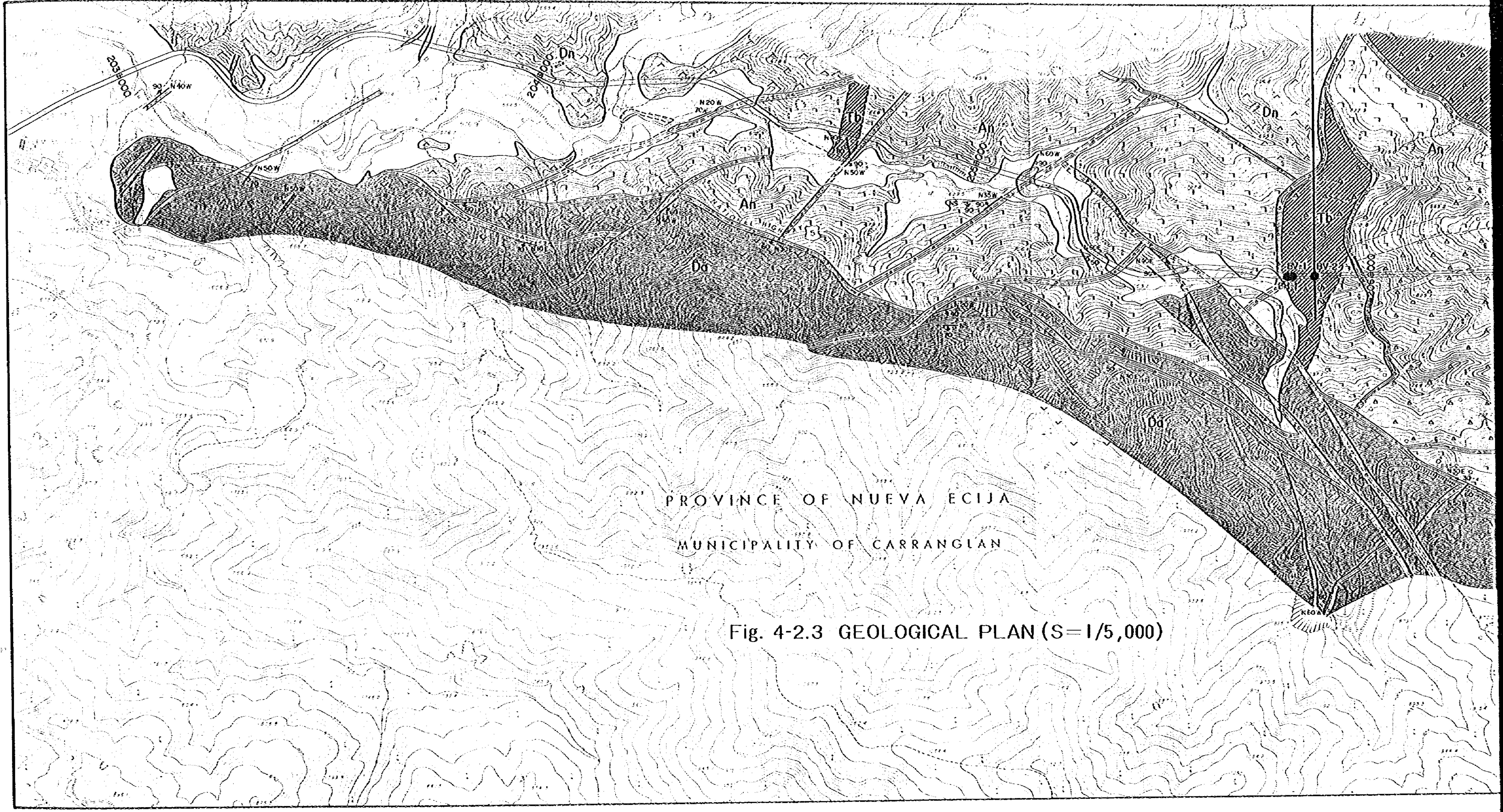
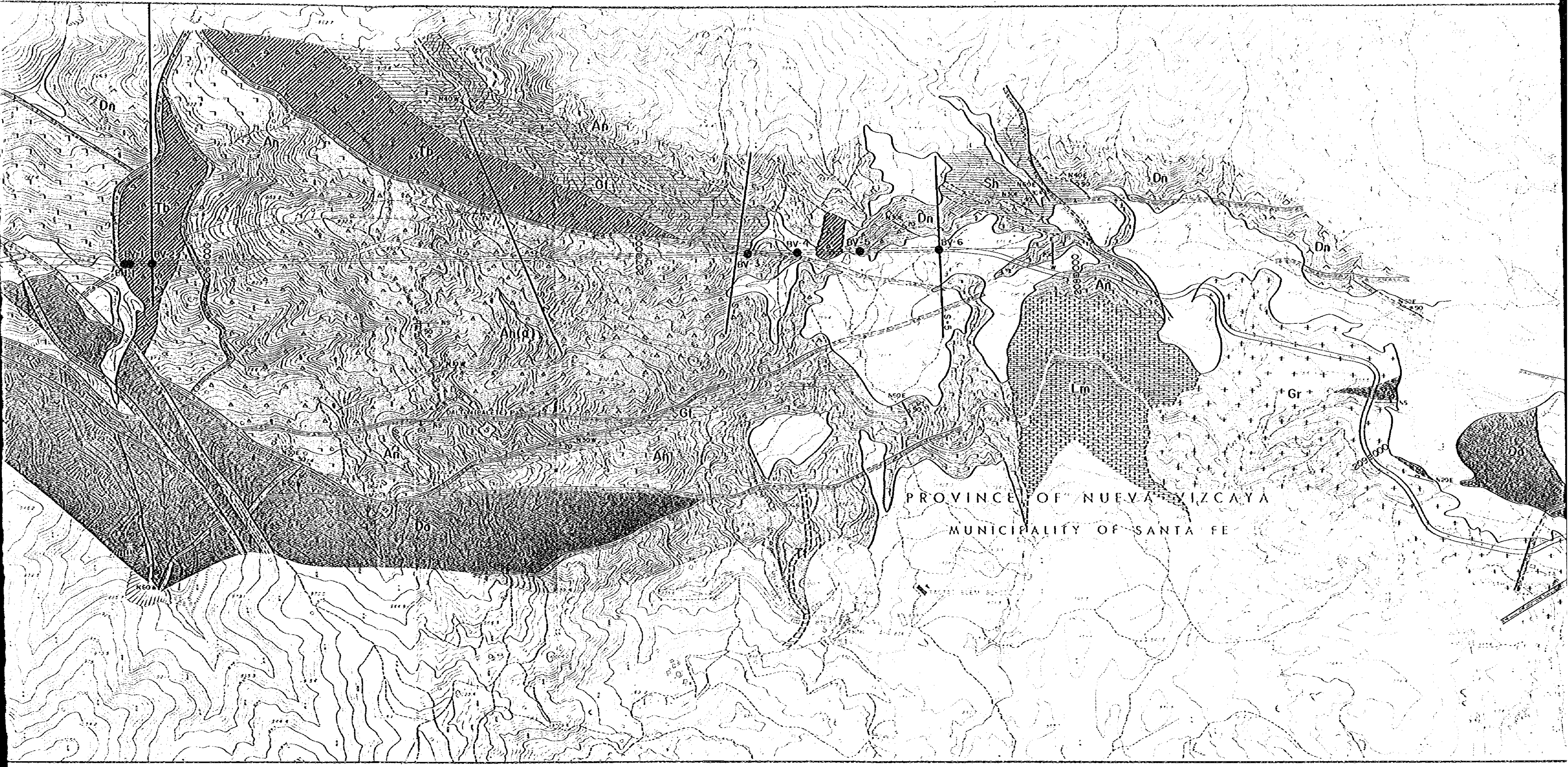
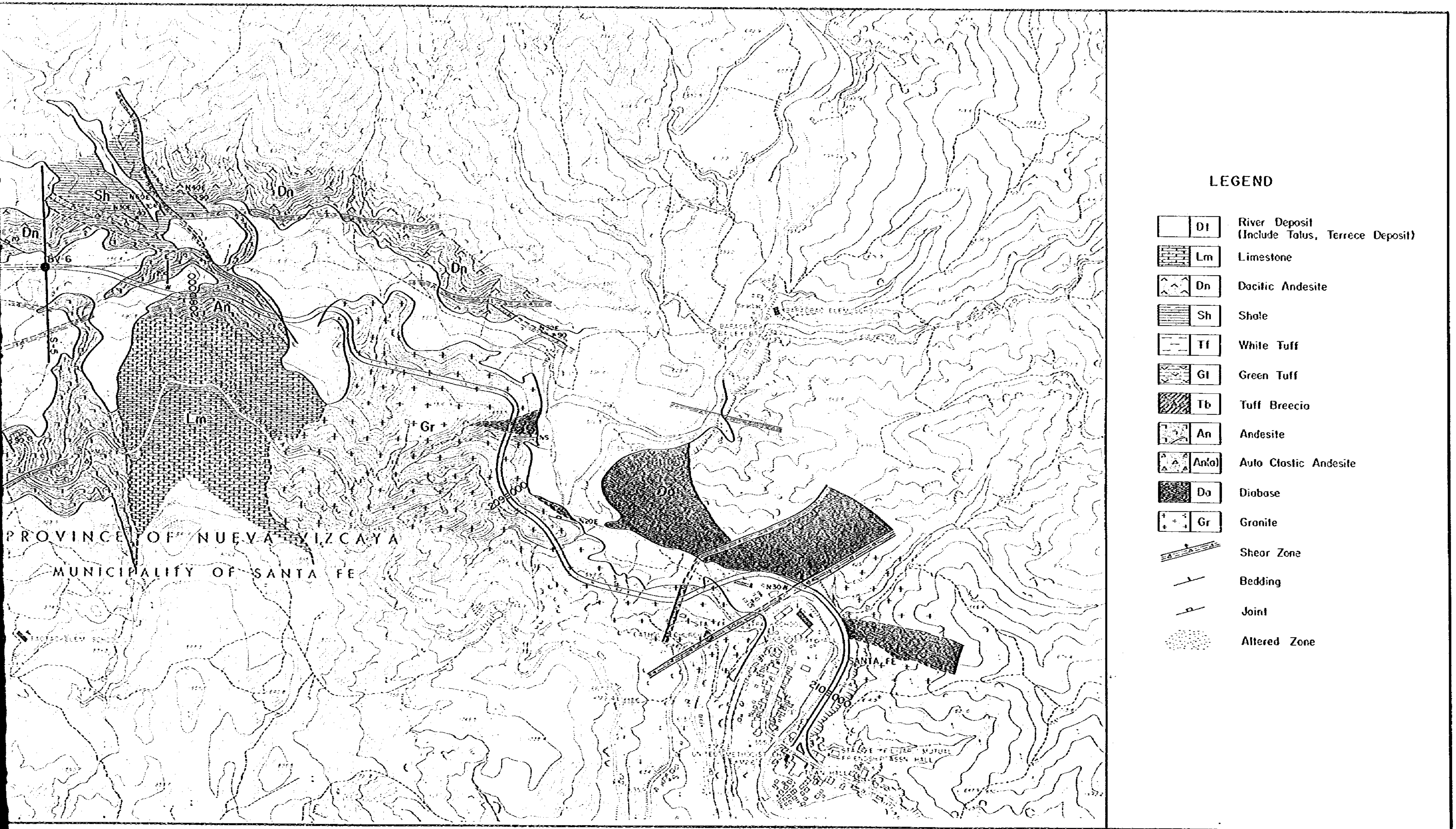


Fig. 4-2.3 GEOLOGICAL PLAN (S=1/5,000)



PROVINCE OF NUEVA VIZCAYA
MUNICIPALITY OF SANTA FE
Dn
An
Tb
Gf
Sh
BV-1
BV-2
BV-3
BV-4
BV-5
BV-6
An(d)
SGf
Dd
Lm
Gr
Tf



Sh
Dn
An
Lm
Gr
Da
PROVINCE OF NUEVA VIZCAYA
MUNICIPALITY OF SANTA FE
SANTA FE
LEGEND
Dt River Deposit (Include Talus, Terrece Deposit)
Lm Limestone
Dn Dacitic Andesite
Sh Shale
Tf White Tuff
Gt Green Tuff
Tb Tuff Breccia
An Andesite
An(a) Auto Clastic Andesite
Da Diabase
Gr Granite
Shear Zone
Bedding
Joint
Altered Zone

Slope protection by planting should be planned in cutting slopes consisting mainly of soil to prevent erosion. However, for cut slopes consisting of weathered rock, concrete spraying or pre-cast concrete frame can be applied. For the embankment slope, seeding with topsoil of 10 cm. must be carried out completely.

4-3 Geotechnical Investigation for Tunnel

Geotechnical investigation was conducted to get the basic information needed for tunnel design and to anticipate construction problems that my arise.

Geotechnical investigation include boring investigation, seismic prospecting, permeability test using the boring hole and rock specimen test.

Location map of Geotechnical Investigation is shown in Drawing GS-6.

4-3-1 General Topography and Geology of Tunnel Section

The proposed tunnel is located at the Dalton Pass section and lies within 500 meters west of the existing Route 5.

(1) Summary of Topography and Geology

1) Topography

Topography of the southern portal is mountainous consisting of steep slopes. The ridge as well as the adjacent river extends basically in the N-S direction. It is inferred by the direction of the ridge and the river, that the tectonic line including the fault, shearing zone and altered zone extends in the N-S direction.

The topography of the northern portal predominantly consists of gentle slopes with flat ricefields. Moreover, the topography in this area is disturbed and the depth of ground water is shallow as evidenced by the presence of marshes. Based on the above, the topography is believed to be of landslide or debris-flow feature.

2) Geology

With reference to the geological maps presented in Fig. 4-2.3, (GS-4) the geological condition of proposed tunnel is described hereunder.

The base rock consists of andesite and tuff breccia in the Caraballo Group that was formed by volcanic activity during Upper Cretaceous to Eocene period. Observation of rock outcrops show that the andesite is harder than tuff breccia, but is severely fissured. On the other hand, the tuff breccia is loose and can easily be broken.

As can be seen on the geological profile presented in Drawing GS-5 the geological distribution of the proposed tunnel formation consists mainly of andesite. However, near both portals of the proposed tunnel, tuff breccia and autoclastic andesite are intercalated with andesite. Thus, it is expected that the tectonic zone including fault, shear zone and altered zones on the tunnel formation exist at Km. 206+100 and Km. 207+300. In the light of these findings, it is believed that at Km. 206+100 and between Km. 207+300 to Km. 207+700, the tunnel's geological condition will be poor for the following reasons:

(i) At Km. 206+100, the tunnel alignment crosses the tectonic zone directly and a lot of springwater is foreseen near the tectonic zone, thus making the geological condition poor.

(ii) Between Km. 207 + 300 and Km. 207 + 700, as the tunnel alignment is running nearly parallel with the tectonic zone, it is expected that geological condition will be very poor. This situation is further worsened by the presence of large amounts of springwater flowing out along these tectonic lines. Moreover, the depth of the tunnel from the surface is less than 30 meters with topographic feature on the upper part of tunnel showing landslide topography and with the geology consisting of alternation of andesite, autoclastic andesite and tuff breccia. Facie change is remarkable.

(iii) On the other sections, it is believed that good geological condition exists, since the base rock of the tunnel formation is fresh andesite and tectonic zones are not expected.

4-3-2 Planning of Geotechnical Investigation

(1) Boring Investigation

Boring survey aims to obtain the geological condition of southern tunnel portal and also at the northern part of the tunnel between Km. 207+300 to Km. 207+700. On the southern tunnel portal, two vertical borings with a depth of 20 to 30 meters each, and one horizontal boring with a length of 70 meters were made. For the northern part of the tunnel, four vertical borings with depths ranging from 25 to 40 meters each were conducted.

(2) Seismic Prospecting

Seismic prospecting was intended to get the the P wave velocity of the the base rock of tunnel formation. This is because the classification of base rock in tunnel design based on the P wave velocity, otherwise known as the elastic wave, passes through at high velocity in hard rock more than in soft rock.

Planned survey line is composed of one longitudinal line with length of 2,420 meters and four crosslines with length of 400 to 660 meters each. The interval of geophones is five meters near the tunnel portal and ten meters on other sections. Explosives were used to generate seismic waves.

The location map of geotechnical investigation including boring points and seismic survey lines are shown in Drawing GS-6.

(3) Permeability Test

It is anticipated that one of the problems which might be encountered during the construction of the tunnel is the abundance of springwater within the tunnel formation. In order to determine the quantity of springwater from the tunnel, permeability test was conducted using the vertical borings after completion of drilling operations.

In the permeability test, a portion of the packer was set on the upper part of the tunnel crown and in accordance with pressure test, permeability factor of the base rock for tunnel formation was determined.

(4) Rock Specimen Test

Rock specimen tests were done to get the necessary data for tunnel design. The types of rock specimen tests conducted were unconfined compression tests.

4-3-3 Program of Geotechnical Investigation

The program of geotechnical investigation is shown on Table 4-3.1.

TABLE 4-3.1 LIST OF GEOTECHNICAL INVESTIGATION

(1) Boring

Boring	Station (km.)	Drilling Depth (m)	Rock Specimen Test (piece)	Permeability Test (Times)	Remarks
BH-1	Km. 205 + 815	70	2	0	Horizontal boring, southern portal
BV-1	Km. 205 + 830	20	0	1	Vertical boring, southern portal
BV-2	Km. 205 + 850	30	1	1	Vertical boring, southern portal
BV-3	Km. 207 + 250	40	3	1	Vertical boring, northern portal
BV-4	Km. 207 + 370	35	2	1	Vertical boring, northern portal
BV-5	Km. 207 +510	30	1	1	Vertical boring, northern portal
BV-6	Km. 207 + 700	25	2	1	Vertical boring, northern portal
	Total	250	11	6	

(2) Seismic Prospecting

Survey Line	Survey Length	Remarks
S-1	2420	Longitudinal Line
S-2	660	Cross Line
S-3	605	Cross Line
S-4	440	Cross Line
S-5	440	Cross Line
	Total 4565	

4-4 Results of Geotechnical Investigation

4-4-1 Boring Investigation

(1) Summary of Each Boring

The results of the boring investigation is presented in the logs of the borings and boring core photographs. Based on the boring investigation the findings can be summarized as follows with results shown in Fig. 4-4.1.

1) BH-1 (Km. 205 + 815)

BH-1 boring is a horizontal boring that was conducted at Km. 205+815 near the southern portal. The boring core from the surface down to 2.70 meters consists of topsoil which is sandy, with presence of small breccia.

At 2.70 meters existence of old river deposits consisting of tuff breccia and andesite breccia and other breccia were confirmed up to a depth of 10 meters.

The boring core between 10 meters to 70 meters consists of tuff breccia which is partly sheared and andesite sheets. The tuff breccia is very loose and easily broken due to abundant fissures. Thus, the core samples are in fragmented form while some are short and long.

Between 39.50 to 46.0 meters, which is probably the shearing zone, the formation shows clayzation and chloritization. Between 46.0 meters to 50.0 meters and from 69.50 to 69.80 meters the cores consist of hard andesite sheets.

2) BV-1 (Km. 205 + 830)

BV-1 boring was conducted at the southern entrance of the tunnel up to a depth of 20 meters. The boring core from the surface to 3.0 meters consists of topsoil and between 3.0 to 6.0 meters consists of weathering soil which is sandy in nature.

At 6.0 meters, the existence of base rock consisting of tuff breccia was confirmed. The condition of this tuff breccia from 6.0 meters to 20.0 meters is very loose and easily broken due to weathering. Thus, the cores are almost in fragments, including the short core with length of less than 5 cm. between the drilling depth of 10 meters to 15 meters.

3) BV-2 (Km. 205 + 880)

BV-2 boring was conducted at Km. 205 + 880 and its drilling depth is 30 meters. Boring core from surface to 2.5 meters is ordinary topsoil. At 2.5 meters the base rock encountered consisted of tuff breccia just as in BV-1.

The condition of base rock between 2.5 meters to 7.5 meters appears to be of weathering soil which includes rock fragments but the matrix of weathering soil is sticky and rich in clay. Between 7.5 meters to 10.5 meters is breccia with clay zone assumed to be the shearing zone.

Between 10.5 meters to 25.5 meters, boring core including short cores consist mainly of fragments. Between 25.5 meters to 30 meters, core recovery showed a mixed zone with core length of 5 to 10 cm.

Based on the result of BV-1 and BV-2 borings, the geological condition at the tunnel portal can be considered poor. However, observation on the existing outcrop near the boring point indicates that even though the tuff breccia is loose and weathered, it may not be bad for tunnel geology because the tuff breccia has little or no cracks.

4) BV-3 (Km. 207 + 250)

BV-3 boring was conducted at Km. 207 + 250 with a drilling depth of 40 meters. The investigation results are as follows:

The boring from surface to a depth of 2.2 meters is brown topsoil including small breccia with diameters ranging from 2 cm. to 5 cm. The sample showed a 10 to 20% mixture percentage ratio of breccia and the matrix is clayey in nature.

At a depth of 2.2 meters, andesite corresponding to the base rock was confirmed. The condition of the core samples between 2.2 to 3 meters is weathered andesite and are highly fractured. The cores are almost fragments with red brown cracked planes caused by oxidation.

At a depth of 3 to 4 meters, 6 to 7 meters and 15 to 30 meters, the cores recovered are soft with fragments due to chloritization. Furthermore, at the drilling depth of 25 meters to 27.0 meters, a layer of shearing zone is believed present due to the condition of the boring cores.

However, the existing condition of the base rock is considered generally fair for tunnel geology on most occasions since the cores recovered are long with lengths of 5 cm. to 15 cm. and with high percentage of core recovery.

5) BV-4 (Km. 207 + 370)

BV-4 was done at Km. 207 + 370 with a depth of 35 meters. As shown by the results of the boring, the materials from the surface to 6.7 meters depth consists of talus and river deposits. At 6.7 meters depth, andesite corresponding to the base rock in this area was confirmed by the boring survey.

The base rock between 6.7 meters to 10 meters is weathered andesite. The core is fragmented with brown cracked planes due to oxidation. Between 10 meters to 35 meters, core recovery consists of short and long cores with lengths of more than 5 cm. Along the cracked planes in the long cores, oxidation signs evidenced by brownish color reveals the presence of ground water in these cracks.

6) BV-5 (Km. 207 + 510)

BV-5 boring was done at Km. 207+510 with a depth of 30 meters. Near the survey area, outcrops of andesite base rock are observed. It is believed that topsoil in the area is thin and was confirmed by the boring survey to be only 1.2 meters. The base rock between 1.2 meters to 4 meters is weathered andesite and are highly fractured. Therefore, core recovered consists of fragments. Between 4 to 9 meters, core is mainly more than 5 cm. long. The cracks show the presence of ground water as evidenced by oxidation. Between 9 to 20 m, core recovered consists of fragments with andesite. Between 20 to 30 m, short and long cores were recovered.

7) BV-6 (Km. 207+700)

BV-6 was conducted at the tunnel entrance of northern portal with a drilling depth of 25 meters. In this area, the existing topography shows gentle slope and landsliding features. Here, the thickness of the topsoil is believed to be more than 10 meters. Based on boring tests, from surface to 14 meters depth is talus deposit. This talus deposit shows changing facies like breccia mixed clay, gravel bed, and white clay at the boundary of talus deposits and base rock. Based on the above facies, it is expected that the topography consisted of repeated movements of landslide and debris flow.

From a depth of 14 meters, andesite forming the base rock was confirmed. The base rock between 14 meters to 20.3 meters is highly fractured weathered andesite. As before, the cores are fragmented. Between 20.3 meters to 25 meters,although some fragmentation exists, most of the cores recovered are long, showing good condition.

The results of the boring surveys at the northern part of the tunnel clearly indicates that the geological condition at the tunnel formation is made up of andesite. With appropriate measures undertaken for control of springwater from the cracks, no major problem is foreseen in tunnel construction. However, it is deemed necessary for future structures, to conduct a detailed survey which will include the landslide area.

8) Relation Between RQD and Condition of Base Rock

Using the drilling core length, the RQD on each boring was determined as shown on Fig. 4-4.1. Based on the RQD on each boring, condition of the base rock can be estimated.

Generally, the relation between RQD and condition of the base rock is shown in Table 4-4.1

TABLE 4-4.1 RELATION BETWEEN RQD AND CONDITION OF BASE ROCK

RQD (%)	Condition of Base Rock
0 - 25	Very Poor
25 - 50	Poor
50 - 70	Fair
75 - 90	Good
90 - 100	Excellent

As shown on the above table, relation between RQD and base rock condition on each boring are as follows:

a) SOUTH PORTAL

BH-1 RQD is less than 10%, condition of base rock is very poor.

BV-1 RQD is averaging between 10% to 20%, the condition of base rock is very poor.

BV-2 RQD is less than 10%, condition of base rock is very poor.

In cases where the base rock is tuff breccia, RQD is approximately less than 10%; thus, it can be inferred that the condition of base rock is very poor, since tuff breccia is very loose, porous and easily broken.

b) NORTH PORTAL

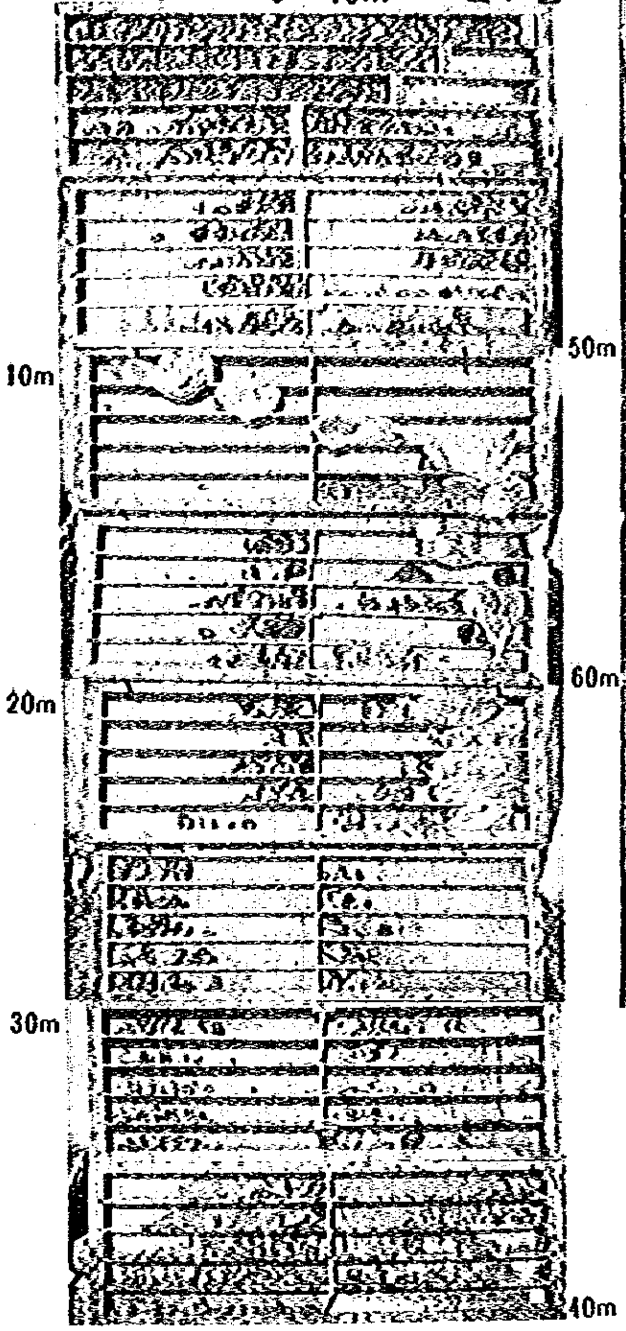
With Fig. 4-4.1 as basis, the relation between RQD and base rock were taken for the following:

BV-3

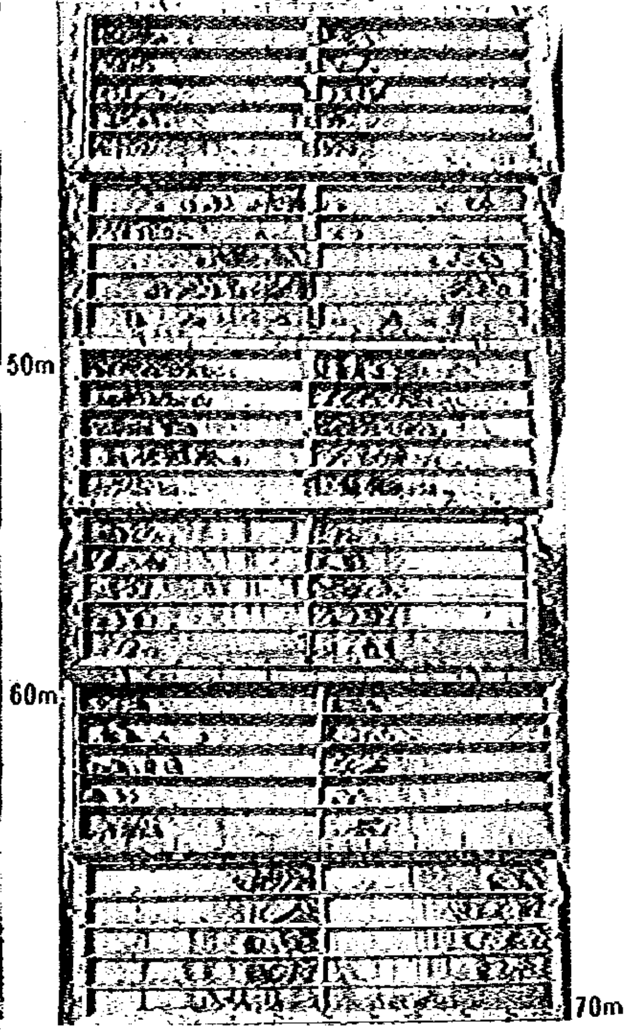
TABLE 4-4.2 CONDITION OF BASE ROCK IN BV-3 BASED ON RQD

Depth	Condition of Base Rock
3.0 - 10.0 m	Poor
10.0 - 30.0 m	Very Poor
30.0 - 35.0 m	Poor
35.0 - 38.0 m	Fair
38.0 - 40.0 m	Poor

BH-1 ℓ=0~40m

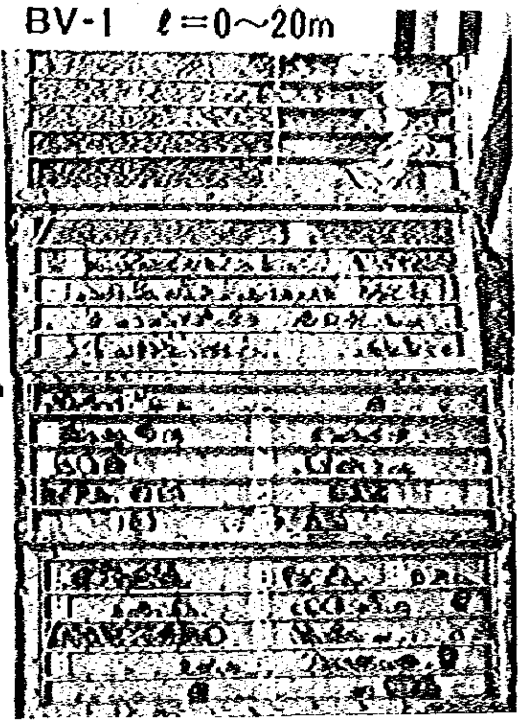


BH-1 ℓ=40~70m

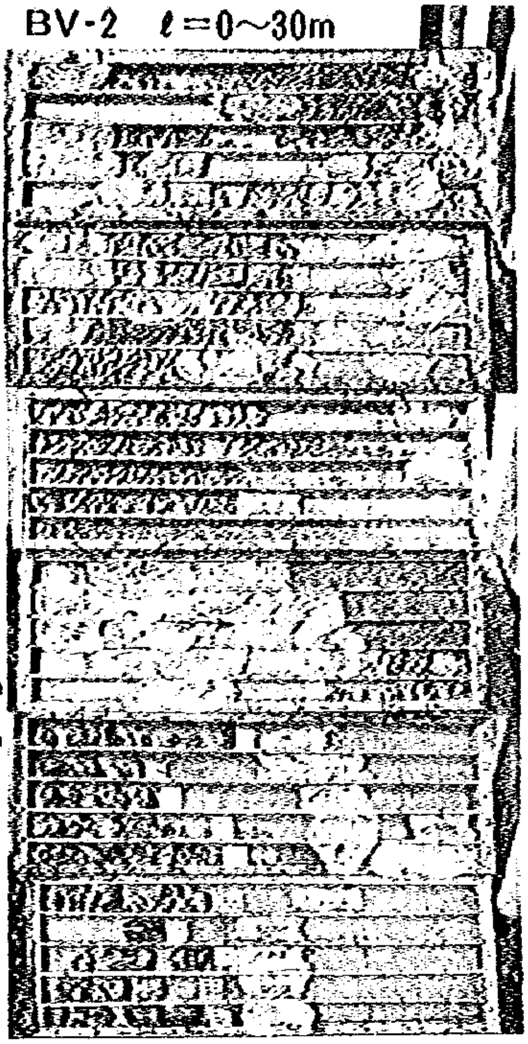


GEO-TECHNICAL INVESTIGATION RESULTS. Boring investigations were carried out at seven (7) different stations in order to gather accurate geo-technical findings of the area. The following pages are photos of core samples that were taken from each boring point.

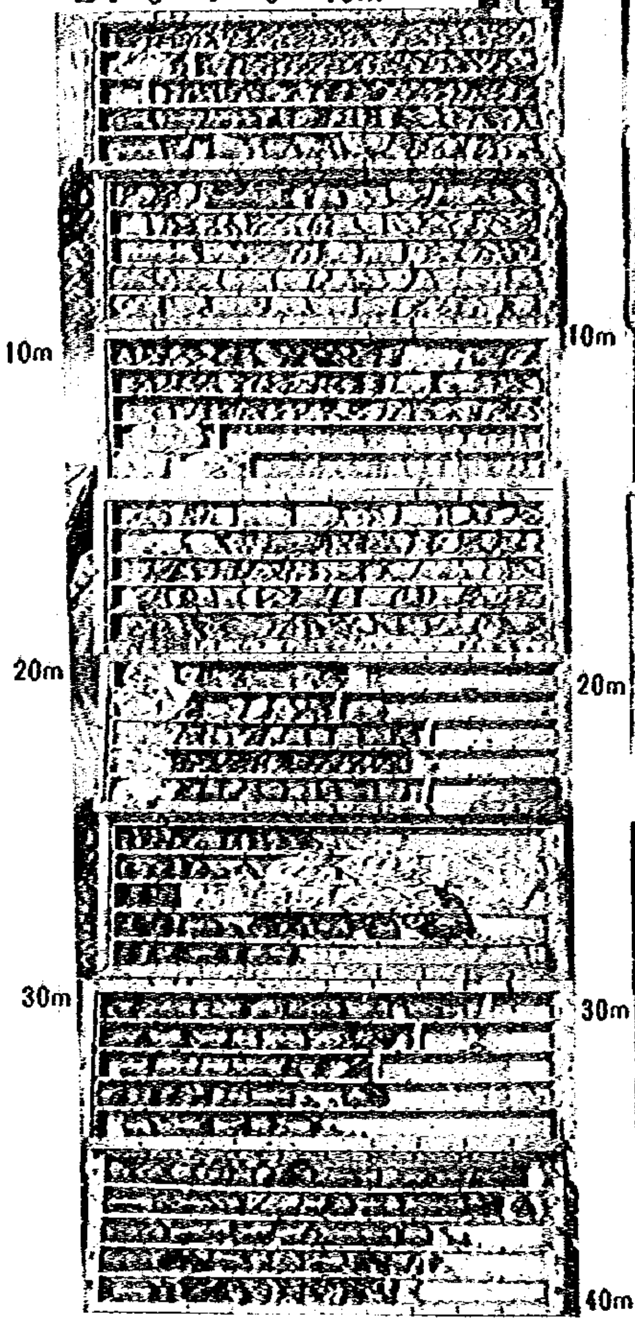
BV-1 ℓ=0～20m



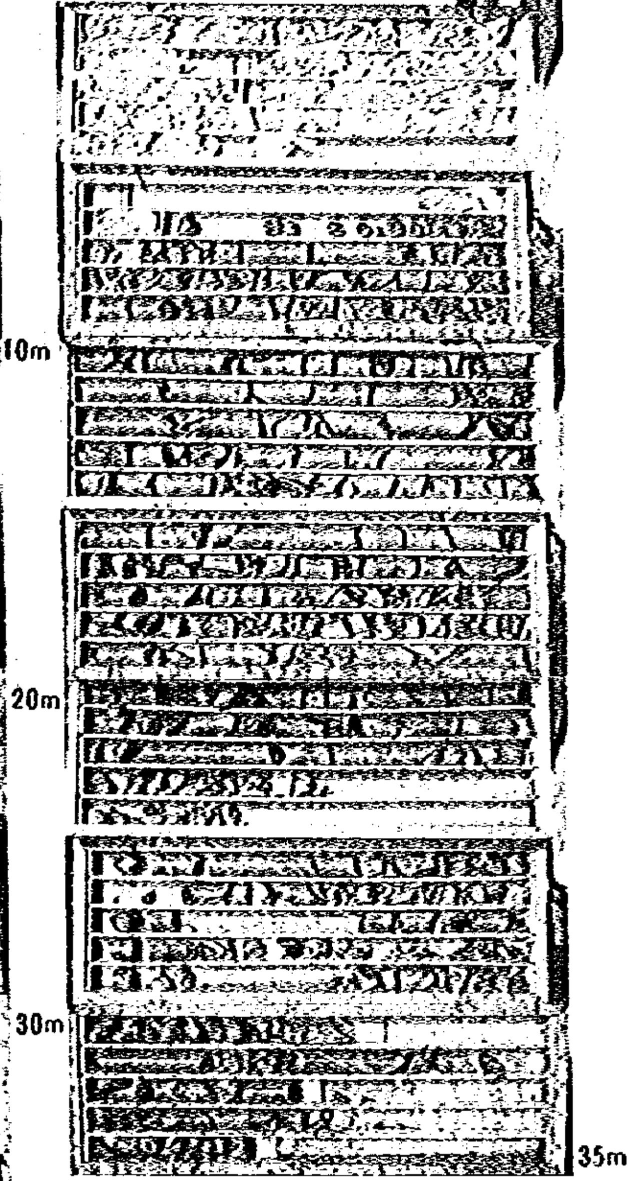
BV-2 ℓ=0～30m

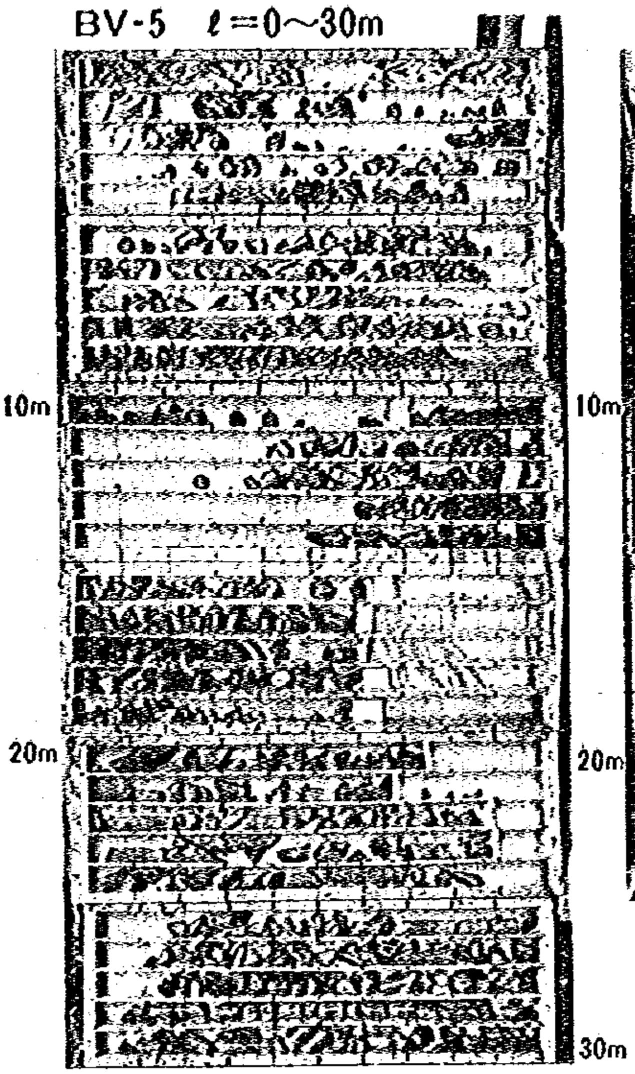


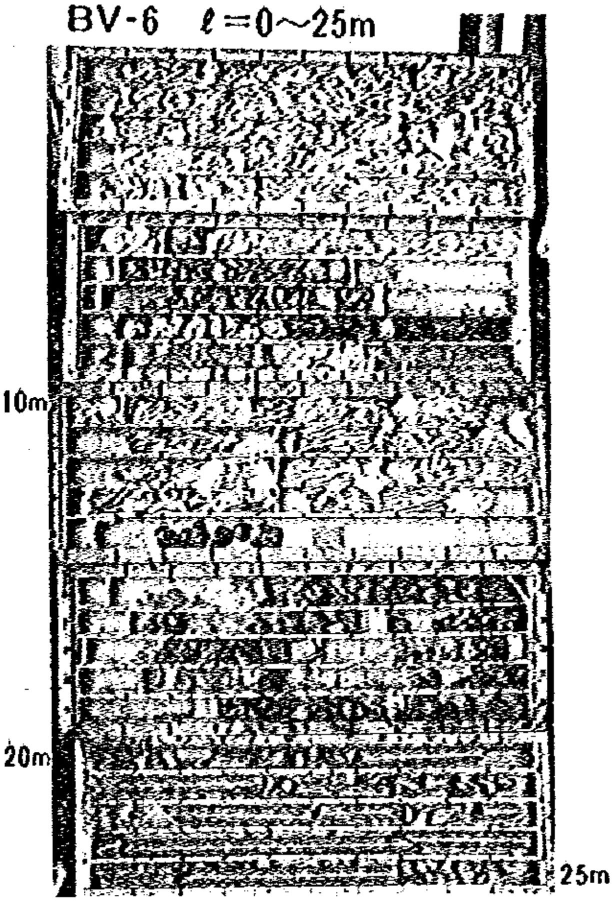
BV-3 ℓ=0～40m



BV-4 ℓ=0～35m




BV-5 ℓ=0～30m
10m
20m
30m


BV-6 ℓ=0～25m
10m
20m
25m

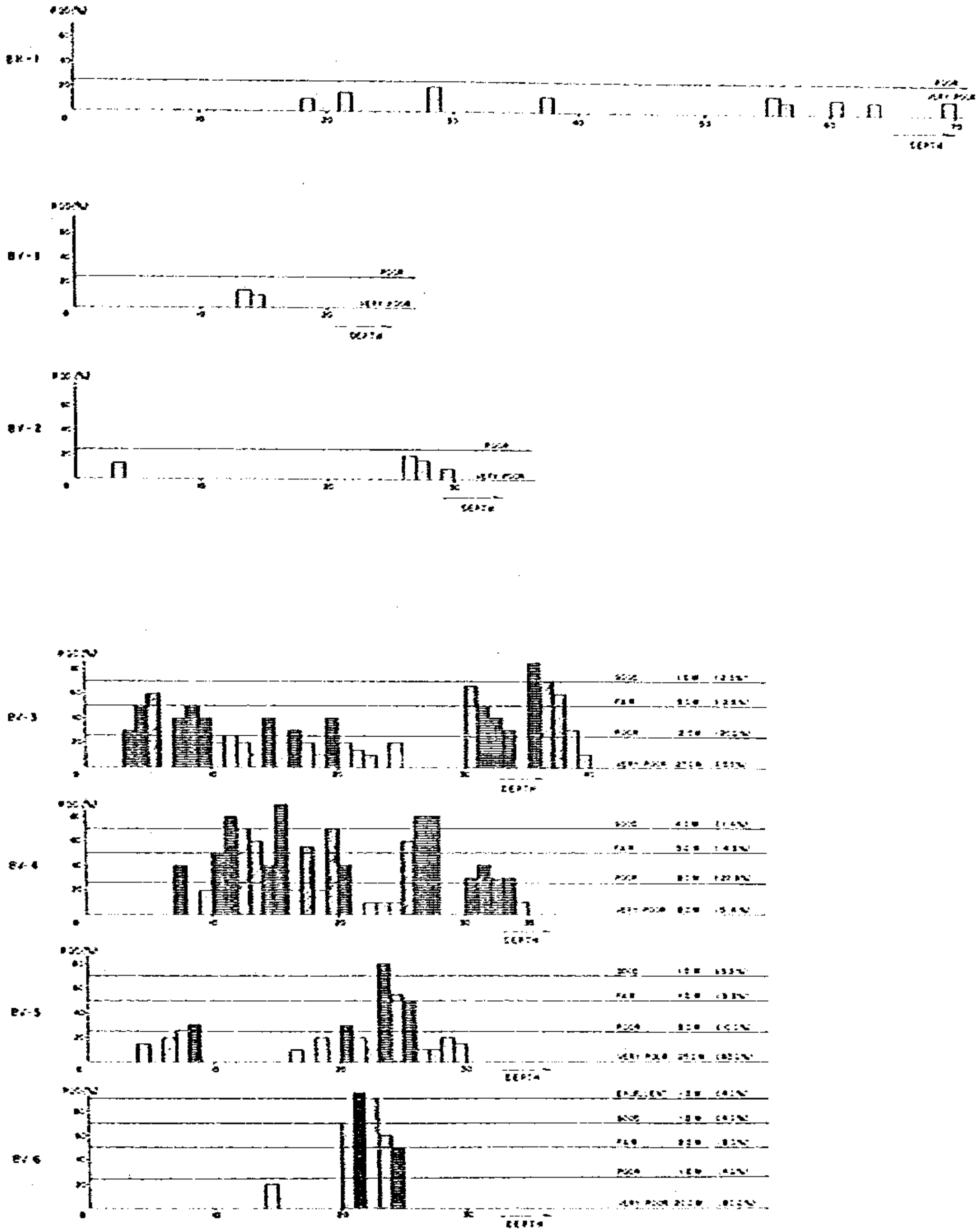


Fig. 4-4.1 RELATION BETWEEN RQD AND CONDITION OF BASE ROCK (TUFF BRECCIA)

BY-4

TABLE 4-4.3 CONDITION OF BASE ROCK IN BV-4 BASED ON RQD

Depth	Condition of Base Rock
0 - 10.0 m	Very Poor
10.0 - 21.0 m	Fair
21.0 - 25.0 m	Very Poor
25.0 - 28.0 m	Good
28.0 - 30.0 m	Very Poor
30.0 - 35.0 m	Poor

BV-5

TABLE 4-4.4 CONDITION OF BASE ROCK IN BV-5 BASED ON RQD

Depth	Condition of Base Rock
0 - 23.0 m	Very Poor
23.0 - 26.0 m	Fair
26.0 - 30.0 m	Very Poor

Based on RQD, condition of base rock is approximately poor or fair.

4-4-2 Seismic Prospecting

Seismic surveys were done to get the P wave velocity of base rock at tunnel formation. This is necessary because the classification of base rock in tunnel design is based on the P wave velocity.

(1) Distribution of P Wave Velocity

According to seismic prospecting, distribution of P wave velocity is divided into four (4) velocity layers. The relation between layer velocity and type of rock and its condition on each survey line is shown as Table 4-4.5 to Table 4-4.9

1) S-1 Survey Line (Fig. 4-4.2 (GS-12))

TABLE 4-4.5 DISTRIBUTION OF P WAVE VELOCITY IN S-1 SURVEY LINE

Velocity Layer	P Wave Velocity	Thickness	Condition of Rock
1st	0.2 km/sec	less than 5.0 m	Topsoil
2nd	0.8-1.6 km/sec	5 to 30 m	Lower Talus
			Weathering zone
3rd	2.7-3.3 km/sec		Base rock
low velocity	1.0-1.8 km/sec	20 to 40 m	Four (4) group shearing zone

2) S-2 Survey Line (GS-13)

TABLE 4-4.6 DISTRIBUTION OF P WAVE VELOCITY IN S-2 SURVEY LINE

Velocity Layer	P Wave Velocity	Thickness	Condition of Rock
1st	0.2 km/sec	less than 5.0 m	Topsoil
2nd	0.5-1.1 km/sec	5 to 25 m	Weathering zone
3rd	2.3-3.3 km/sec		Base rock
low velocity	1.6 km/sec	25 m	One (1) group shearing zone

3) S-3 Survey Line (GS-14)

TABLE 4-4.7 DISTRIBUTION OF P WAVE VELOCITY IN S-3 SURVEY LINE

Velocity Layer	P Wave Velocity	Thickness	Condition of Rock
1st	0.2 km/sec	less than 5.0 m	Topsoil
2nd	0.4-0.9 km/sec	5 to 20 m	Weathering zone
3rd	2.0-2.8 km/sec		Base rock
low velocity	0.7-1.2 km/sec	15 to 30 m	Three (3) group shearing zone

4) S-4 Survey Line (GS-15)

TABLE 4-4.8 DISTRIBUTION OF P WAVE VELOCITY IN S-4 SURVEY LINE

Velocity Layer	P Wave Velocity	Thickness	Condition of Rock
1st	0.2-0.4 km/sec	less than 5.0 m	Topsoil
2nd	0.8-1.1 km/sec	5 to 15 m	Weathering zone
3rd	3.0-3.3 km/sec		Base rock
low velocity			

5) S-5 Survey Line (GS-16)

TABLE 4-4.9 DISTRIBUTION OF P WAVE VELOCITY IN S-5 SURVEY LINE

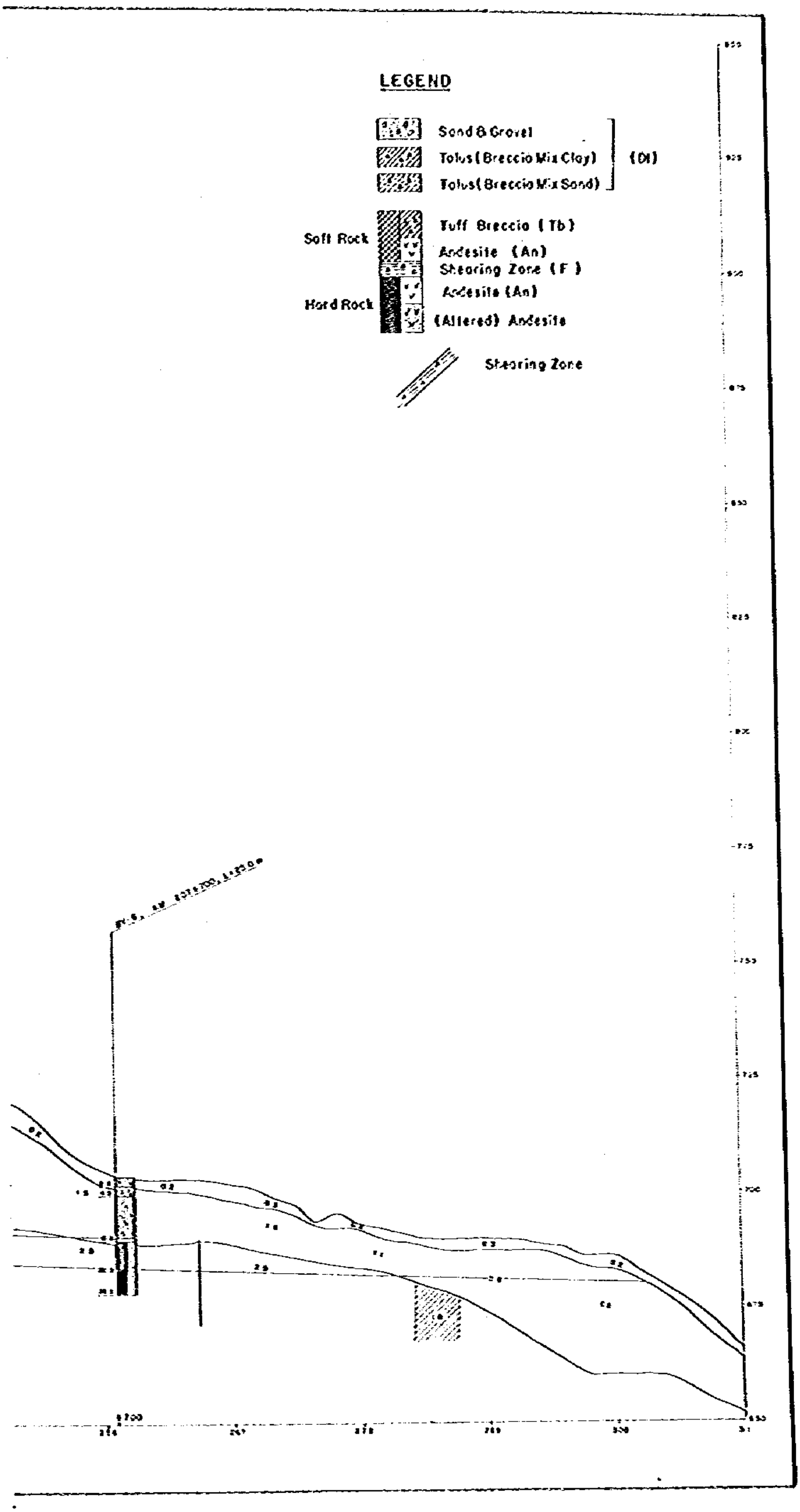
Velocity Layer	P Wave Velocity	Thickness	Condition of Rock
1st	0.2-0.3 km/sec	less than 5.0 m	Topsoil
2nd	0.8-1.7 km/sec	3 to 30 m	Weathering zone
3rd	2.5-2.8 km/sec		Base rock
low velocity			

According to the seismic prospecting P wave velocity at tunnel formation approximately belongs to the third layer. It can therefore be considered that, geological condition at tunnel is fair.

Based on Fig. 4-4.3 (GS-17), low velocity zone is distributed at the southern portal, near the Dalton Pass and along its direction. It is expected that it extends NE-SW and NW-SE crossing 30°-40° with respect to tunnel alignment, and then classification of velocity on tunnel length are as follows:

TABLE 4-4.10 CLASSIFICATION OF P WAVE VELOCITY AT TUNNEL FORMATION

Velocity	Length	Percentage Ratio
2.5-3.0 km/sec	700 m	37.4%
more than 3.0 km/sec	1,110	59.4%
low velocity	60	3.2%
Total	1,870	


LEGEND
Sand & Gravel
Talus (Breccia Mix Clay)
Talus (Breccia Mix Sand)
(Dl)
Soft Rock
Tuff Breccio (Tb)
Andesite (An)
Shearing Zone (F)
Hard Rock
Andesite (An)
(Altered) Andesite
Shearing Zone

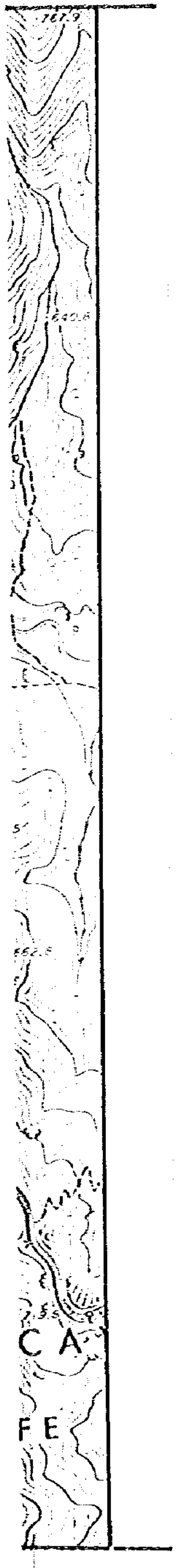

CA
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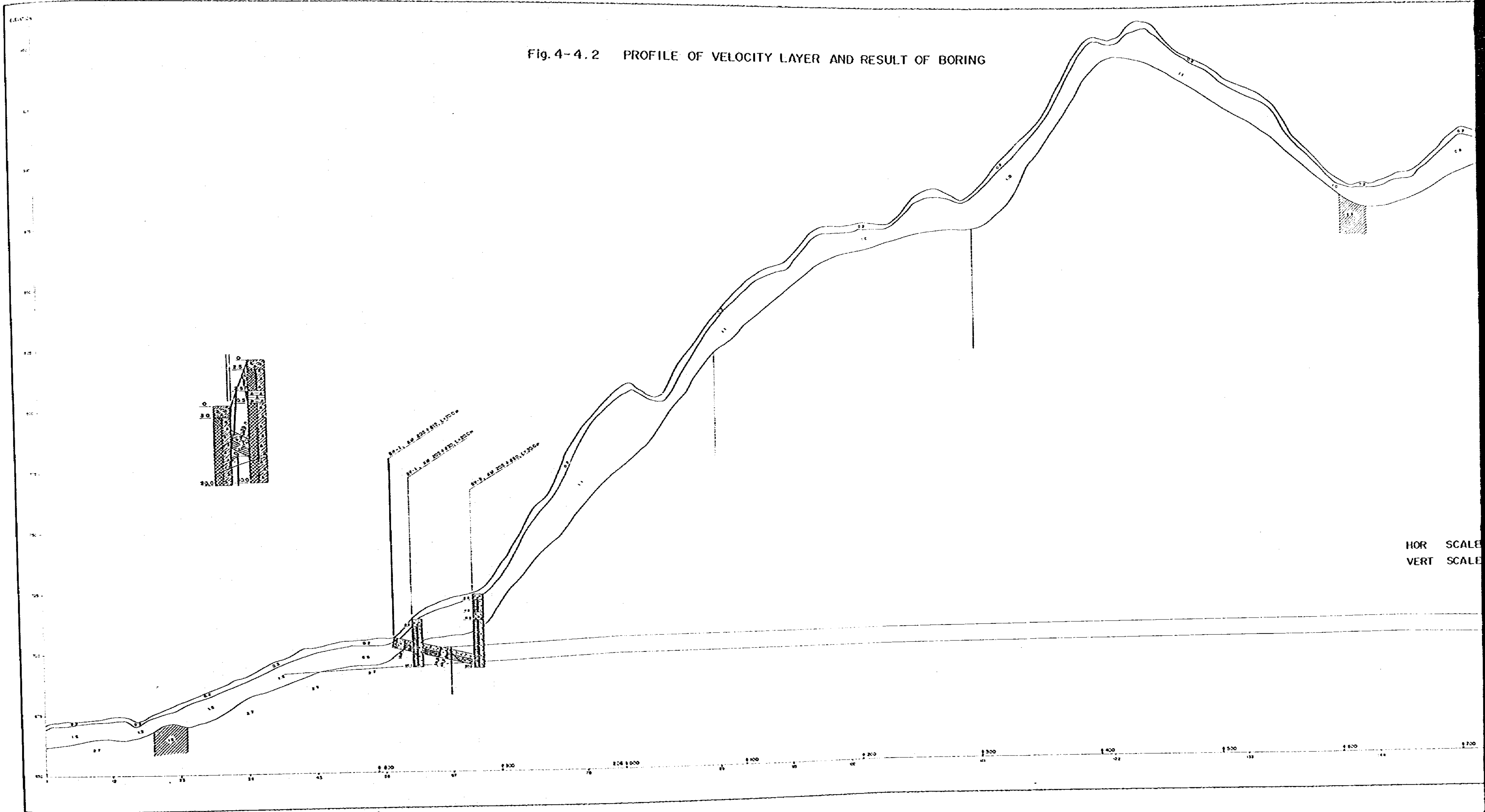
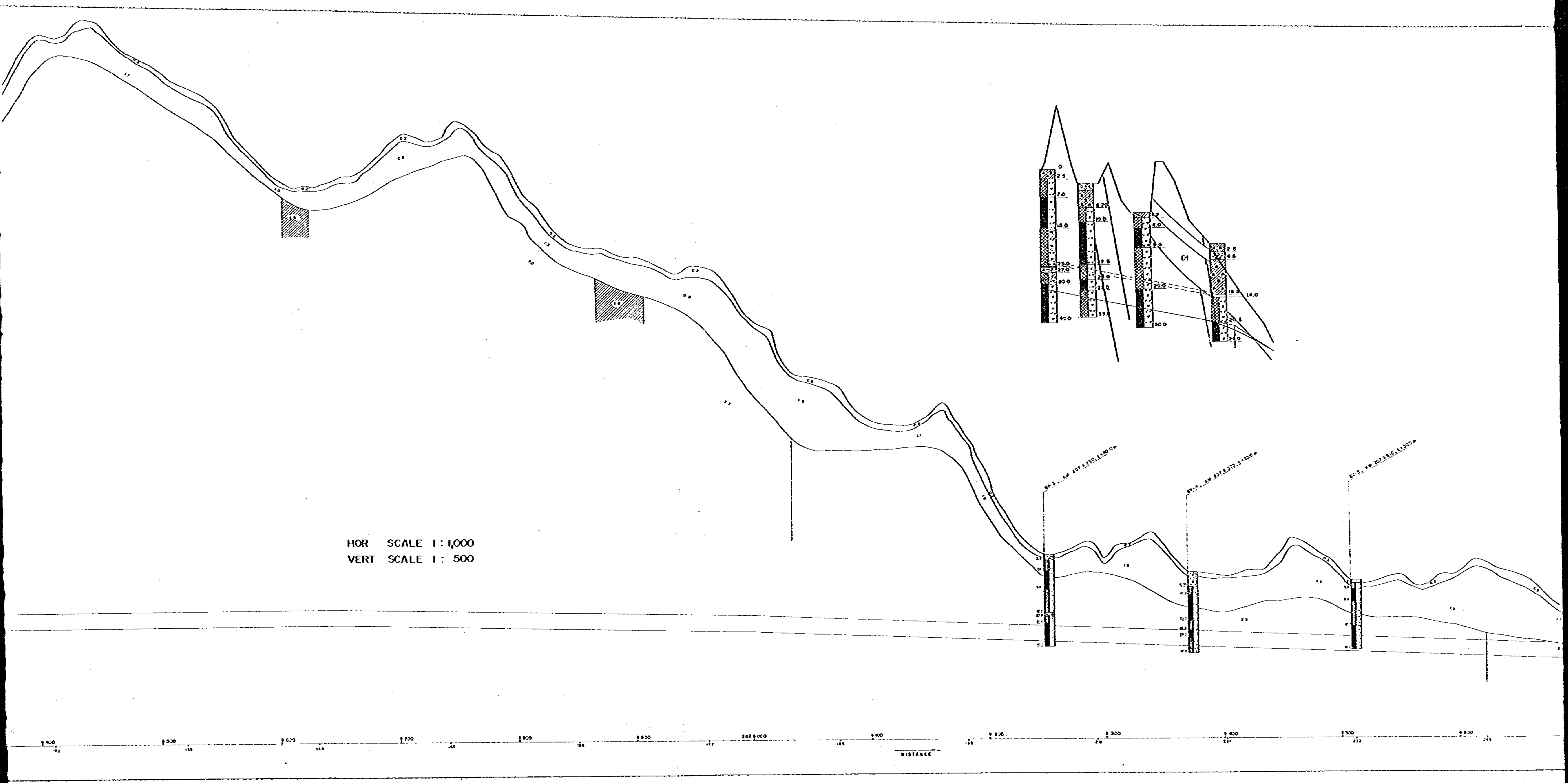
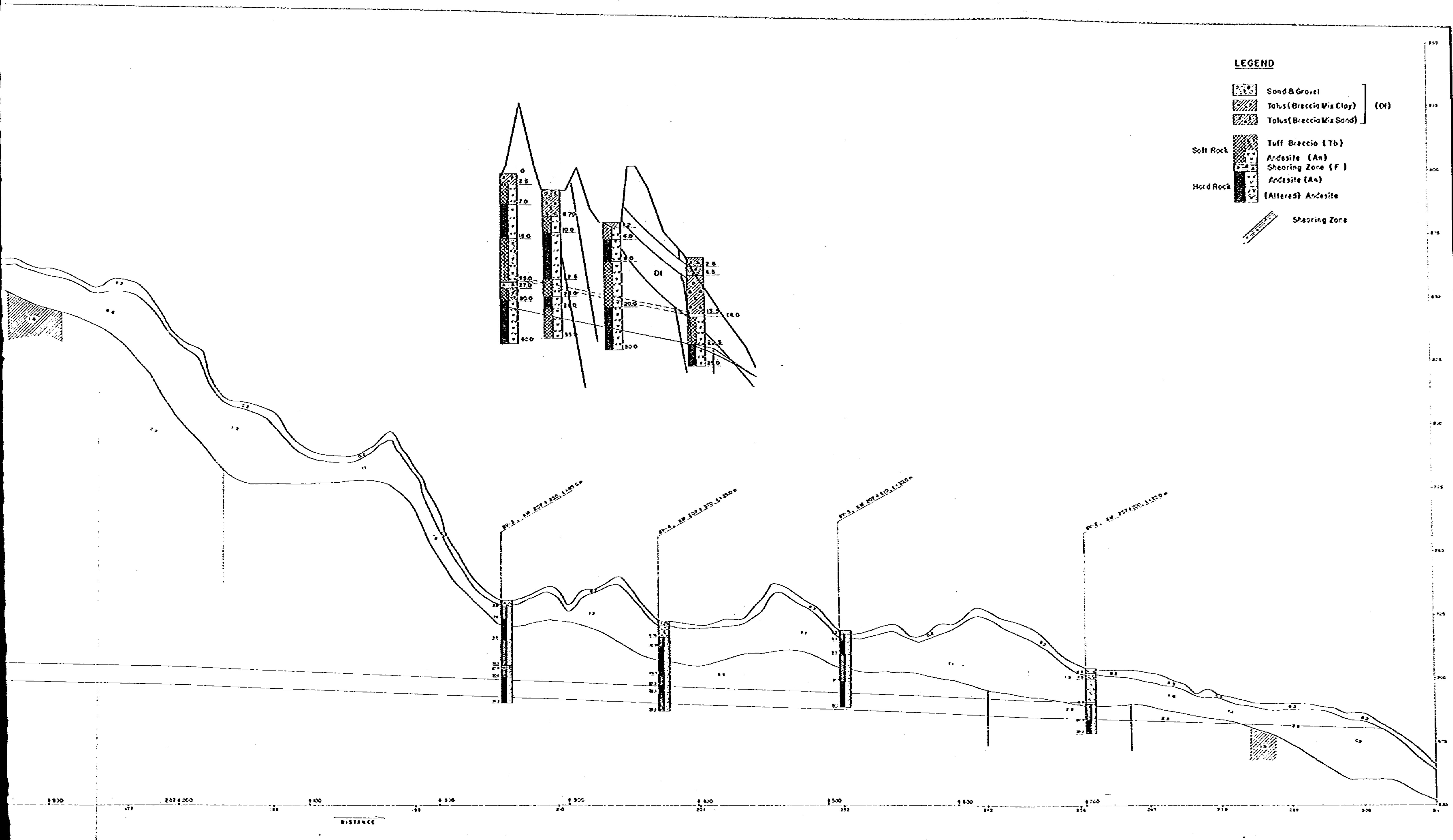
Fig. 4-4.2 PROFILE OF VELOCITY LAYER AND RESULT OF BORING




HOR SCALE 1:1,000
VERT SCALE 1: 500
D1
DISTANCE


LEGEND
Sand & Gravel
Talus (Breccia Mix Clay)
Talus (Breccia Mix Sand)
(Dt)
Soft Rock
Tuff Breccia (Tb)
Andesite (An)
Shearing Zone (F)
Hard Rock
Andesite (An)
(Altered) Andesite
Shearing Zone
Dt
DISTANCE

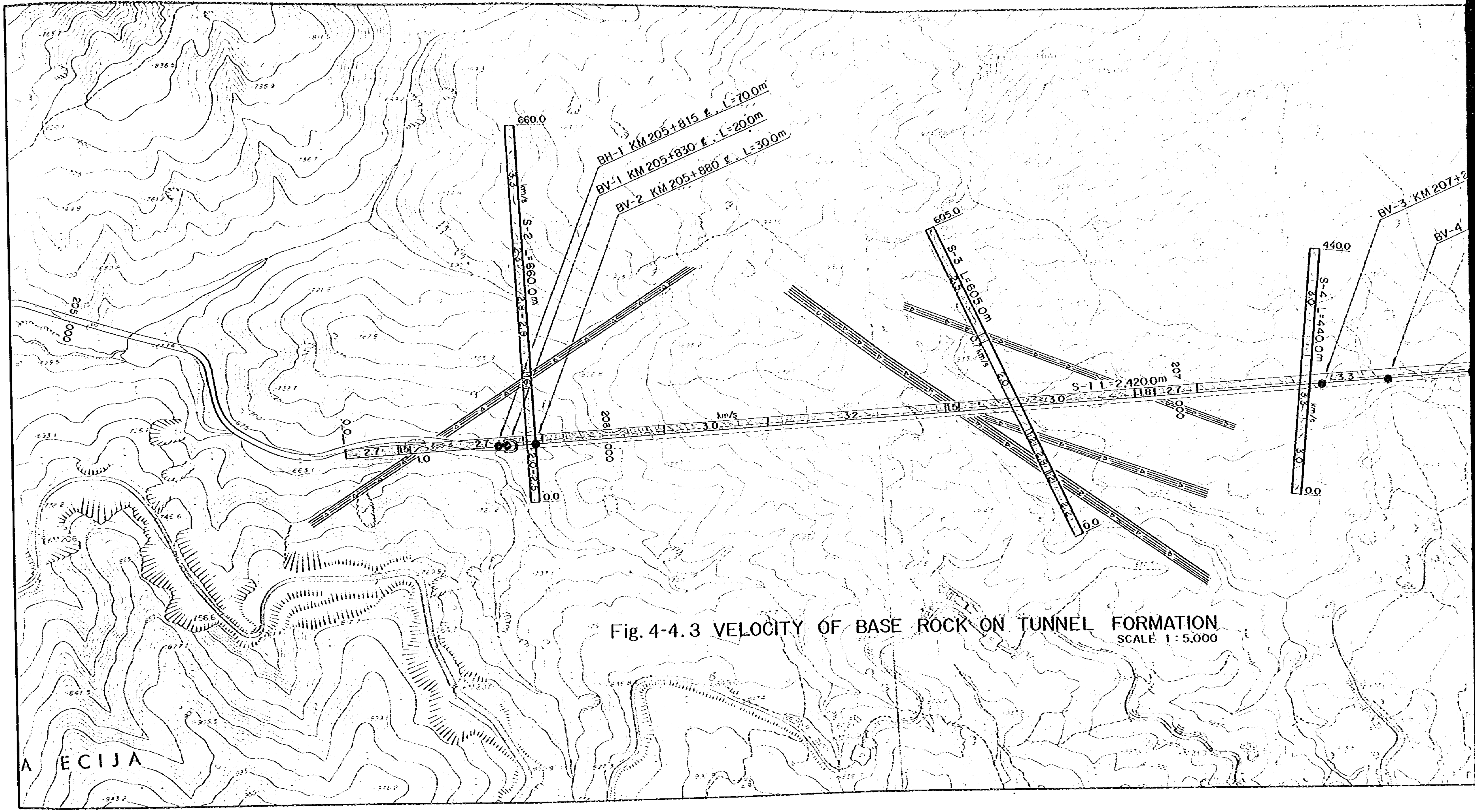
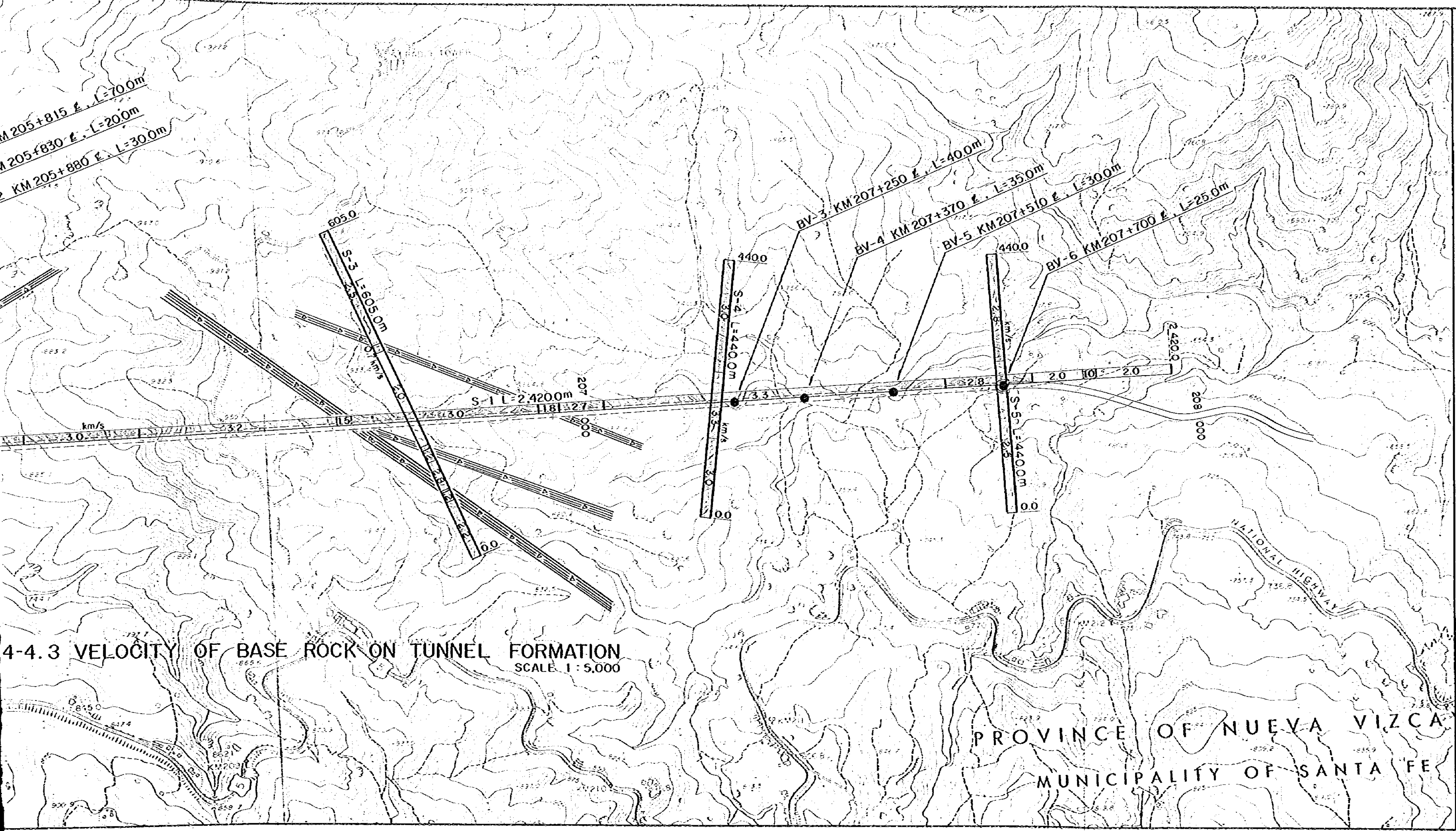


Fig. 4-4.3 VELOCITY OF BASE ROCK ON TUNNEL FORMATION
SCALE 1:5,000



4-4.3 VELOCITY OF BASE ROCK ON TUNNEL FORMATION

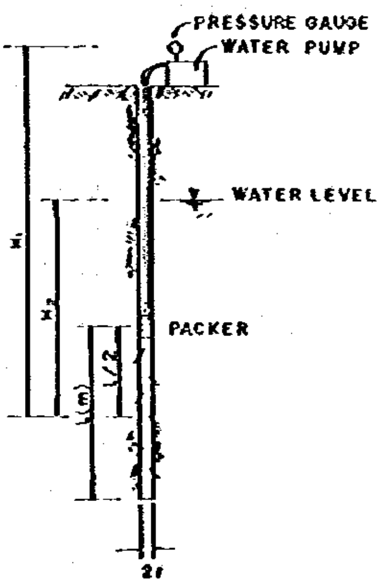
SCALE 1 : 5,000

At the tunnel formation, velocity of base rock of more than 3.0 km/sec occupies about 60% of tunnel length. From this point of view, it can be considered that the geological condition of the tunnel formation is fair.

4-4-3 Permeability Test

(1) General

Permeability test was intended to determine the permeability factor of base rock near the tunnel formation. Permeability test was conducted using the water pressure method, and the section permeability was calculated by the following equation.



$$K = \frac{Q}{2\pi R(L)(H)} \ln\frac{L}{r}$$

WHERE:

K = PERMEABILITY CONSTANT (m/min)
Q = CONSTANT RATE OF FLOW (L/min)
L = LENGHT OF THE TEST SECTION (m)
H = DIFFERENTIAL HEAD ON THE TEST SECTION (m)
r = RADIUS OF THE BOREHOLE (m) NQ BIT

$$H = H_1 - H_2 + H_3$$

WHERE:

H_1 = DEPTH OF L/2 + HEIGHT OF PRESSURE GAUGE
H_2 = DEPTH OF L/2 − WATER LEVEL
H_3 = PRESSURE × 2.31

(2) Results of Permeability Test (GS-18)

Average permeability factor on each vertical boring are shown on the following Table 4-4.11

TABLE 4-4.11 RESULTS OF PERMEABILITY TEST ON VERTICAL BORING

Boring No.	Depth of Packer	Permeability Factor	Geology
BV - 1	10 m	1.545×10^{-5} m/min	Tuff breccia
BV - 2	20 m	1.293×10^{-5} m/min	Tuff breccia
BV - 3	30 m	5.75×10^{-6} m/min	Andesite
BV - 4	25 m	4.64×10^{-6} m/min	Andesite
BV - 5	20 m	9.22×10^{-6} m/min	Andesite
BV - 6	15 m	1.147×10^{-5} m/min	Andesite

In accordance with Table 4-4.11, the permeability of the tuff breccia seems to be more than the andesite. This may be so because the tuff breccia appears to be more porous.

4-4-4 Rock Specimen Test (GS-19, GS-20)

Using the intact core rock samples, unconfined compression tests were conducted. Results of these tests are as follows:

TABLE 4-4.12 RESULTS OF ROCK SPECIMEN TEST

Boring No.	Depth (m)	Q, kg/cm²	Wo (%)	Wet Unit Weight (g/cc)	Geology
BH-1	27.50—27.60	262.15	2.7	2.59	Andesite
	69.50—69.70	197.21	3.2	2.56	Andesite
BV-2	26.1 —26.3	144.92	7.4	2.46	Andesite
BV-3	7.3 — 7.50	48.05	6.4	2.49	Andesite
	14.8 —15.0	151.61	6.1	2.51	Andesite
	30.7 —30.8	139.45	5.7	2.41	Andesite
	21.2 —21.5	90.85	8.8	2.54	Andesite
BV-4	30.0 —30.2	58.81	5.7	2.42	Andesite
BV-5	25.30—25.50	100.75	4.6	2.42	Andesite
BV-6	14.30—14.50	126.1	7.3	2.49	Andesite
	22.60—22.60	254.08	7.2	2.56	Andesite

The tests showed the andesites core samples with maximum compressive stress values between 100 to 250 kg/cm^2. These values are generally less than the expected compressive strength of the andesites. It is believed that this is due to the presence of natural fissures and fractures within the rock specimens.

4-5 Application of Investigation Results

4-5-1 Classification of Base Rock at Tunnel Formation

Generally, in Japan, the classification of base rock is based on the velocity of the P wave. Geological classification, fracture and fissure pattern and degree of weathering play a major role in the design of the tunnel.

The summary of the general and structural geology as well as degree of weathering are summarized in Table 4-5.1

TABLE 4-5-1 ROCK MASS AND ROCK CLASSIFICATION

Rock Mass Classification	Petrological Classification	Standards for Rock Mass Classification: Standard by Seismic Wave Velocity (Km/sec) 1.0 2.0 3.0 4.0 5.0 KM/S	Standards for Rock Mass Classification: Standards by Geological Conditions (conditions investigated by geological survey or excavation face)	Standards for Rock Mass Classification: Standard by Boring Core Sample (condition of core sample)	Standards by Observation: Hitting with Hammer	Standards by Observation: Interval of Cracks	Standard by Geological Condition (condition after excavation)
A	a, b, c		Rock mass is very hard and fresh and consists of massive blocks with scarce crack. Continuous and stable over the large area.	Core recovery is more than about 90% with complete column shape, having length of more than about 20cm, without containing small pieces.	Hammer is bounded. The rock is cracked with fresh surface only when hitted strongly.	50 cm or more	No plastic zone in ground. Height of looseness, less than 1.6 m.
	d, e		Rock mass is very hard, fresh, massive and hardly contains cracks.		The rock develops cracking or cut relatively largely along the joint or crack when hitted strongly.		
B	a, b, c, d, e		(1) Rock is fresh, hard and contains relatively less cracks. (2) Rock is relatively hard but shows some what altered property due to weathering. (3) The rock is hard but assumes a layer form having bedding or schistosity and tends to be cracked along the surface.	Core recovery rate is more than about 70% without complete column shape and concerning some small pieces. Core having diameter of about 5 cm or more can be obtained.		10 to 50 cm	Earth pressure due to plastic zone does not usually act. But may act due to crushed rock and ground water. Height of looseness, 1.5 to 3.0 m.
C	a, b, c, d, e		(1) Rock is altered due to weathering. (2) Rock is relatively hard but contains many small cracks thereby showing the appearance of small masses. Joint may contain clay deposit. (3) Bedding and schistosity are remarkable. Easy cleavage with thin layer. (4) Narrow, small sectional soil areas are contained.	Core recovery rate is roughly 40-70% containing many cracks. The rock also cracks easily to a mass of less than 10 cm.	Crushed easily with hammer. Crushed into small pieces along the cracked face. Faces without containing cracks are hardly crushed.	2 to 10 cm	Earth pressure due to plastic zone may often be exerted. Plastic range or height of looseness, 2.0 to 4.0 m.
D	a, b, c, d, e		(1) Considerably weathered rock mass, partly showing soiled part containing some hard parts. Soft rock mass having many small cracks. Rock can be crushed easily from any part other than crack. (2) Crushed rock zone in which tendency toward clay and fine pieces are being mixed. There is not much progressing. Clay may be contained some hard components. (3) Sandy soil, talus, etc.	Core recovery reduces down to less than 40%. The core may be composed of fine pieces or may be a rubble-sand state or fine-sand state.	Crushed easily if hitted by a hammer. Crushed easily by fingers.	—	Earth pressure due to plastic zone is exerted. Large eccentric earth pressure may sometimes be exerted. Plastic range or height of looseness, 3.0 to 6.0 m.
E	a, b, c, d, e		(1) Fault and crushed rock zone or talus with a considerable width, on which considerable eccentric earth pressure is acting.	—	—	—	Earth pressure due to plastic zone exerts. Considerable eccentric earth pressure is often exerted. Plastic range larger than 7.0 m.

Rock mass classification (1): Used for design of support and lining.
Petrological classification

a. Metamorphite (phyllite, graphite schist, silicic graphite schist, quartz schist, greenschist, gneiss, serpentine, hornfels, etc.)
Plutonite (gabbro, etc.)

b. Paleozoic strata and mesozoic formation (slate, sandstone and psephite, graywacke, limestone, quartzite, schalstein, etc.)

c. Volcanic rock (liparite, andesite, basalt, etc.)
Dike rock (quartz porphyry, granite, diabase, etc), Plutonite (granite, diorite, etc.)

d. Tertiary formation (clay pelite, silicic pelite, sandstone and psephite, limestone, tuff, breccia, agglomerate, etc.)

e. Diluvium (loam and clay, volcanic crushed formation, etc.)
Alluvium (talus, surface soil, etc.)

In this respect, the classification of rock at tunnel formation is as follows:

TABLE 4-5.2 CLASSIFICATION OF BASE ROCK AT THE TUNNEL FORMATION

Classification of Base Rock	Total Length	Percentage Ratio
A	0	
B	890	47.6%
C	750	40.1%
D	170	9.1%
Low velocity zone	60	3.2%
Total	1,870 m	

In accordance with Table 4-5.2, base rock at tunnel elevation consists of grade B and grade C rock. This condition may be considered fair.

4-5-2 Groundwater at Tunnel Formation

One of the major problems to be encountered during tunnel construction is the presence of substantial amounts of spring water and/or groundwater. Drainage of the tunnel area is of utmost importance during construction.

It is therefore necesary to obtain a good estimate of the quantity of spring water that may be expected at the tunnel elevation. However, it is difficult to estimate the actual quantity of spring water due to so many factors and assumptions that are made in the calculations.

It was therefore decided that the method generally used in Japan be used.

(1) Method of Calculation

For simplification, a flow chart is presented herein below describing the calculation procedure.

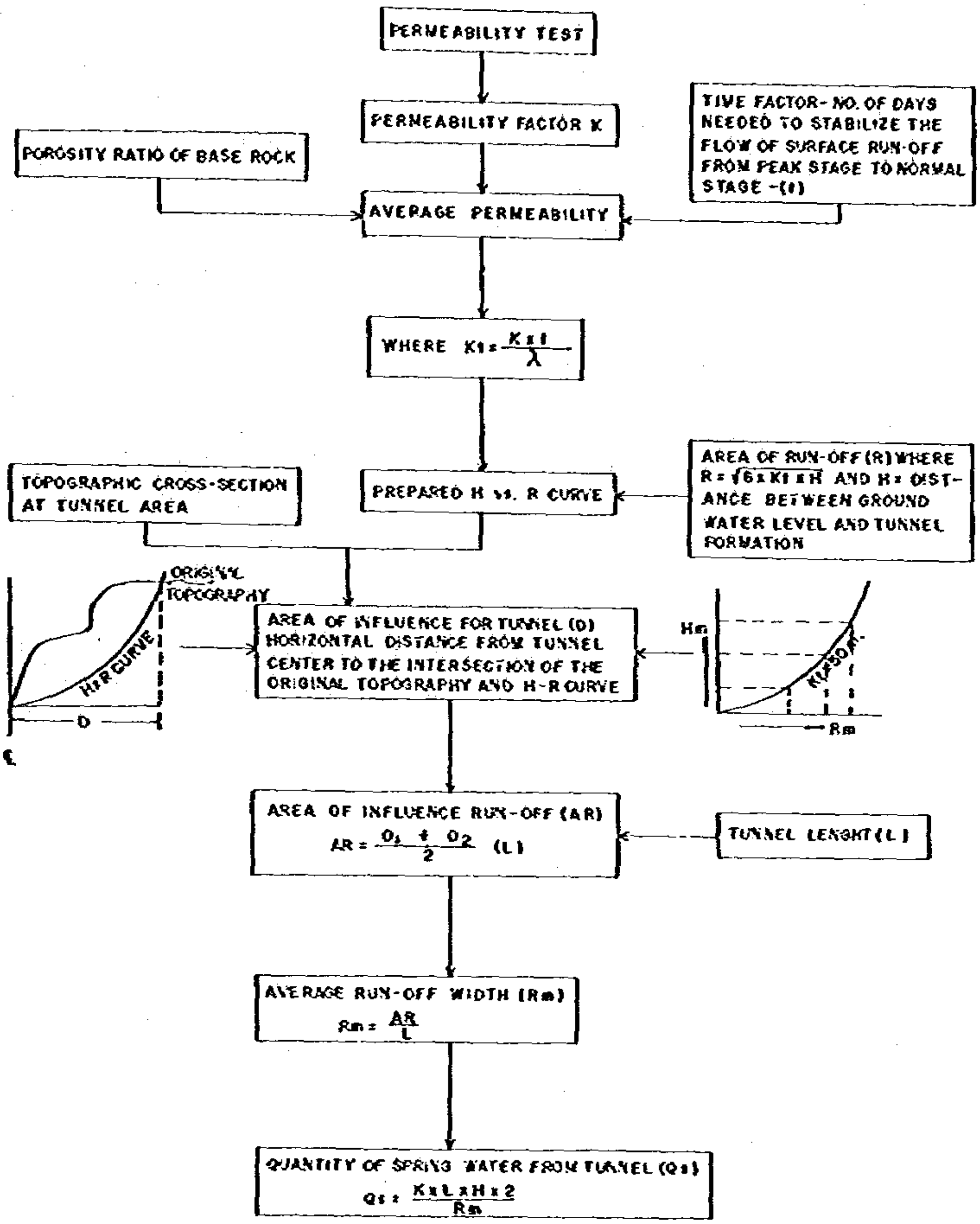


Fig. 4-5.1 FLOW CHART FOR CALCULATION OF SPRING WATER FROM TUNNEL

(2) Result of Calculation

1) Permeability factor (K)

The use of the calculations previously described are presented herein below in Table 4-5.3

TABLE 4-5.3 RESULT OF PERMEABILITY TEST OF EACH BORING

Boring No.	Test Depth	Geology	Permeability Factor
BV-1	10.0-20.0 m	Tuff Breccia	$K=1.545 \times 10^{-5}$ m/min
BV-2	20.0-30.0 m	Tuff Breccia	$K=1.27 \times 10^{-5}$
BV-3	30.0-40.0 m	Andesite	$K=5.75 \times 10^{-6}$
BV-4	25.0-35.0 m	Andesite	$K=4.646 \times 10^{-6}$
BV-5	20.0-30.0 m	Andesite	$K=1.12 \times 10^{-6}$
BV-6	15.0-25.0 m	Andesite	$K=1.147 \times 10^{-5}$

It can then be inferred that the permeability factor of the base rock is influenced by the geology, the degree of weathering and fracture patterns. Thus, estimating the permeability factor, it is best to classify its geology and find the average value of the permeability factor.

The average value of permeability factor on each rock are as follows:

Tuff Breccia $K = 1.4075 \times 10^{-5}$ m/min
Andesite $K = 8.2665 \times 10^{-6}$ m/min

2) Porosity of Rock

The base rocks encountered have the following porosity based on the core recovery and intact rock samples obtained.

Tuff Breccia $\lambda = 20\%$
Andesite $\lambda = 15\%$

3) Time requirements to increase the flow of river by rainfall will return to base flow (t)

$t = 25$ days

4) Results of calculations

Average permeability (Kt)

$$Kt = \frac{k \cdot t}{\lambda}$$

Tufff Breccia	Andesite
$k = 1.4075 \times 10^{-5}$ m/min	$k = 8.2665 \times 10^{-6}$ m/min
$\lambda = 20\%$	$\lambda = 15\%$
$t = 25$	$t = 25$

Tuff breccia $\quad Kt = \dfrac{1.4075 \times 25 \times 24 \times 60 \times 10^{-5}}{0.20}$

$= 2.53$ m

Andesite $\quad Kt = \dfrac{8.2665 \times 25 \times 24 \times 60 \times 10^{-6}}{0.15}$

$= 1.983$ m

Area of run off (R)

$$R = \sqrt{6 \times Kt \times H}$$

$Kt = 2.53$ m for tuff breccai
$Kt = 1.93$ m for andesite

Tuff breccia $\quad R = \sqrt{6 \times 2.53 \times H} = 3.9\sqrt{H}$
Andesite $\quad R = \sqrt{6 \times 1.98 \times H} = 3.4\sqrt{H}$

Using above value of R, rotation curve between H and R was drawn. Based on this tunnel length (L), the area of influence run off (AR) and average runoff width (Rm), quantity of spring water from tunnel (Qs) was calculated based on the following equation:

$$Qs = \frac{K \times L \times H^2}{Rm}$$

H : Distance between water level and tunnel formation

The calcualted amount of spring water is shown in Table 4-5.4 and Drawing GS-21

TABLE 4-5.4 RESULT OF CALCULATION FOR TUNNEL GROUND WATER

Section	AR (m^2)	L (m)	Rm (m)	K (m/min)	H (m)	Qs (m^3/min)
1	8,400	210	40	1.4075×10^{-5}	40	$0.012m^3$
2	39,775	430	93	8.2665×10^{-6}	100	$0.382m^3$
3	40,500	450	90	8.2265×10^{-6}	100	$0.413m^3$
4	18,975	330	58	8.2665×10^{-6}	55	$0.142m^3$
5	9,000	450	20	8.2665×10^{-6}	20	$0.074m^3$
Total		1,870				$1.023m^3$

Based on the above findings and those earlier described in this report, the proposed tunnel alignment will be passing through relatively fair geological condition and no major problems are expected to be encountered in the tunnel construction.

4-6 Evaluation of Tunnel Rock Condition

Based on the findings presented above, it is expected that the problem areas will be mainly concentrated on the south portal and in the fault zones based on geological condition. The rocks on the north portal, although significantly fractured, can be considered fair. The fragmented tuff breccia at the south portal will need extra care during tunnel construction due to its friability.

The volume of the ground or seepage water at the tunnel elevation, which was observed during the drilling operations, is not expected to pose a major problem during construction. Drainage of the tunnel site will be of utmost importance during construction operations. According to calculation on tunnel spring water, it is considered normal.

Seismic investigation of the tunnel alignment shows reasonably high seismic wave velocity particularly at the main tunnel alignment indicating fair geologic conditions.

Chapter 5.

ENGINEERING STUDY

CHAPTER 5
ENGINEERING STUDY

5-1 Introduction

The major theme for discussion in this chapter is the preliminary design of the "Most Likely Route". Preceding works were conducted such as selection of alternative routes, preliminary engineering studies of alternative routes and selection of the "Most Likely Route". Basic data and results obtained from geo-technical investigations and traffic surveys are reflected herein. These are discussed in detail under separate chapters.

Special attention was given to studying the *implementation program and construction* scheduling, considering that this is the first time a construction of a tunnel of this magnitude will be done in the Philippines.

Construction costs were estimated based on quantities and related works identified in the preliminary design phase. Analysis of unit prices of construction items and materials were also considered.

The maintenance costs for the existing "open" road sections have also been estimated based on current maintenance costs data from the MPWH.

The study is composed of nine basic phases namely:

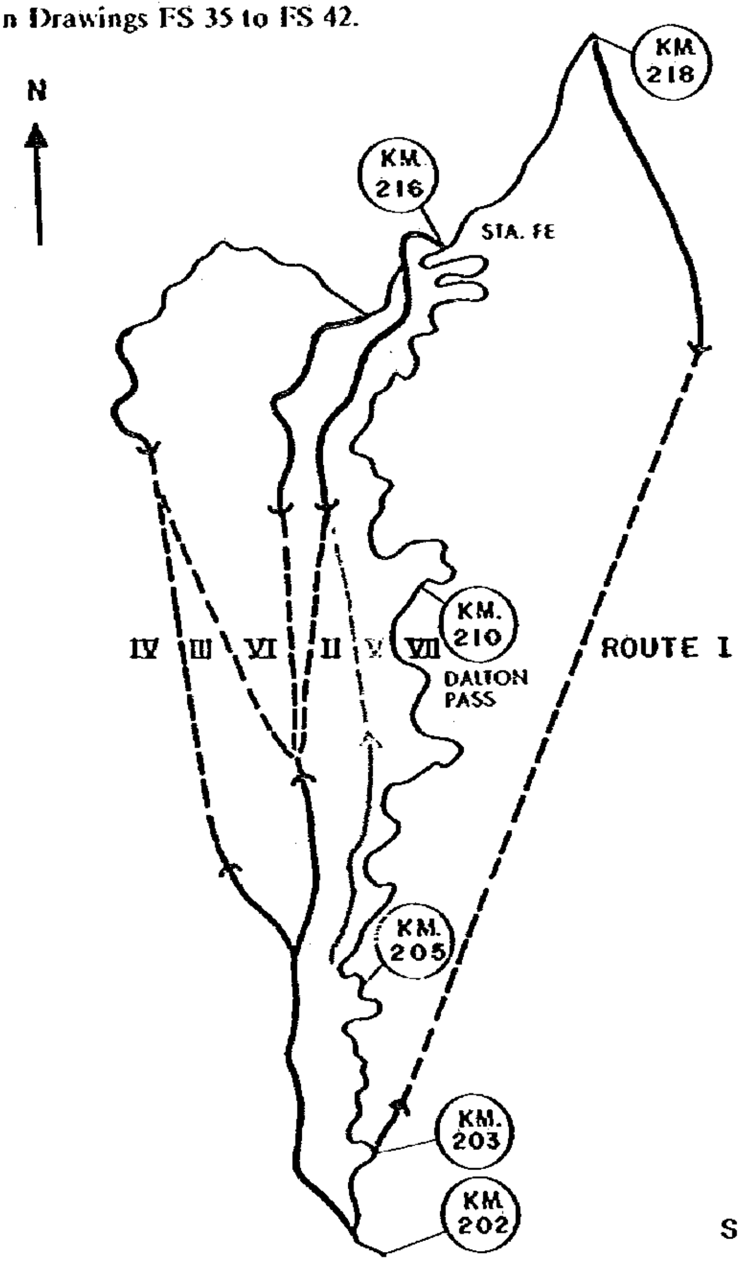
a) Data Collection
b) Reconnaissance
c) Selection of Alternative Routes
d) Preliminary Engineering Study of Alternative Routes
e) Cost Estimates of Alternative Routes
f) Selection of "Most Likely Route"
g) Preliminary Design of "Most Likely Route"
h) Construction Plan and Schedule
i) Construction and Maintenance Costs

5-2 Selection of the "Most Likely Route"

5-2-1 Study of Alternative Routes

Based on the reconnaissance and also on available data and information gathered, several probable alterative routes to Section A were identified and plotted on 1:50,000 scale topographic map.

All these alternative routes were planned within the area between Km. 202 and Km. 218 of the existing Route 5. The outlines of the alignments of these alternative routes are briefly described below and shown in Fig. 5-2.1. The profile of each alternative route is also shown in Drawings FS 35 to FS 42.



Scale : 1 : 50,000

Fig. 5-2.1 ALTERNATIVE ROUTE I TO VII

Route I : New alignment on the far eastern side of the existing road.

Route II : New alignment passing the west bank side of the Digdig River and the east bank side of Sta. Fe River.

Route III : New alignment passing the west bank side of the Digdig River and connecting with the existing forest road, using the western alignment approach of Route II.

Route IV : Modification of Route III, passing the farthest west side of the existing highway.

Route V : New alignment on the west side nearest and running almost parallel to the existing highway.

Route VI : Route passing in between Route II and Route III using the southern approach of Route II.

Route VII : Route considering a tunnel along the existing highway.

The general features of the above routes are shown in Table 5-2.1 The comparative analysis of the probable alternative routes were evaluated in detail in the basis of length, alignment, probable major structure, topography, geology, sabo analysis, and difficulty in construction. After the detailed examination, Routes II and V were selected for the succeeding preliminary engineering study. The basis for selection is summarized as follows:

Route I : The total length is the shortest of the alternative routes, about 9.68 km., but the tunnel length is the longest about 5.7 km., which is more than twice the length of the others. The construction and the maintenance costs on this route are estimated to be enormous because of the length of the tunnel.

Route II : Lengthwise, Route II is the second shortest route, consisting of about 10.17 km. and the tunnel is about 2.28 km., nearly equal to that of the other routes, except Route I. Moreover, the open section can be designed in such a way that expected disaster sites can be avoided. Thus, this route is selected for succeeding engineering study, being the route with relatively minimum encumbrances.

TABLE 5-2.1 GENERAL FEATURES OF ALTERNATIVE ROUTES

ITEM		ROUTES		I	II	III	IV	V	VI	VII	EXISTING ROAD
		DESCRIPTION		SEE PAGE							
		TOTAL LENGTH	(km)	9.68	10.17	12.43	12.36	10.17	10.60	12.87	16.00
LENGTH (KM)		NEW CONSTRUCTION SECTION	(Km)	8.58	6.97	6.03	6.56	4.57	6.90	2.80	0
	SECTION	TUNNEL SECTION	(Km)	5.70	2.28	2.80	3.66	2.00	2.07	2.12	0
		EARTHWORK SECTION	(Km)	2.26	3.89	3.08	2.15	2.07	3.88	0.78	0
		TO-BE-IMPROVED SECTION	(Km)	1.10	3.20	5.80	5.80	5.60	3.70	10.07	16.00
	SECTION	EXISTING NATIONAL HIGHWAY ROUTE 5	(Km)	1.01	2.80	2.80	2.80	5.20	2.80	10.07	16.00
		EXISTING FOREST ROAD	(Km)	0	0.40	3.00	3.00	0.40	0.90	0	0
ALIGNMENT		MAXIMUM GRADE (%)		7%	7%	7%	8%	7%	8%	7%	10%
STRUCTURES		LONG BRIDGE (SITES)		1 (300 m)	9 (700 m)	6 (600 m)	5 (600 m)	3 (400 m)	6 (800 m)	2 (330 m)	0
		MEDIUM AND SHORT BRIDGE (SITES)		3 (120 m)	4 (100 m)	6 (150 m)	5 (150 m)	3 (100 m)	4 (150 m)	2 (75 m)	0
TOPOGRAPHY AND GEOLOGY		DISTRIBUTION OF LANDSLIDE SECTION		NONE	FEW	FEW	FEW	SOME	FEW	MUCH	
		FAULT & SHEARED ZONE		LESS	AVERAGE	AVERAGE	AVERAGE	MUCH	AVERAGE	MUCH	
		FAILURE		FEW	SOME	SOME	MANY	SOME	MANY	MANY	
		LENGTH PASSING THROUGH SOFT GROUND AREA		NONE	FEW	FEW	FEW	NONE	FEW	NONE	
		TOPOGRAPHY OF NATURAL SLOPE		MOUNTAINOUS	MOUNTAINOUS TERRACE	MOUNTAINOUS TERRACE	MOUNTAINOUS TERRACE	MOUNTAINOUS	MOUNTAINOUS TERRACE	MOUNTAINOUS	
		GEOLOGY AND GEOLOGICAL STRUCTURES		DIABASE	ANDESITE	ANDESITE	ANDESITE	DIABASE & ANDESITE	ANDESITE	DIABASE	
TUNNEL		TOPOGRAPHY & GEOLOGY AT PROPOSED SITE OF TUNNEL		RIDGE DIABASE	RIDGE TALUS, ANDESITE	RIDGE ANDESITE	RIDGE TALUS, ANDESITE	RIDGE TALUS, ANDESITE	RIDGE	RIDGE	
		DIFFICULTY IN CONSTRUCTION		AVERAGE	AVERAGE	AVERAGE	AVERAGE	AVERAGE	AVERAGE	DIFFICULT	
SABO		PROBLEMS ON RELATIONSHIP WITH RIVER		FEW	SOME	SOME	SOME	SOME	SOME	SOME	
		PROBLEMS OF CONDITIONS OF CROSSING STREAM & CREEK		AVERAGE	MANY	MANY	MANY	MANY	MANY	MANY	
		EXPECTED DISASTER SITE		FEW	SOME	SOME	SOME	MANY	SOME	MANY	
		EVALUATION		BAD	FAIR	VERY BAD	VERY BAD	FAIR	BAD	BAD	
		REMARKS		THE LANDSLIDE IN ROUTE V DISTRIBUTED ALONG THE EXISTING HIGHWAY IS EXPECTED TO BE PREVENTED BY APPROPRIATE COUNTER MEASURES.							

Route III : The total length of this route is the second longest, about 12.43 km. and the tunnel is also fairly long, about 2.8 km. Moreover, the occurrence of disasters is expected along the existing forest road which is to be improved.

Route IV : The total length is almost the same as Route III, being the second longest of the alternative routes. The tunnel is also the second longest, about 3.8 km.

Route V : The total length of this route is second to the shortest of the alternative routes, about 2.0 km. Inspite of the fact that the occurrence of disasters is expected on the approach section of the south side of the tunnel, this route is worth considering for subsequent engineering study because its tunnel is the shortest.

Route VI : Both the tunnel and the total road length of this route are relatively short, about 10.6 km. and 2.07 km., respectively. However, there are many expected disaster areas along the approach section of the north side of the tunnel. The countermeasure works are expected to be very difficult.

Route VII : Inspite of the tunnel length being relatively short, about 2.13 km. which is nearly the same as that of Routes II and V, the total length is the longest of the alternative routes. This route is therefore deleted.

5-2-2 Probable "Most Likely Routes"

Through the preliminary study of alternative routes discussed in Section 5-2-1, Routes II and V were identified from studies based mainly on the Scale 1:50,000 map upon which the Most Likely Route is selected. Based on the Scale 1:5000 map, these routes were modified or refined with the addition of three more routes (Routes II', II'' and II+V). The routes are further described as follows and shown in Fig., 5-2.2 and Drawing FS-34.

1) Route II

Route II begins at the eastern side of Digdig River at Km. 202 and diverts from the existing highway at Km. 202+500. The total length of this route is 10.16 km. including 10 bridges of approximately 800 meters and tunnel length of 2880 meters generally located at the mid-section of the route. The section up to the tunnel is generally made up of half-cut and half-embankment. On the other hand, the route after the end of the tunnel consists mainly of cut section. It is proposed that the existing forest road after Sta. Fe River be improved and widened to connect to the existing highway in Sta. Fe.

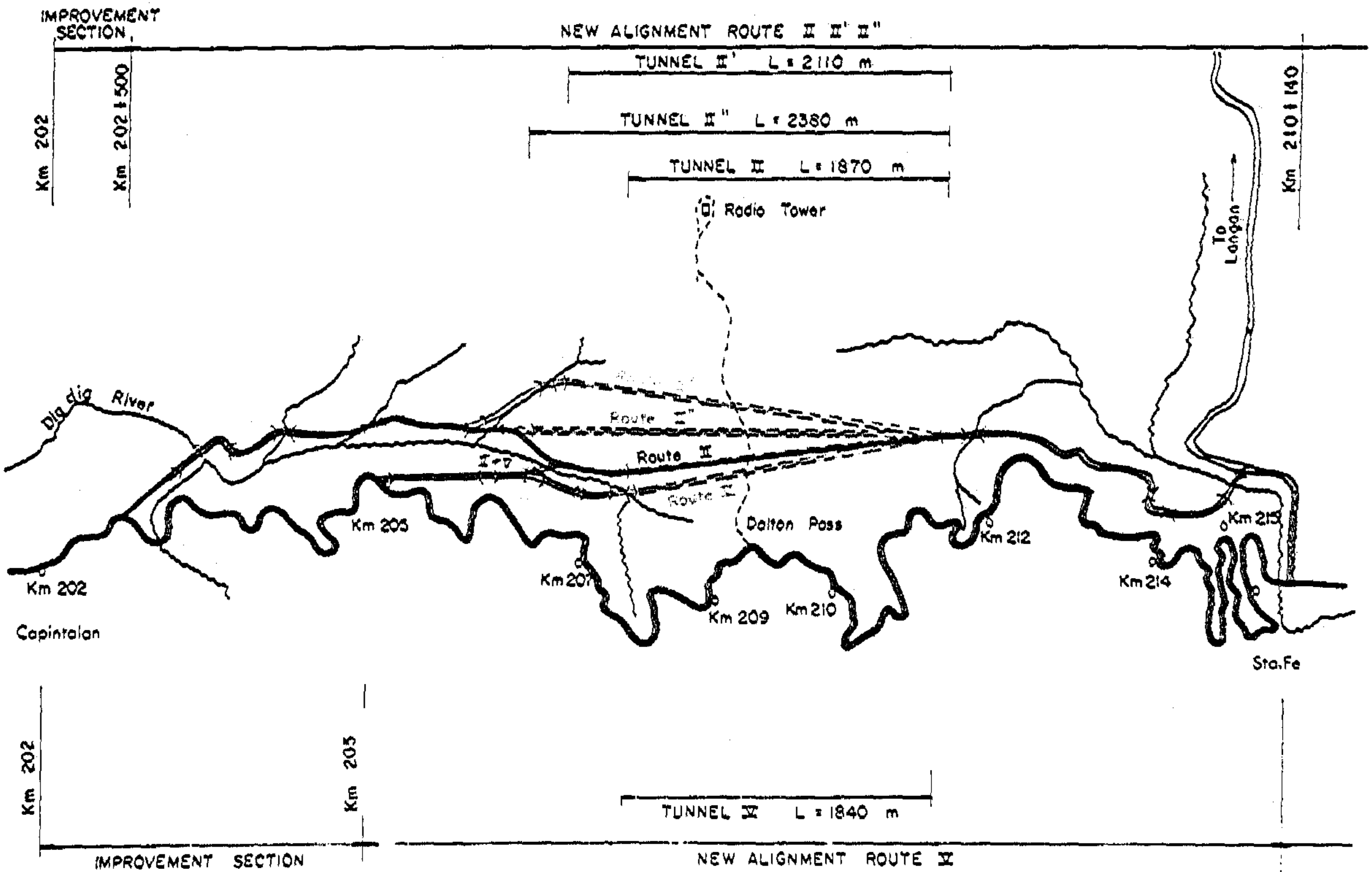
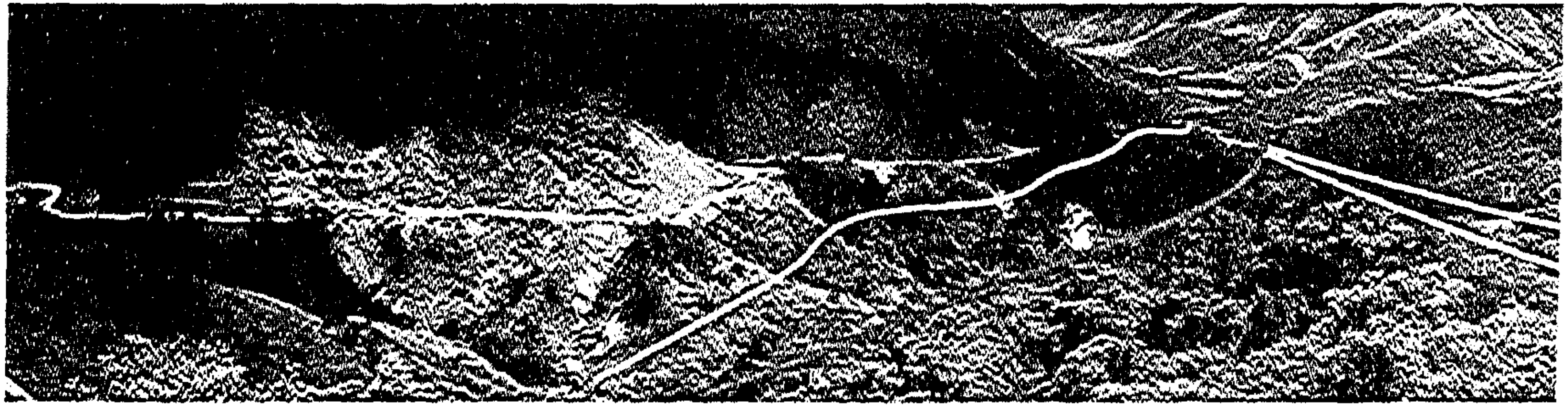
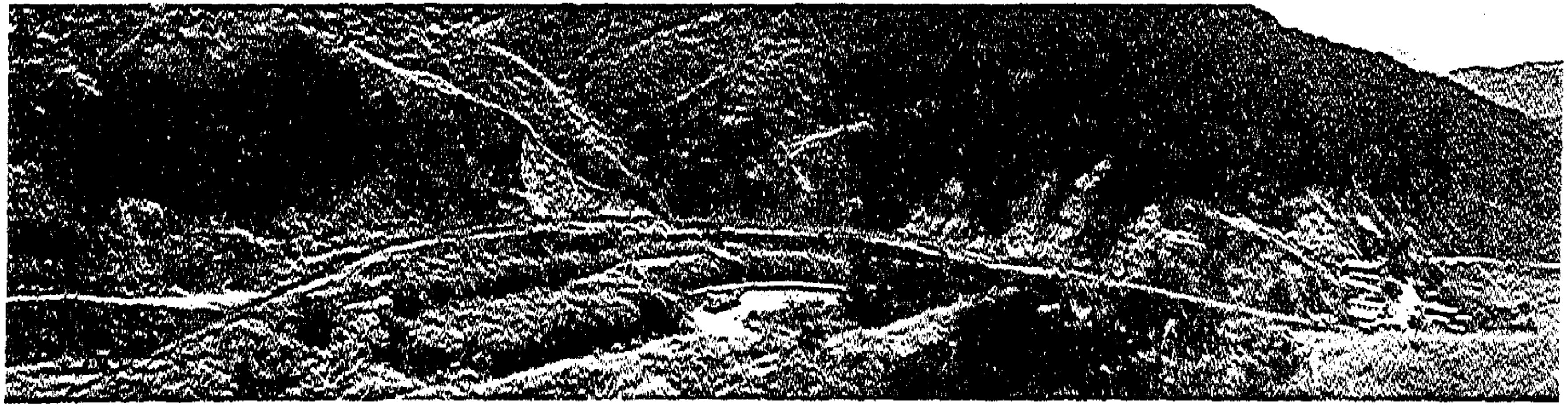


Fig. 5-2.2 GENERAL ALIGNMENT



TUNNEL APROACHES. Upper photo shows the tunnel approach from the Manila side. Superimposed on darkened lines are the four routes herein indicated. Corresponding alignments are herein seen. On the other hand, lower photo is a panoramic view of the Dalton Pass Tunnel approach from the Sta. Fe side with the town and existing Route 5 seen at far right. The proposed route is indicated with darkened line.



2) Route V

Route V proposes to utilize the existing highway with some improvements from Km. 202 to 205, after which a new alignment is designed parallel to the existing highway up to the tunnel. Route II and V are identical in the sections after the end of the tunnel. This route is 10.65 km. in total length including the bridges of about 1,035 m. and the tunnel of about 1,860 m. The route up to the tunnel is generally made up of the cut section.

3) Route II'

Route II' is the modified version of Route II. This route is designed as the new alignment section between Km. 205 and Km. 207+500 of Route II. The total length is 10.13 km. including bridges of about 760 m. and the tunnel of about 2,110 m. The section on the north side of the tunnel uses the same route as Route II. In the south side of the tunnel however, the route is designed shifting westward to avoid the section along Digdig River.

4) Route II"

Route II" is designed modifying Route II. The tunnel of this route starts from Km. 205+220 of Route II, therefore becoming the longest of the alternative routes. By this modified alignment however, the disasters which are expected in the open section will be avoided. The total length of this route is 10.06 km. involving bridges of 795 m. and the tunnel of 2,380 m.

5) Route II + V

Route II + V is planned as the route connecting Route V with Route II by a bridge crossing Digdig River in the south side of the tunnel to keep away from the section of Route V, along Digdig River wherein a sizeable landslide exists. The total length is 10.88 km. including the bridges of 990 m. and the tunnel of 1,870 m.

5-2-3 Preliminary Engineering Studies

The selected alternative routes were studied in detail adopting standard engineering methods of investigations and analysis permissible within the scope of the project. In general, the engineering studies focused on the following items:

- Extent of road and/or tunnel to be constructed
- Geological condition and topographical features around tunnel portal
- Long span bridges and other major structures
- Embankment and steep slopes
- Problem section for particular attention
- Difficulty of construction

The engineering studies on the selected routes described were carried out in the following manner for each item:

1) Earthwork

Based on 1:5000 scale map, cross sections were made to estimate the embankment and cut volumes. The cross sections were drawn at about 200 m. interval mainly selecting sag and crest points of the road profile designed in advance.

2) Slope Protection

For the embankment slope, the seeding with topsoil of 10 cm. thick was adopted. Slope protection by planting trees and grass was planned and cut slopes to prevent erosion.

For the cut slopes, consisting of weathered rocks, the concrete spreading or the precast concrete frame were applied corresponding to their geological features.

The structures of the slope protections by planting and the precast concrete frame adopted are illustrated in Drawings Volume.

3) Minor Structures

The standard design of retaining walls was studied based on the results of stability calculations, dividing two types of walls: gravity wall and inverted T section wall. Heights of walls studied are indicated below while the standard types are shown in Appendix E.

Gravity wall : each meter from 1.0 m to 5.0 m
Inverted T Section Wall : each meter from 3.0 m to 7.0 m

Using the results derived from aforementioned studies, the most economical type wall was mainly applied to sections in which road width were difficult to obtain.

Standard types of stone masonry were determined referring to types which are commonly used in road construction in the Philippines and Japan. These are shown in Appendix E. The stone masonry was mainly adopted to prevent the toe of embankment in the sections adjacent to Digdig River and Sta. Fe River from scouring caused by stream flow.

4) Cement Concrete Pavement

The structure of cement concrete pavement was determined referring to the structure of the existing highway in the project area. The structure is as follows:

Cement concrete surface	23 cm.
Crushed stone base	20 cm.

5) Drainage

Concrete side ditch (0.5 m x 0.5 m) was placed at the toe of cut slope in whole section as shown in typical cross section.

Pipe culvert (diameter 1.2 m) and box culvert (1.5 m x 1.5 m) were temporarily set by the following interval in this stage without discharge estimation.

Pipe culvert	about 150 m.
Box culvert	about 300 m.

6) Tunnel

The lining thickness was designed to be 45 cm. at sections with good geological conditions and 60 cm. at sections with poor geological conditions.

The ventilation system should be selected based on the required amount of fresh air. However, since at the time this work was undertaken, it has not yet been determined, the shaft type was adopted temporarily for the ventilation system, based on tunnel length.

Required lighting facility was roughly designed from the design speed and the kind of pavement in the tunnel based on Japanese experience.

Distribution lines to supply the electric power for maintenance of the tunnel was assumed to be 40 km. in length.

7) Bridges

Standard design of bridges were studied prior to adopting the actual bridge design.

For the super-structure, several types of girder with various spans were studied and the standard design of each type was determined as follows:

RCDG (Reinforced Concrete Deck Girder)	10m, 12m, 13m, and 15m span
PCG (Pre-stressed Concrete Girder)	20m, 25m, 30m, and 35m span
ST (Steel Truss)	40m, 50m, 60m, and 70m span
PCSG (Pre-stressed Concrete Segmentary Girder)	40m, 50m, 60m, and 70m span

For the sub-structure the abutments and piers of following heights were checked corresponding to the super-structure to be applied.

Abutment	5m, 7m, 9m and 11m high
Pier	10m, 15m, 20m, 25m, 30m and 35m high

After the combined use of standard design was determined, the most economical type of bridge was selected, conforming to field conditions such as topographic and geological conditions.

8) Sabo (Erosion and Sediment Control)

Major works of sabo design are channeling work of Sta. Fe River and sabo dam construction in Digdig River.

The channeling work of Sta. Fe River was planned in order to obtain road width in the forest road improvement section, and to provide an area for dump surplus materials derived from the tunnel excavation. This work will also contribute in protecting the Municipality of Sta. Fe from the disaster caused by the Sta. Fe River overflow.

The sabo dam of Digdig River was planned to prevent serious erosion on both banks of the Digdig River. By this work the safety of the planned route can be maintained.

9) Improvement of Existing Highway

In this item, overlay work using asphalt concrete and improvement of existing drainage system were planned.

Construction Quantities and Cost Estimates

Construction quantities and cost estimates of selected routes were estimated for comparative study on the basis of the quantities of major work items and the corresponding unit costs, which were identified under the preliminary engineering studies.

The major work items were categorized as follows:

a) Earthworks
b) Slope Protection
c) Minor structures
d) Pavement
e) Drainage
f) Tunnel
g) Bridge
h) Sabo
i) Improvement of existing highway

The unit costs for all the items except tunneling work were based, generally adopting experiences of the similar projects in Luzon: Laoag-Allacapan Road Project, Circumferential Road No.3 Project and National Transport Planning Project as of December 1980.

No construction cost data for similar tunneling projects is available in the Philippines. Thus, based on the experiences undertaken by the Ministry of Construction and Japan Highway Public Corporation in Japan and considering the availability of labor and materials procured locally, the conversion factors to the costs in the Philippines are estimated and finally, the costs adopted for this project are determined. For reference, major items which are locally available and imported are shown as follows:

Items Available in the Philippines	Items to be Imported
	Engineers
Unskilled labor	Skilled labor for tunnel work
Mechanical technician (50%)	Mechanical technician (50%)
Electrical technician (50%)	Electrical technician (50%)
Explosives	Support materials (H-beam)
Timber (sheet pile)	Rod bit
Concrete (cement, aggregates)	
Concrete pipe	Major construction equipment for tunnel
Construction field office	Steel form
	Materials for electrical and emergency facilities

Table 5-2.2 shows the construction costs adopted in Japan for the cost estimate at the preliminary engineering phase in 1981 along with the adjusted costs adopted for this study.

The conversion factor (in this case 1.35) to the Philippine prices was estimated in a manner shown in Table 5-2.3. Conversion factors adopted for each work item was computed in the same manner as the above.

The estimated quantities and constuction costs of each selected routes are summarized in Table 5-2.4.

TABLE 5-2.2 CONSTRUCTION COSTS OF EXCAVATION (TOP HEADING)

Rock Classification	Costs Adopted by Ministry of Construction in Yen	Costs Adopted by Japan Highway Public Corp. in Yen	Costs Adopted As Base for this Study in Yen	Costs Adopted For This Study (× 1.35)[1] in Yen	Costs in Pesos (P1 = ¥28) in Pesos
A	—	—	10,700	14,400	515
B	10,700	10,300	10,300	13,900	495
C	10,900	11,500	10,900	14,700	525
D	12,200	12,000	12,000	16,200	550

[1]The conversion factor to Philippine costs as computed in Table 5-2.3.

TABLE 5-2.3 EXCAVATION COST FOR TOP HEADING (ROCK CLASSIFICATION C)

Assumed Progress: 2 Shifts a day 3.6 m, 1.8 m Per Site
Assumed Excavation Volume: 42.3 $m^3/m \times 1.8$ m = 76.14 m^3/m/Site

PER 10 WORKING HOURS PER SITE IN YEN

ITEM	UNIT	QUANTITY	COSTS IN JAPAN UNIT PRICE	COSTS IN JAPAN COST	ESTIMATED COSTS IN PHILIPPINES UNIT PRICE	ESTIMATED COSTS IN PHILIPPINES COST
MATERIALS:						
Dynamite No. 2 Sakura 0.8kg/m^3	kg	54.6	850	46,410		46,410
Electric Detonator DS D 1-10 Step	unit	163.8	160	26,208		26,208
Rod Bit	unit	1.7	11,500	19,550		38,000
Sheet Pile t =30	m^3	0.85	54,000	45,900	27,000	22,950
Rent of Tools, Misc Materials	set	1		4,142		4,142
LABOR:					(Allowance 550)	
Skilled Labor for Tunnel	persons	7	23,000	161,000	35,000	295,000
Labor	persons	4	18,500	74,000	12 persons × 1,000	12,000
EQUIPMENT:						
Rent of Equipment 6-Boom Jumbo	unit	1	95,000	95,000		192,500
Operating Cost of Equipment						
Dump Truck 1.6 t	hr	10	11,000	110,000		185,000
Dump Truck 11 t	hr	30	8,000	240,000		324,500
Rent of Equipment Coal Pick	unit	1	110	110		200
Breaker 30 kg	unit	1	330	330		594
Winch	unit	1	2,200	2,200		3,960
MISCELLANEOUS	set	1		4,000		4,000
TOTAL			10,900	828,850 (÷76.14) =10,886	15,200	1,155,454 (÷76.14)

Therefore: the conversion factor to Philippine costs is computed as
15,200 ÷ 10,900 = 1.39 ≒ 1.35

TABLE 5-2.4 COMPARISON OF QUANTITIES AND CONSTRUCTION COST OF SELECTED ROUTES (P1000)

as of Jan. 1991

DESCRIPTION	Units	II Quantity	II Cost	II' Quantity	II' Cost	II'' Quantity	II'' Cost	V Quantity	V Cost	II+V Quantity	II+V Cost	Remarks
1. Earthwork			20,319.0		20,624.0		20,815.0		16,320.0		16,884.0	
Surplus Material of Excavation	m^3	532,330	12,563.0	478,651.0	11,282	536,647.0	12,665.0	558,090.0	13,170.0	513,450.0	12,110.0	
Embankment	m^3	245,035	8,356.0	275,725.0	9,402.0	239,005.0	8,150.0	92,370.0	3,150.0	140,000.0	4,774.0	
2. Slope Protection			8,663.0		7,942.0		7,859.0		4,948.0		1,638.0	
Planting	m^2	72,327	2,188.0	77,614.0	2,097.0	67,822.0	2,014.0	42,627.0	1,653.0	24,175.0	358.0	
Structure	m^2	23,555	6,475.0	21,365.0	5,845.0	21,305.0	5,845.0	12,135.0	3,295.0	5,000.0	1,280.0	
3. Minor Structures			7,918.0		7,433.0		7,433.0		6,828.0		6,367.0	
Retaining Wall	m	290.0	7,140.0	695.0	6,655.0	695.0	6,655.0	325.0	6,528.0	325.0	6,305.0	
Stone Masonry	m	630.0	778.0	630.0	778.0	630.0	778.0	0	0	150.0	62.0	
4. Cement Concrete Pavement	m	7,450.0	10,681.0	7,450.0	10,639.0	7,345.0	10,459.0	5,192.5	7,415.0	5,440.0	7,768.0	
5. Drainage		8,379.0	6,818.0		6,249.0		6,004.0		4,264.0		4,617.0	
Pipe Culvert	m	583.0	765.0	555.0	728.0	516.0	677.0	348.0	456.0	318.0	417.0	
Box Culvert	m	252.0	868.0	242.0	826.0	223.0	768.0	161.0	550.0	161.0	554.0	
Others	m	5,804.5	5,185.0	5,328.0	4,655.0	5,127.0	4,559.0	3,742.0	3,258.0	4,127.0	3,576.0	
6. Tunnel	m	1,870.0	233,000.0	2,110.0	266,000.0	2,380.0	291,000.0	1,840.0	230,000.0	1,870.0	233,000.0	
Main Work			134,000.0		152,600.0		171,000.0		133,000.0		134,000.0	
Traffic Safety and Control			88,000.0		103,000.0		109,000.0		86,000.0		88,000.0	
Distribution Line			11,000.0		11,000.0		11,000.0		11,000.0		11,000.0	
7. Bridge	m	760.0	21,554.0	760.0	22,097.0	795.0	23,025.0	1,315.0	53,010.0	965.0	43,029.0	
Long Span (L>50m)	m	675.0	19,223.0	635.0	18,115.0	710.0	20,294.0	85.0	50,279.0	85.0	40,298.0	
Short and medium Span (L≤50m)	m	85.0	2,731.0	125.0	3,982.0	85.0	2,731.0	1,270.0	2,731.0	960.0	2,731.0	
8. Sabo	each	6	20,000.0	6	20,000.0	6	20,000.0	6	29,000.0	6	24,000.0	
9. Improvement			1,376.0		1,376.0		1,376.0		13,954.0		13,554.0	
Overlay	m	500.0	375.0	500.0	375.0	500.0	375.0	3,000.0	1,875.0	3,000.0	1,875.0	
Others	m	1,920.0	1,001.0	1,920.0	1,001.0	1,1920.0	1,001.0	1,930.0	12,129.0	1,929.0	12,129.0	
10. Direct Cost			331,329.0		362,420.0		388,001.0		356,749.0		347,267.0	Total 1 to 9
11. Miscellaneous Minor Work			49,699.0		54,363.0		58,200.0		53,512.0		52,093.0	10×0.15
12. Sub Total			381,028.0		416,783.0		446,201.0		410,261.0		399,360.0	10+11
13. Right-of-Way			754.0		682.0		560.0		622.0		576.0	
14. Sub Total Cost			381,782.0		417,465.0		446,761.0		410,883.0		399,956.0	12+13
15. Design & Supervision			53,344.0		58,445.0		62,569.0		57,524.0		55,994.0	12×0.14
16. Total Cost			435,126.0		475,910.0		509,330.0		468,407.0		455,950.0	14+15

5-2-4 Selection of "Most Likely Route"

On the basis of preliminary engineering studies and cost estimates conducted in the preceding sections, the "Most Likely Route" was selected.

The selection was made on the basis of comparative analysis considering factors such as topographical and geological conditions, technical features, construction difficulties and construction costs.

Features of the main factors and items of each route are shown in Table 5-2.5.

The results are summarized as follows:

1) Total lengths of Route V and II+V are somewhat longer than other selected routes — about 10.8 km. The others are almost the same, about 10.1 km.

2) Tunnel lengths of Route II' and II'' are slightly more than 2000 m. The others are almost the same, about 1,800 m.

3) No difference on the alignment of each route is noticed.

4) Route II, II' and II'' pass through moderate open section area at 50% or 60% ratio to whole section. On the other hand, Route V and II+V pass through a flat area of only about 20%.

5) Comparatively more collapsed or landslide sites exist along Route V and II + V in terms of size and number. It is expected that it will be difficult to provide countermeasures in these routes.

6) Geological features of the tunnel section are almost the same.

On the basis of the above-mentioned facts, Route II was selected as the "Most Likely Route" for the following reasons:

1) The length of the tunnel is about 1.8 km.

2) It will avoid a sizeable existing disaster between Batiling and Sta. Fe.

3) Topographical feature of the open section is on a flat plane and gentle slope; thus, difficulty in construction is less than that of the other routes.

4) The construction cost of Route II is cheapest.

TABLE 5-2.5 FEATURES OF EACH ROUTE

ITEM		ROUTE II	II'	II"	V	II+V
LENGTH						
Total Length	(m)	10,160.0	10,130.0	10,060.0	10,882.5	10,850.0
New Construction Section	(m)	8.240.0	8.210.0	8,140.0	6,462.5	6,430.0
Tunnel	(m)	1,870.0	2,110.0	2,380.0	1,840.0	1,870.0
Cut	(m)	1,992.0	1,879.0	1,912.0	1,552.0	1,647.0
Embankment	(m)	1,797.5	2,090.0	1,750.0	1,112.0	1,090.0
Cut and Embankment	(m)	1,820.5	1,371.0	1,303.0	678.0	833.0
Bridge	(m)	760.0	760.0	795.0	1,270.0	990.0
Short & Medium Span	(m) each	85.0 (3)	125.0 (4)	85.0 (3)	85.0 (3)	85.0 (3)
Long Span	(m) each	675.0 (7)	635.0 (7)	710.0 (7)	1,185.0 (9)	905.0 (6)
Section to be Improved	(m)	1,920.0	1,920.0	1,920.0	4,420.0	4,420.0
National Highway	(m)	1,920.0	1,920.0	1,920.0	4,420.0	4,420.0
Forest Road	(m)	(450.0)	(450.0)	(450.0)	(450.0)	(450.0)
Forest Road is included in the New Construction Section						
ALIGNMENT						
Open Section						
Minimum Radius of Curve	(m)	R=50	R=50	R=50	R=50	R=50
Curve Section R≤50 m	(m)	1,000	950	1,000	2,100	2,150
Maximum Gradient	(%)	7	7	7	7	7
Steep Gradient i≥5%	(m)	2,250	1,850	2,100	2,400	2,400
Tunnel Section						
Minimum Radius of Curve	(m)	R=∞	R=2,000	R=∞	R=∞	R=∞
Gradient		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
TOPOGRAPHIC FEATURE						
South Side Open Section	(m)	3,880	3,500	3,220	4,582.5	4,520
Moderate	(m)	2,900	2,500	2,700	200	2,00
Steep	(m)	930	1,060	520	4,382.5	4,360
North Side Open Section	(m)	4,460	4,460	4,460	4,460	4,460
Moderate	(m)	2,330	2,330	2,330	2,330	2,330
Steep	(m)	2,330	2,330	2,330	2,330	2,330
Tunnel						
South Portal		stream	ridge	ridge	ridge	ridge
North Portal		talus	talus	talus	talus	talus
GEOLOGICAL FEATURE						
South Side Open Section						
Geology		andesite	dacite	andesite	andesite	andesite
Collapsed or Landslide Site	(each)	4	2	3	6(3)*	4(3)*
Number of Faults Crossed		7	6	5	12	12
Crossing Angle (30°-90°)		7	4	5	10	10
Crossing Angle (0°-30°)		0	2	0	2	2
North Side Open Section						
Geology		granite	granite	granite	granite	granite
Collapsed or Landslide Site	(each)	3	3	3	3	3
Number of Fault Crossed		3	3	3	3	3
Crossing Angle (30°-90°)		3	3	3	3	3
Crossing Angle (0°-30°)		0	0	0	0	0
Tunnel						
Geology		andesite	andesite	andesite	andesite	andesite
Number of Fault		2	3	3	1	1
Crossing Angle (30°-90°)		2	3	2	1	1
Crossing Angle (0°-30°)		0	0	1	0	0
Number of Valleys						
Crossing Angle (30°-90°)		4	4	4	4	4
Crossing Angle (0°-30°)		1	1	1	1	1

*number on existing highways

5-3 Preliminary Design of "Most Likely Route"

5-3-1 Geometric Design Standards

(1) Design Standards

To pursue the succeeding preliminary design works, the design standards to be adopted were first examined and determined.

The Ministry of Public Works and Highways at the moment has not yet established the common geometric design standard but applies road standard on a project to project basis. Some of the major standards adopted are as follows:

* AASHTO Design Standards
* Manual of Highway Design for Philippine Highways
* National Highway Progress and Future Development Plans of the Philippine Geometric Standards - IBRD
* Guidelines for Access Roads leading to the Philippine-Japan Friendship Highway
* The Philippine-Japan Highway Loan Project-Phase II (Laoag-Allacapan Road Project)

The geometric design standards adopted for the PJHL-Phase II, Laoag-Allacapan Project were prepared on the basis of the standard for IBRD projects and which others were previously adopted considering Japanese highway design standards. Since the features and characteristics of this project site under study resembles those of the mountainous sections of Laoag-Allacapan Road, it was judged as adequate to adopt the standards of the said project for the Dalton Pass Tunnel Project.

1) Roadway

The geometric standard adopted for this study is shown in Table 5-3.1.

2) Bridges

The design standards of bridges adopted for this study is fundamentally based on the standard specification for highway bridges, AASHTO - 1977. However, slight modifications were made for practical and convenient reasons without altering the specifications.

Dead Load

The following weights were assumed in computing the dead loads.

	$\#/ft^3$	t/m^3
Concrete (plain or reinforced)	150	2.40
Pavement	150	2.40
Compacted sand, earth, gravel or ballast	120	1.90

Deadload of pavement is taken as $50\#/ft^2$ (assuming 10 cm. concrete pavement).

3) Drainage Criteria

The Rational Method of peak flow curves was adopted for this study with the design storm frequency of 25 years.

4) Tunnel

Considering the characteristics and features of Route 5 at the Dalton Pass section, this section was classified as Category 3-Grade 3 by Japanese Road Standards. Consequently requirements based on this classification was fulfilled.

(2) Typical Cross Section

1) Typical Cross Section for Roadway

Based on the design standards discussed, the typical cross sections were determined as shown in Fig. 5-3.1 and Fig. 5-3.2. Especially in the mountainous sections, reduction of shoulder width, and installation of drainage structures and safety facilities such as guard rails are considered.

2) Typical Cross Section for Tunnel

Clearance for Tunnel

The roadway width on normal open section is 11.7 m. with 6.7 m. carriage way and 2.5 m. shoulder on both sides.

TABLE 5-3.1 DESIGN STANDARD

ITEM \ TERRAIN	FLAT	ROLLING	MOUN-TAINOUS	EMBANKMENT SECTION
1. Design Speed (Km/h)	70	60	50	30
2. Pavement width (m)	6.7	6.7	6.7	6.7
3. Shoulder width (m)	2.5	2.5	1.0 to 2.0	1.0 to 2.0
4. Right-of-Way (m)	20 to 30	30	*40	*40
5. Non-passing Sight Distance	90	80	60	30
6. Minimum Radius (m)	170	120	80	30 (*15)
7. Maximum Superelevation (%)	10	10	10	10
8. Maximum Grade (%)	3	5	10	10
9. Minimum Length of Vertical Curve (m)	60	60	60	30
10. Minimum K for Crest Vertical Curve (m)	1500	1200	1000	300
11. Minimum K for Sag. Vertical Curve (m)	1500	1000	800	300
12. Vertical Clearance for Superstructures	4.80	4.80	4.8	4.80

NOTE 1 =*Variable

2 = For hair-pin curves, a radius of 15m. is allowable, exeptionally

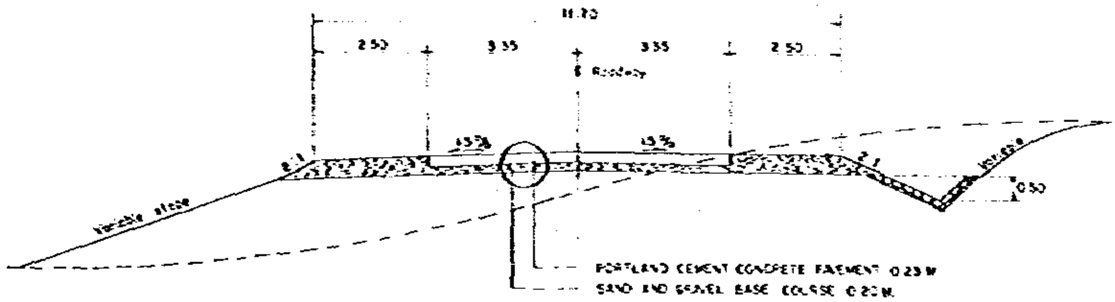


Fig. 5-3.1 TYPICAL CROSS SECTION FOR FLAT AND ROLLING SECTION
SCALE 1:100

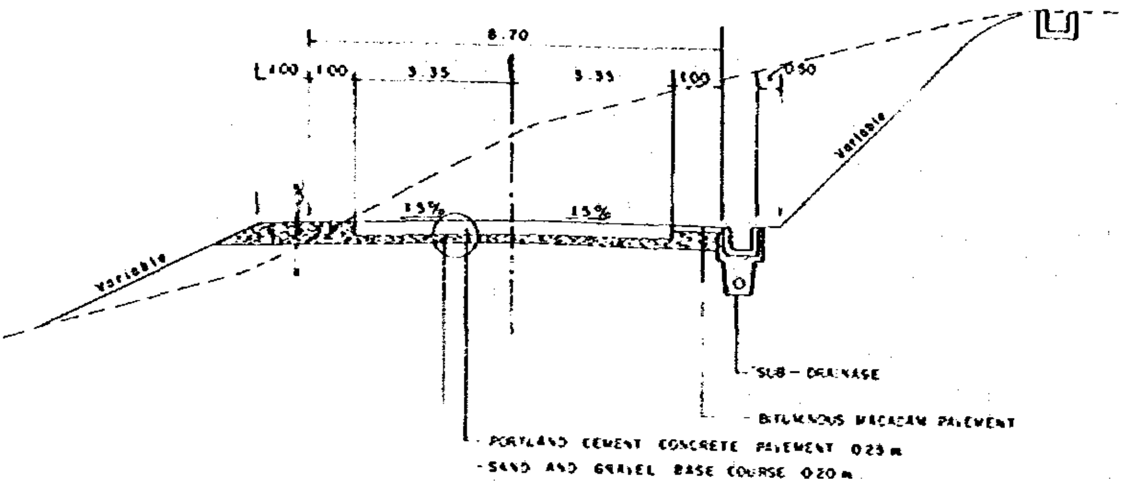
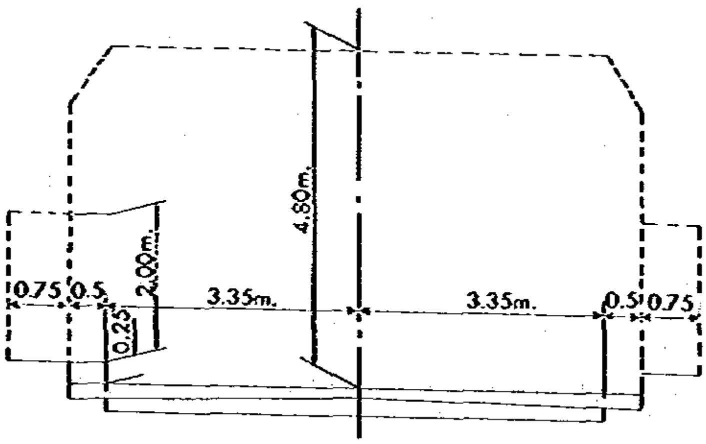


Fig.5-3.2 TYPICAL CROSS SECTION FOR MOUNTA I NOUS SECTION

The length of the proposed tunnel was estimatd to be more or less 2 km. Considering the estimated length of tunnel and also the future traffic volume and compositions, the side pass for surveillance of 0.75 m. wide on each side of the tunnel was provided according to the Japanese standards.

Taking the aforementioned factors into consideration, the clearance in tunnel section was determined as shown in the following:



Note: Vertical clearance is 4.5 m. by Japanese standards. However, it is determined as 4.8 m. according to Philippine standards.

Whether the sidewalk for pedestrians shall be provided or not was also studied. However, in this case the sidewalk was not considered because increased cross section of tunnel area increases the cost.

Typical Cross Section

For determing the cross section in the tunnel, aside form the above-mentioned clearance, the space or area which can be accommodated, the facilities for ventilation, lighting, safety, drainage and others were considered.

Based on the previous experiences in Japan and the Japanese Standard Specifications for tunnel, the lining thickness in the tunnel was determined as follows:

45 cm — for sections with good soil conditions and in rock classification A and B

60 cm — for sections located in weathered rock area and in rock classification C and which have unsuitable materials or unstable ground in rock classification D

When the classification is below D, the invert with thickness of 50 cm shall be additionally constructed.

The typical cross section of the tunnel adopted is shown in Fig. 5-3.3.

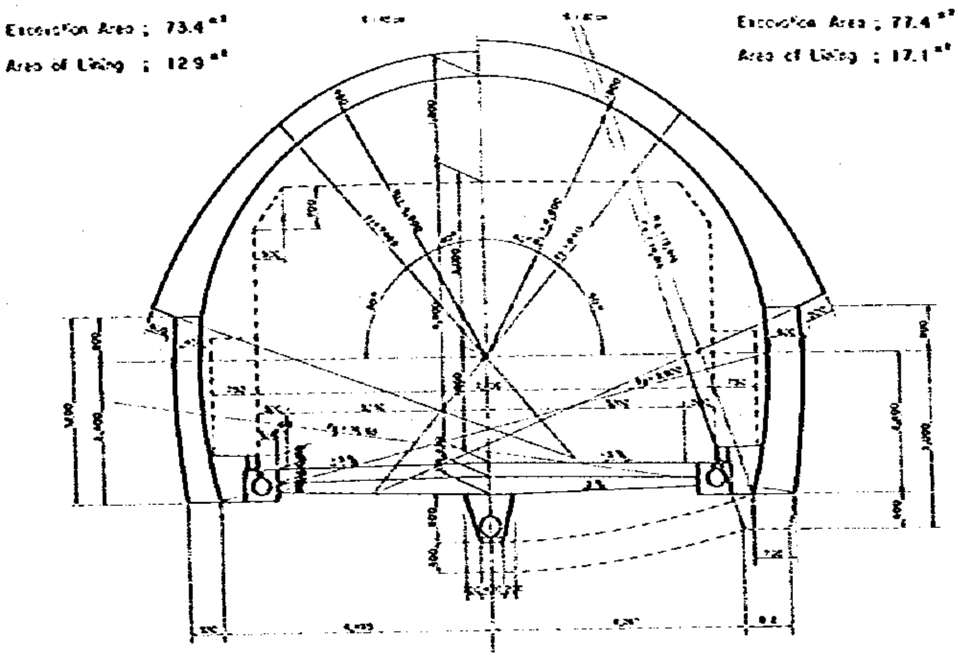


Fig. 5-3.3 TYPICAL CROSS SECTION OF TUNNEL 1/50

2) Typical Cross Section for Bridge

The typical cross section of bridges adopted is shown below in Fig. 5-3.4.

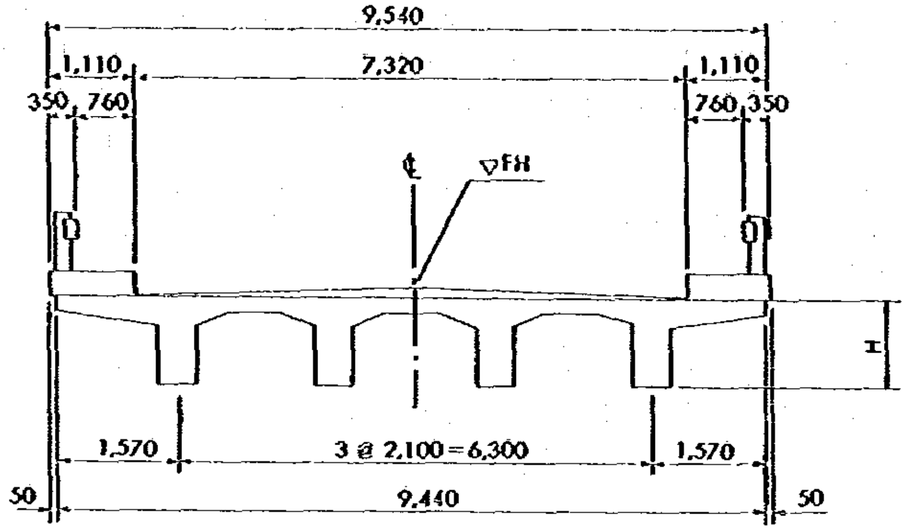


Fig. 5-3.4 TYPICAL CROSS-SECTION FOR BRIDGE

5-3-2 Description of Route Segmentation

The finalization of alignments, balanced combination of horizontal and vertical alignments, and also major structures such as bridges were considered and refined based on studies which were discussed in the preceding sections.

Considering presentation of construction quantities, the section under study was divided into four segments based on the balance of earth and also construction procedures. Estimate of construction costs to be discussed later is also based on the segmentation of the section.

Table 5-3.2 shows the length of each segment by construction items.

Segmentation is shown in Fig. 5-3.5.

TABLE 5-3.2 LENGTH OF EACH SEGMENT BY CONSTRUCTION ITEM

SEGMENT		1	2	3	4	
KM.		Km 202+000 Km 204+225	Km 204+225 Km 206+800	Km 206+800 Km 208+450	Km 208+450 Km 218+000	TOTAL
TOTAL LENGTH		2,225.0	2,575.0	1,650.0	3,110.0	9,560.0
NEW CONSTRUCTION SECT		1,725.0	2,575.0	1,650.0	1,715.0	7,665.0
EARTH WORK		1,325.0	1,490.0	705.0	1,367.0	4,887.0
CUT	Left side	660.0	1,412.5	250.0	425.0	2,747.5
	Right side	605.0	332.5	567.5	667.0	2,172.0
	Average	632.5	872.5	408.75	546.0	2,459.75
FILL	Left side	665.0	77.5	455.0	942.0	2,139.50
	Right side	720.0	1,157.5	137.5	700.0	2,715.0
	Average	692.5	617.5	296.25	821.0	2,247.25
STRUCTURES		400.0	1,055.0	945.0	378.0	2,808.0
TUNNEL		0	970.0	900.0	0	1,870.0
BRIDGE L ≤ 50		0	0	45.0 (1)	108.0 (3)	153.0
BRIDGE L > 50		400.0(3)	115.0(1)	0	270.0(3)	785.0
TO BE IMPROVED SECT.		500.0	0	0	1,395.0	1,895.0

INCLUDES SANTA FE BRIDGE (L=30 m.) in Segment 4

5-3-3 Horizontal Alignment

Since 90% of the total stretch along the selected route is in mountainous terrain (steep) the design speed adopted was 30 to 50 km/hr.

However, since the design is made on a plan in which scale is 1:5000, it was determined that the minimum radius of curve is 50 m. and that the maximum gradient is 7% respectively.

The transition of super-elevation is presumed to be made within a curve section since the detailed study on alignments is not appropriate on the plan with a scale of 1:5000. Thus, the transition curve through the clothoid curve which is applied more effectively on a larger scale (say 1;1000) was not considered.

1) Location of junction with the existing Route 5. (Particular attention was paid to Elementary School of Capintalan).

2) Location of crossing over Digdig River and protection of roadway structures from river flow. (Bridge No. 1 and protection by stone masonry)

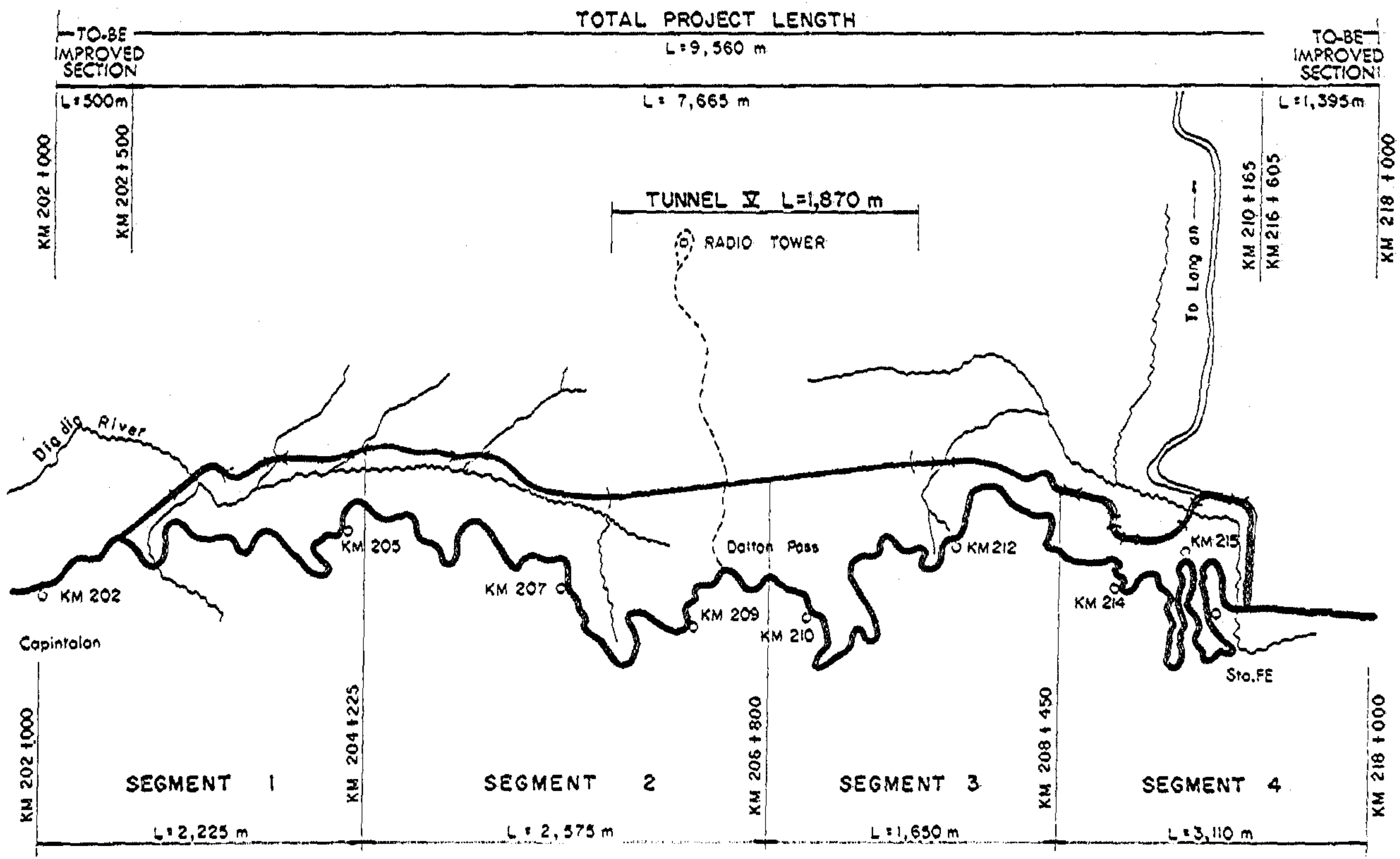


Fig. 5-3.5 SEGMENTATION

3) Consideration in crossing over branches of Digdig River (Alignment passing ridge and the location of abutment of bridge were determined for Bridge No.2 and No.3)

4) Section from 204 + 500 to Km. 205 + 000 passed through the landslide area, thus high cut slope was eliminated and embankment roadway was adopted. The route was planned to pass over deposit area by application of sabo dam.

5) South portal side (Capintalan side)

The location of the south portal of the tunnel was determind considering the area required for construction preparation and geological conditions. The proximity of Digdig River may however pose some constriction although measures are being proposed which will be discussed subsequently.

Although at the moment, construction of a sabo dam was considered, review of vertical slope of Digdig River will be required in the detailed engineering phase since the scale used for the study is only 1:5000.

6) North Portal Side (Sta. Fe side)

This item was discussed in detail in the preceding chapter.

7) Effect on Sta. Fe River

The channel work along Sta. Fe River affecting the northern alignment approach to the tunnel was considered.

5-3-4 Vertical Alignment

In order to shorten the tunnel length as much as possible, the steepest allowable grade was adopted at the approach of the tunnel. As a result, the length of sections with grade of more than 6% totals approximately 3,000 m. which is more than 30% of the total stretch.

Furthermore, climbing capacity for trucks (climbing lane) and necessity for securing the space for emergency and traffic safety were considered.

The control point for the south portal side is the grade of Digdig River bed and its branches. For the north portal side the high cut slope which is located parallel to the existing highway at Km. 208+700 and Km. 209, and the streams flowing into Sta. Fe River are considered. The following solutions were adopted:

1) Bridges over major steams (Bridge No. 5 to No. 10)
2) Protection of embankment slope by stone masonry on relevant sections of Digdig River
3) Application of slope protection (stone masonry and netting) along unstable high cuts.

5-3-5 Design of Climbing lanes

Assuming the design speed and allowable minimum operating speed as 50 km/hr., and 30 km/hr., respectively, the climbing capacity diagram was prepared as shown in Fig. 5-3.6.

The sections where the operating speed is presumed to be less than 30 km/hr, are as follows:

Section	Station	Length	Remarks
ⓐ	Km. 205+680 to 205+860	180 m	South side of tunnel
ⓑ	Km. 207+540 to 208+240	700 m	North side of tunnel
ⓒ	Km. 208+900 to 209+200	300 m	North side of tunnel

The design of the climbing lanes in the aforementioned three sections were based from the following considerations:

1) At the southern approach to the tunnel, section ⓐ, the length required is less than 200 m., the climbing lane is not necessary.

2) For the sections totalling 1000 m, study on adoption is necessary (Section ⓑ and Section ⓒ)

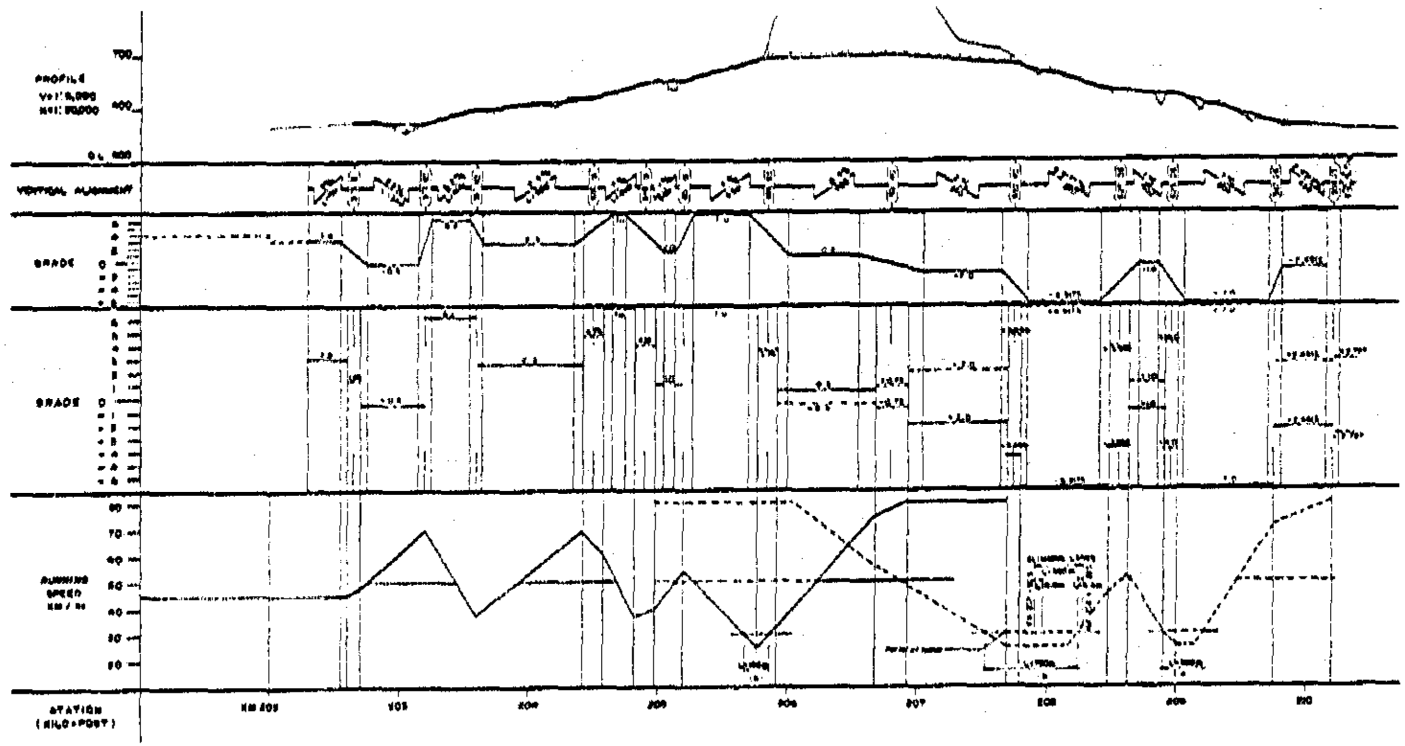
3) The climbing lane shall be extended inside the tunnel by 150 m. in section ⓑ in order for the operating speed to recover above the allowable minimum speed of 30 km/hr.

4) In Section ⓑ, a bridge with a length of 45 m. is included.

5) In Section c , two long-span bridges (L= 54 m and L= 60 m) are included.

6) In the sections of earthwork of section ⓑ, problems are foreseen for widening of the construction of climbing lane.

7) Since the steep slope is included in the sections of earthwork of section ⓒ, major effects are foreseen on construction volume and structure (Retaining wall, etc.)



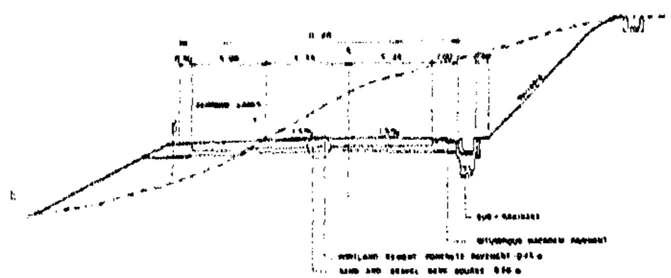


Fig. 5-3.6 CLIMBING CAPACITY DIAGRAM

The results discussed above are summarized as follows:

1) Section ⓑ : The climbing lane is required even up to the extent of inside the tunnel. However, it is possible to limit the lane before the bridge of north portal by constructing a passing bay.

2) Section ⓒ: Since the section where the lane to be located is too short (L = 300 m), with distance between two long-span bridges (L = 230 m), the climbing lane can be omitted by providing a passing bay in the cut section (Km 209+100).

The typical cross section with a climbing lane of which the width is 3 m. is shown in Fig. 5-3.6.

Considering the taper lane with 45 m. at the start and with 60 m. at the end, with adjacent bridges and tunnel portal, the construction of climbing lane was planned at the section from Km. 207 + 925 to Km. 208 + 285, of which total length is 360 m.

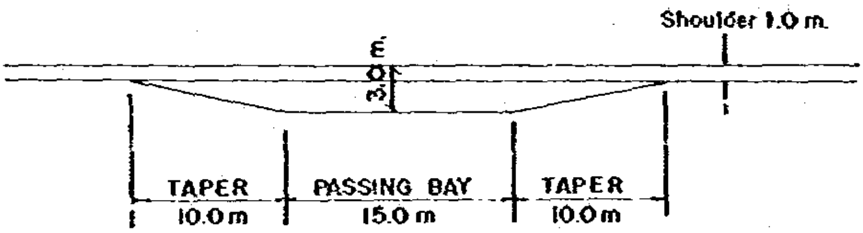
Even though the length is not sufficient, the deficiency shall be fulfilled by providing a parking space.

5-3-6 Passing Bay and Clearance for Safety

1) Passing Bay

For safety measures it is considered necessary to provide a passing bay at the section where the long steep gradient 7% is planned.

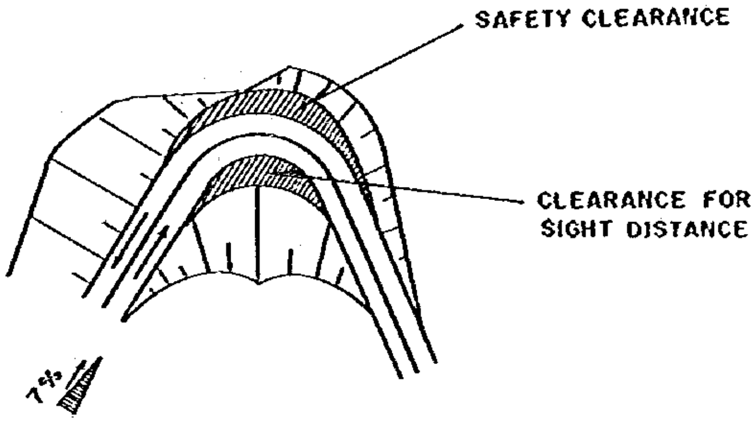
The following passing bays were planned at the climbing side and low-cut sections as follows:



i) Km. 204 + 750 - In the middle of 7% grade, valley side on embankment section
ii) Km. 205 + 600 - Before tunnel portal, half-way of steep grade valley side on low embankment section
iii) Km. 207 + 500 - Before tunnel portal, or planning of parking area
iv) Km. 208 + 100 - 7% upgrade, half-way, valley side of low-cut section
v) Km. 209 + 100 - 7% upgrade, half-way, cut section

2) Clearance for Safety

The clearance for safety was planned, for the section where it has steep down-grade with a small radius curve as shown below.

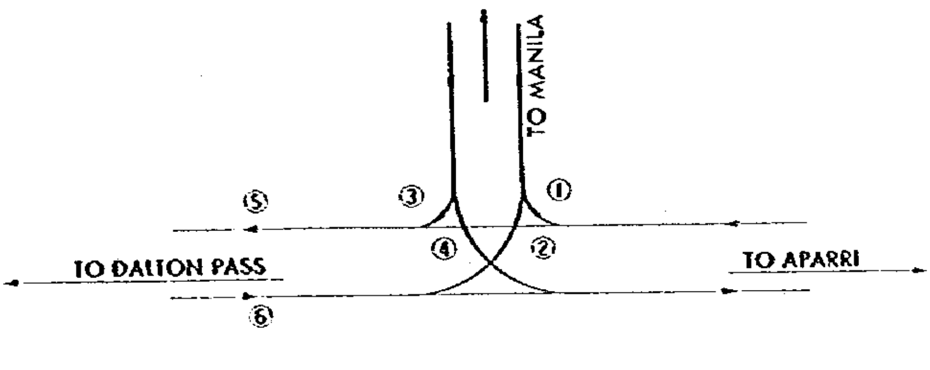


Specifically, this was considered at the following locations:

Location : Km. 208 + 350
Gradient : 6.9%
Horizontal Alignment : R = 50 m.

5-3-7 Sta. Fe Junction

The preliminary study on junction connecting the existing Route 5 at Sta. Fe was undertaken.



The type of junction with the existing highway is "T" type. Since the flow of ① and ②, connecting Manila and Aparri is expected to be a major flow, this was presumed to be a through-artery-line.

Considering the convenience and access to inhabitants living along the existing highway at Dalton Pass Area, at-grade junction with ramps for directions of ③ and ④ was planned along with the improvement of Sta. Fe Bridge. See Drawing Fs-12.

5-3-8 Bridges

Based on the design standards adopted for this study and data, especially on hydrology, the preliminary designs of the following bridges were made. (See Table 5-3.3 and Fig. 5-3.7.

The plans and construction quantities of bridges were prepared and are shown in Drawings FS-13 to FS-19 and Appendix F.

TABLE 5-3.3 LIST OF BRIDGES

STA. (Km)	BRIDGE NAME	TYPE	LENGTH (m)	REMARKS
202+560.0	S.D.P No. 1	PCG	220	
203+702.5	S.D.P No. 2	PCG	90	
204+180.0	S.D.P NO. 3	PCG	90	
205+095.0	S.D.P NO. 4	RCDG PCG	115	
		SUB-TOTAL	515	
207+900.0	N.D.P NO. 1	RCDG	45	
208+474.0	N.D.P NO. 2	PCG	48	
208+872.0	N.D.P NO. 3	RCDG PCG	55	SHINSO PILE
209+160.0	N.D.P NO. 4	PCG	60	SHINSO PILE
209+558.5	N.D.P NO. 5	PCG	155	
209+830.0	N.D.P NO. 6	RCDG	30	
216+400.0	STA. FE	PCG	30	
		SUB-TOTAL	423	
		GRAND TOTAL	938	

5-3-9 Sabo (Prevention Against Debris and Sediment Flow)

(1) Hydrological Study

Digdig River originates at the southern part of Dalton Pass, while Sta. Fe River starts at the northern part. The former flows toward south and the latter towards north. In the objective area both streams have a narrow width of watershed. The average widths are confined to only 6 to 8 km. This means that most of the tributaries across the existing road or planned road are to be torrential small streams with little catchment areas.

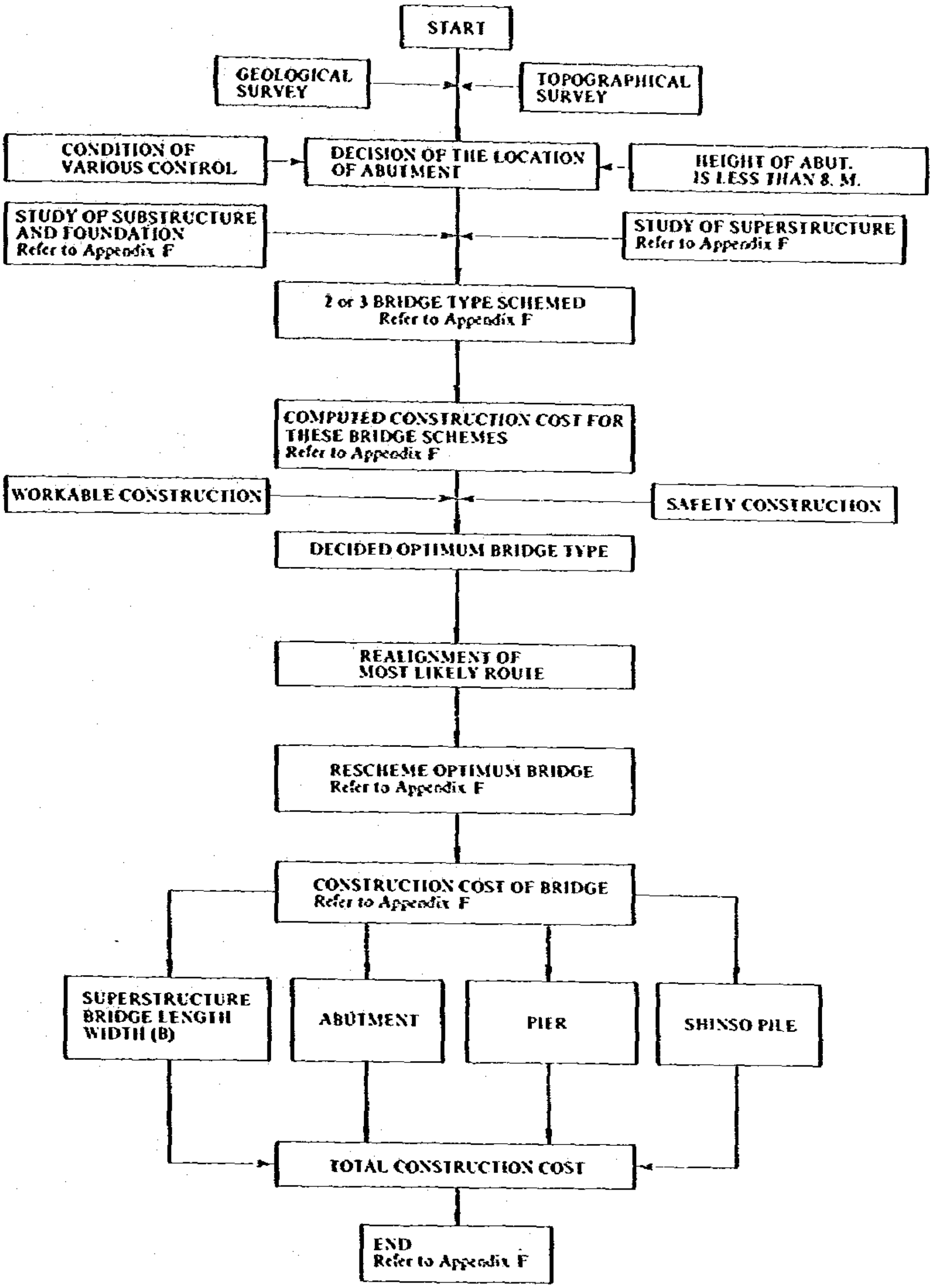
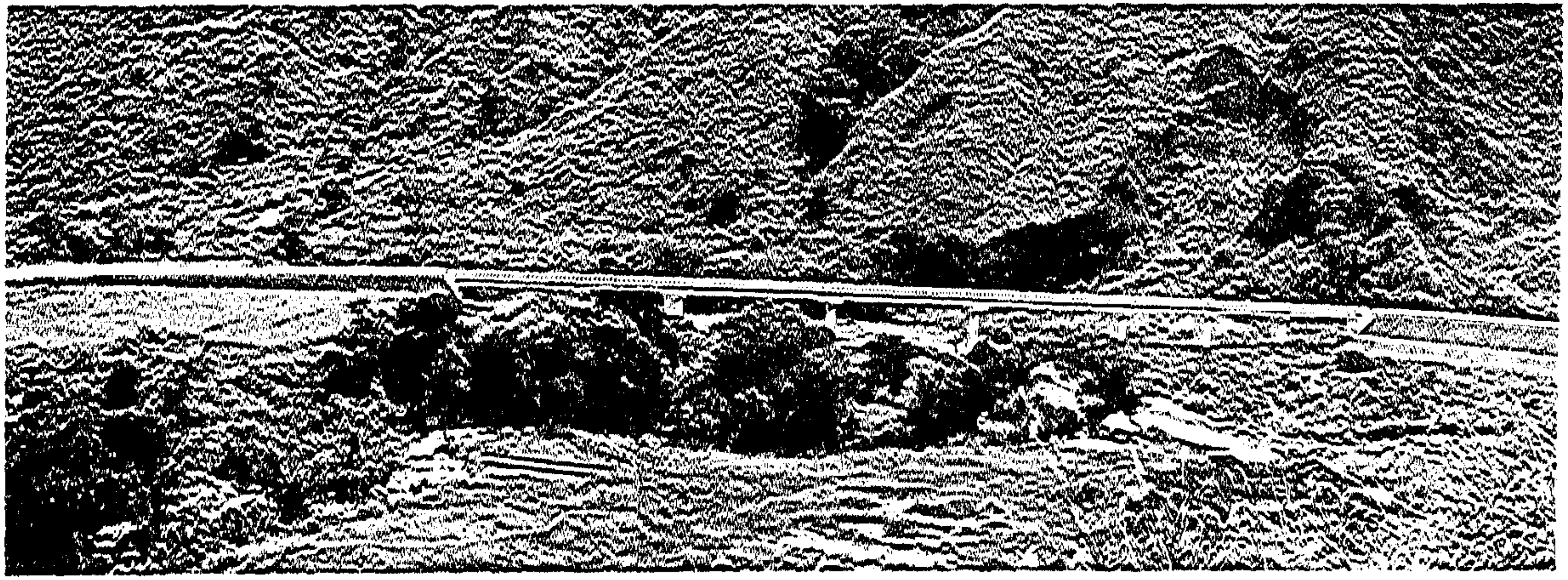


Fig. 5-3.7 FLOW CHART FOR DETERMINATION OF BRIDGE TYPE



PCG BRIDGE. A proposed PCG bridge in Capintalan crossing Digdig River is envisioned as shown above. Bridges were designed considering the economic aspects and conforming to topographical and geological conditions.

It is essential for planning of the road to estimate the flood discharge of these small tributaries. The approximate estimation of flood at the outfall of each tributary will give support to the design capacity of road structures such as bridge, box culvert, drainage pipe and sabo works.

Likewise, the study will also include the two main streams Digdig and Sta. Fe rivers, which have a rather big catchment area of 100 to 300 sq. km. along the influence area. This will be very important in protecting low road sections from destructive floods.

1) Rainfall Intensity Within Two Hours

As far as the small tributaries with less than 30 sq. km. catchment area are concerned, concentration time of flood will be less than one and a half hours. However, there is no record of short time rainfall data available within the project area and the neighboring district. Hence, the data from Rainfall Intensity-Duration-Frequency Data of the Philippines, Vol. I, First Edition, published by the Hydrology and Flood Forecast Center last January 1981 will be used as reference in this study.

From the above report, data at Cabanatuan City and Tuguegarao will be used as representative data on the southern and northern part of Dalton Pass, respectively. However, because of the distance of these two places and their difference in elevation with that of the project area, the data at Baguio City will also be considered simultaneously as a representative data in this project.

The values at different time intervals of ten, thirty, sixty and one hundred twenty minutes at the three places are maximum values computed by Gumbel's method based on more than 20 years records. This was plotted on the same graph triplicatedly as shown in Figs. 5-3.8 and 5-3.9. The former shows the result at a return period of ten years and the latter at twenty-five years.

As shown in the above-mentioned figures, the rainfall intensity curve within one hour closely resembles and shows the same tendency among the three curves. However, in the region over one hour the intensity at Baguio City is conspicously bigger than the other two places. This fact is to be naturally expected because Baguio City is known for its abundant rainfall all year round.

The same figures also show the planned rainfall intensity curve. This could be suitable chiefly in the region of thirty to ninety minutes duration time.

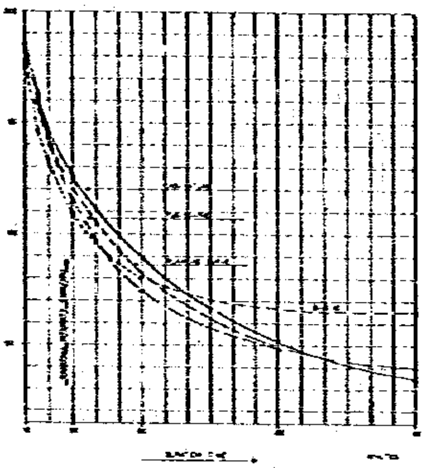


Fig. 5-3.8
RAINFALL INTENSITY DURATION CURVE
COMPUTED AT RETURN PERIOD, 10 YEARS

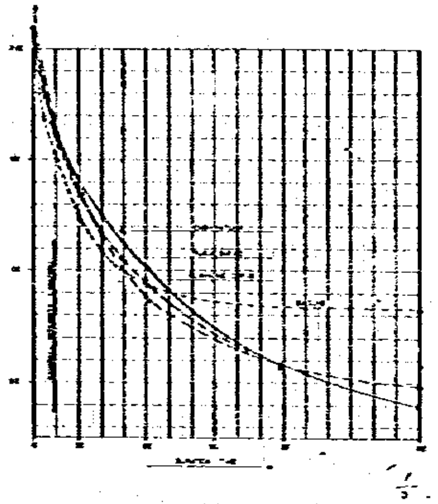
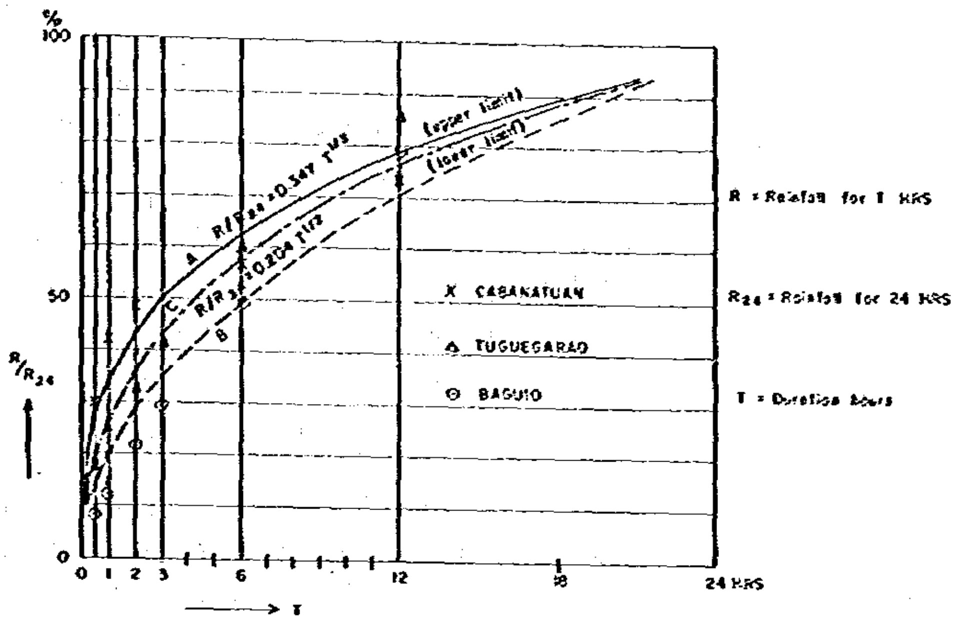


Fig. 5-3.9
RAINFALL INTENSITY DURATION CURVE
COMPUTED AT RETURN PERIOD, 25 YEARS

2) Relation Between Daily Rainfall and Its Several Hours Rainfall

Generally, it is hardly possible to get available record of daily rainfall for the past twenty years; hence, only records covering a ten-year period was used in the study of the relationship between daily rainfall and its several hours rainfall. The purpose of this study is to find out the average rainfall intensity within a certain concentration time of flood peak discharge.

Based on the aforementioned PAGASA's data, the values were plotted on a semi-logarithmic and logarithmic plotting paper simultaneously as shown in Figs. 5-3.10. and 5-3.11. A-curve and B-curve gives the upper and lower limits of R/R_{24}, respectively. Thus, the relationship can now be represented by the C-curve. The figures also show the exponential relations of Curves A, B and C. The C-curve can be adopted as the planned curve.

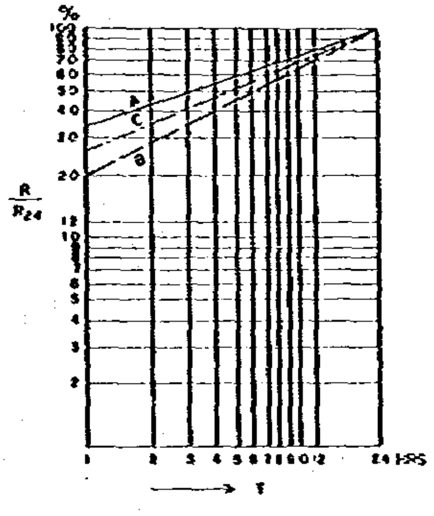


NOTE :

A — curve shows upper limit ,
B — curve shows lower limit of the relation , and intermediate curve
C — shows the following relation to be considered proper for the objective area

$$R/R_{24} = 0.276\ T^{0.42}$$

Fig. 5-3.10 RELATION BETWEEN R/R_{24} AND T



NOTE :

R = Amount of rainfall in the time T hour
R_{24} = Amount of daily rainfall
T = Time of the duration
A — Line shows $R/R_{24} = 0.347\ T^{1/3}$ (upper limit)
B — Line shows $R/R_{24} = 0.204\ T^{1/2}$ (lower limit)
C — Line shows $R/R_{24} = 0.276\ T^{0.42}$ (mean)

Fig. 5-3.11 RELATION BETWEEN R/R_{24} AND T

3) Computation of Probable Daily Rainfall

The aforementioned PAGASA data also shows the computed probable amount of daily rainfall as for Cabanatuan City, Tuguegarao and Baguio City, that is:

PROBABILITY / GAUGING STATION	AT RETURN PERIOD of 10 yrs.	AT RETURN PERIOD of 25 yrs.
Cabanatuan City	209.0 mm	249.6 mm
Tuguegarao	295.1 mm	371.6 mm
Baguio City	673.7 mm	852.5 mm

As seen in the above table, it is particularly distinctive that the daily rainfall at Baguio shows the conspicuous big value. However, it is expected that the pattern of daily rainfall will be different from the project area to a large extent.

Meanwhile, some daily rainfall records were found in Tayabo, San Jose, Nueva Ecija and Consuelo, Sta. Fe, Nueva Vizcaya. The number of samples is not sufficient to analyze statistically, although both stations are very close to the objective area. The probable daily rainfall shall be computed hereunder.

The process of computation and the result obtained by Gumbel's method are shown in Fig. 5-3.12 and 5-3.13. and Table 5-3.4 and 5-3.5 That is:

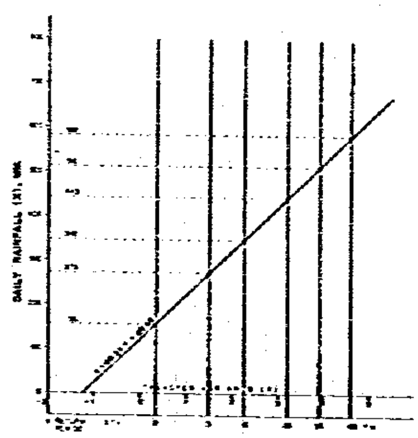


Fig. 5-3.12
PROBABLE DAILY RAINFALL
AT TAYABO, SAN JOSE CITY, NUEVA ECIJA
Computed by Gumbel's Method

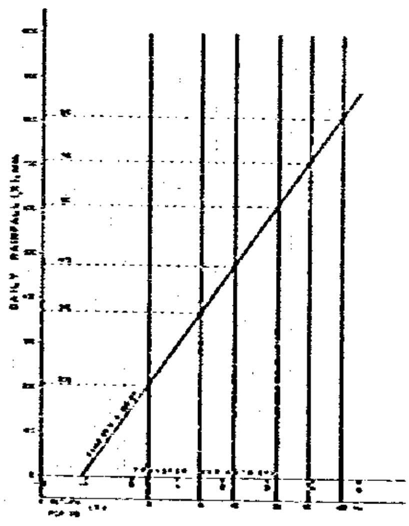


Fig. 5-3.13
PROBABLE DAILY RAINFALL
AT CONSUELO, STA FE, NUEVA VIZCAYA
Computed by Gumbel's Method

TABLE 5-3.4 COMPUTATION OF PROBABLE RAINFALL AT TAYABO, SAN JOSE CITY BASED ON PAG-ASA DATA (GUMBEL'S METHOD)

ORDER NO.	X_i	DATE OF OCCURRENCE	X_i^2
1	424.9	MAY 22, 1976	180,540
2	280.2	NOV. 5, 1980	78,512
3	156.7	JUNE 10, 1974	24,555
4	125.0	AUG. 24, 1978	15,625
5	117.4	JUNE 7, 1979	13,783
6	100.1	AUG. 2, 1977	10,020
7	55.6	AUG. 24, 1975	3,091

$\Sigma Xi = 1259.9 \qquad \Sigma Xi^2 = 326,126$

$X_a = \frac{Xi}{N} \qquad \frac{\Sigma Xi^2}{N} = 46,589.43$

$= 179.99$

$X_a^2 = 32,394.86$

COMPUTATION REMARKS

$s = \sqrt{\left(\frac{N}{N-1}\right)\left[\left(\frac{\Sigma Xi^2}{N}\right) - \left(\frac{\Sigma Xi}{N}\right)^2\right]}$

$= \sqrt{\frac{7}{7-1}\,(46,589.43 - (179.99)^2)}$

$= 128.68$

$K = 0.7797 \times 128.68 = 100.33$

$U = 179.99 - 0.5772 \times 100.33$

$= 122.08$

$X = Ky + U$

$= 100.33Y + 122.08$

Where: Y=−2	X=−78.58
−1	21.75
0	122.08
1	222.41
2	322.74
3	423.07
4	523.40
5	623.73

TABLE 5-3.5 COMPUTATION OF PROBABLE RAINFALL AT CONSUELO, STA. FE BASED ON PAG-ASA DATA (GUMBEL'S METHOD)

ORDER NO.	X_i	DATE OF OCCURRENCE	X_i^2
1	732.0	NOV. 5, 1980	535,824
2	401.8	MAY 24, 1976	161,443
3	228.6	SEPT. 11, 1970	52,258
4	209.6	OCT 16, 1974	43,932
5	193.0	JULY 28, 1972	37,249
6	190.3	AUG. 24, 1978	36,214
7	168.9	OCT. 15, 1973	28,527
8	165.4	OCT. 2, 1979	27,357
9	127.0	OCT. 4, 1971	16,129
10	119.3	NOV. 14, 1977	14,232
11	88.9	AUG. 10, 1975	7,903

$\Sigma Xi = 2624.8 \qquad \Sigma Xi^2 = 961,068$

$X_a = \frac{Xi}{N} \qquad \frac{\Sigma Xi^2}{N} = 87,369.82$

$= 238.62$

$X_a^2 = 56,938.64$

COMPUTATION REMARKS

$s = \sqrt{\left(\frac{N}{N-1}\right)\left[\left(\frac{\Sigma Xi^2}{N}\right) - \left(\frac{\Sigma Xi}{N}\right)^2\right]}$

$= \sqrt{\frac{11}{11-1}\,(87,369.82 - (238.62)^2)}$

$= 182.99$

$K = 0.7797 \times 182.99 = 142.67$

$U = 238.62 - 0.5772 \times 142.67$

$= 156.27$

$X = Ky + U$

$= 142.67Y + 156.27$

Where: Y=−2	X=−129.07
−1	13.60
0	156.27
1	298.94
2	441.61
3	584.28
4	726.95
5	869.62

PROBABILITY / GAUGING STATION	AT RETURN PERIOD of 10 yrs.	AT RETURN PERIOD of 25 yrs.
Tayabo (San Jose)	348 mm	443 mm
Consuelo Sta. Fe	475 mm	610 mm

The values at Consuelo, 475 mm and 610 mm are largely affected by the record of 732 mm daily rainfall at the time of Typhoon Aring, last November 5, 1980; on the other hand, the amount of daily rainfall at Tayabo on the same day was limited only to 280 mm as seen in Table 5-3.4. Inspite of those facts, the conditions of disaster by Typhoon Aring are almost the same on both sides of Dalton Pass.

Therefore, for practical purposes, applying the same value (440 mm to 475 mm) to both sides of Dalton Pass is deemed proper. This means that the record of 732 mm daily rainfall at Consuelo at the time of Tyhoon Aring happened to be an anomalous localized downpour. Hence, it is not considered as representative amount of rainfall for the whole area concerned:

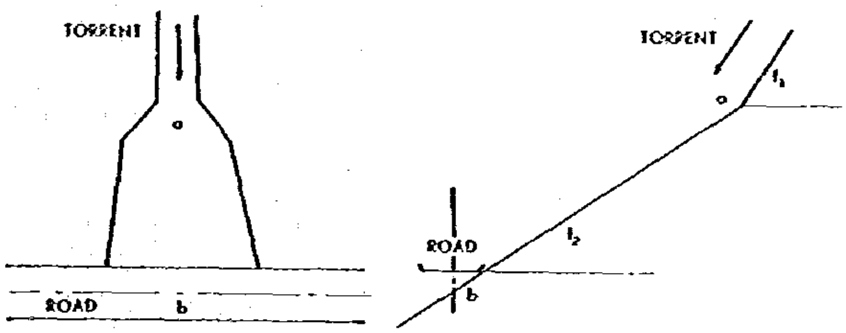
(2) Prevention Against Debris/Sediment Flow

The investigation of the torrents crossed by envisaged Route II was carried out from the viewpoint of engineering judgement which aims to estimate the degree of danger due to debris flow. The result will be able to give an important information to the designer of road structure. The countermeasure against debris flow shall not be treated as a problem of sabo engineering but as a problem of road structure itself, because it is scarcely possible to control debris flow on the roadside only. The countermeasures are presumed to be in such a negative way as to facilitate the passage of debris flow at the intersection point of the road.

Meanwhile, Digdig River is extremely devastated along the main course of its upper reaches, especially on the parcel of and adjacent to tunnel entrance. Accelerated erosion may lead to the outbreak of large-scale landslide and blockade of rivercourse. Sta. Fe River on the other hand always suffers from heavily-loaded sediment flow. The planned route passes by or across this devastated river-course. Therefore, it seems absolutely necessary to stabilize these devastated torrential streams by applying sabo engineering, apart from the problem of road structure itself.

1) Investigation of Debris/Sediment Flow

Along the planned route there are many streams to be crossed by the road. From the viewpoint of disaster prevention, it is important to investigate the conditions of these streams especially for debris flow. Such factors that are considered to be related to the safety of the road are illustrated as shown in the figure below.



L : Distance between "a" point where the torrent extends its width downwards and "b" point where the torrent is crossed by the road.

I_1 : Gradient of torrent at "a" point

I_2 : Gradient of torrent at "b" point

W_1 : Width of torrent at "a" point

W_2 : Width of torrent at "b" point

According to the result of the research conducted by the Public Works Research Institute (Ministry of Construction, Japanese Government), the debris flow can be virtually checked when the stream bed slope is less than 10 and one-half of the former bed slope or when the width of bed extends two or three times the former width. The factors mentioned above are to be surveyed. The results are shown in Table 5-3.6.

In the same table, the problem maximum flood discharge is shown respectively. Each value of maximum flood discharge, should be referred to in order to secure necessary cross sectional area of the road structure at each site. The amount of flood discharge could also suggest the scale of sediment flow and it is usually computed in a varied mode of flood flow.

The characteristics of each torrent are originally derived from the geological and geomorphological conditions of each watershed as well as the above-mentioned other factors totally classified conditions that are shown in the same table. Also the vegetation status indicates the integrated features of the same table.

TABLE 5-3.6 DEBRIS AND SEDIMENT FLOW

	CATCH-MENT (Km.²)	DEBRIS FLOW COEFFICIENT			FEATURES OF WATERSHED		HYDRAULIC CONDITIONS		REMARKS
		L (m)	$\frac{I_2}{I_1}$	$\frac{W_2}{W_1}$	GEO-TOPO STATUS	VEGE-TATION	Q (m^3/sec)	D (m^3)	
Km. 202+950 Km. 203+170	21.27		1/40	35	F	S	596	850,000	Big failure in right side of river BR. II-1
Km. 203+647.5 Km. 203+747.5	6.34	250	$\frac{1/15}{1/8}$	25/15	B	T	178	544,130	Terrace deposit in left side of river BR. II-2
Km. 204+135 Km. 240+225	2.99	300	$\frac{1/20}{1/15}$	25/15	B	T	84	121,160	Many failures in upper stream Br. II-3
Km. 205+326 Km. 205+379	2.24	100	$\frac{1/12}{1/8}$	25/15	B	T	62	24,930	Fault valley, big failure in upper stream BR. II-4
Km. 207+572.5 Km. 207+932.5	0.70	100	$\frac{1/10}{1/6}$	15/10	B	P	19.6	17,000	Fault valley, Fan deposit BR. II-5
Km. 208+450 Km. 208+500	0.07 0.07	—	$\frac{1/12}{1/15}$	—	B	P	2	1,060	Old fan deposit, Boulder on stream BR. II-6
Km. 209+532 Km. 209+587	0.07	—	$\frac{1/18}{1/12}$	—	B	P	2	1,060	Boulder of Limestone, granitic rocks (masa) failure in upper stream BR. II-7
Km. 209 Km. 209	0.30	—	$\frac{1/8}{1/2}$	—	B	P	8.4	9,430	Failure along the stream Granitic rock (masa) BR. II-8
Km. 209 Km. 209	0.02	50	$\frac{1/4}{1/2}$	10/5	B	P	0.6	600	Small failure in upper stream Granitic rock (masa) PIPE 1200 ϕ
Km. 209 Km. 210	0.23	100	$\frac{1/6}{1/4}$	15/8	B	P	7.8	7,530	PIPE 1200 ϕ BOX CULVERT 2×2
Km. 210+090 Km. 210+150	24.60	800	$\frac{1/40}{1/20}$	30/20	F	T	595	922,090	BR. II-9 Outcrop in river bed Granitic rock (masa)
Km. 210+400 Km. 210+410	0.16	20	$\frac{1/18}{1/6}$	20/8	B	S	85	760	Granitic rock (masa) BR. II-10

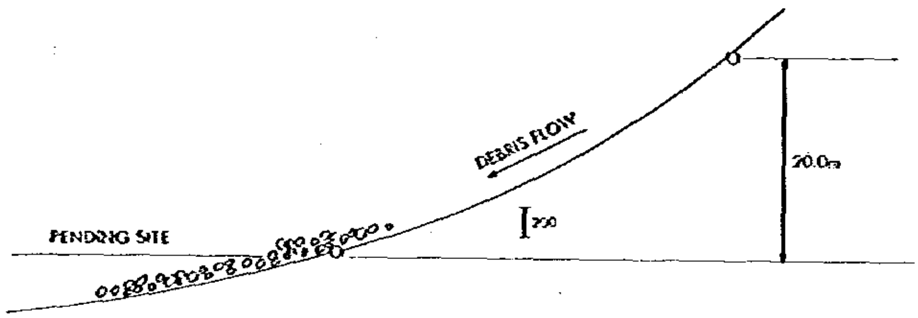
GEOLOGICALLY & GEOMORPHOLOGICALLY
G – good
F – fair
B – bad

VEGETATION CONDITION
R – rich
S – scanty
P – poor

In Table 5-3.6 values of D can be seen. Value D means the maximum amount of debris flow per one flood time. This assumption is given by the Disaster Prevention Institute, Kyoto University, Japan as follows:

$$D = \alpha (A \cdot R_{24} - I_{200})^2$$

D : Amount of debris flow as for one flood time (m^3)
R_{24} : Maximum Daily Rainfall
I_{200} : Gradient along the river course between the pending site and such a site as high as 200 m than the pending site. (See below)
α : Coefficient (α = 7~10)



The result obtained gives an important index to reorganize the maximum scale of debris-flow. The results obtained through the above-mentioned procedure and analysis are reflected into the preliminary design of highways, drainage and others.

2) *Countermeasure on the southern side of Dalton Pass*
The "Most Likely Route" branches off at Km. 202 and crosses Digdig River then proceeds 2,600 m. up to the entrance of the tunnel along the right bank side of main course of the river. The route is crossed by three tributaries of Digdig River, with two to six sq. kms. of watershed. At the interesection point of the planned road debris/sediment flow was studied. Such structures as bridge, box culvert and drainage pipes were properly designed based on the aforementioned result of investigations. Sabo dam or channel work was schemed chiefly due to technical economy.

Meanwhile, five sabo dams were planned in a series along the main course of Digdig River for the following purposes:

a) To support and stabilize the foundation of the newly delineated road.
b) To prevent accelerated erosions on the riparian side of the slope
c) To dispose of tunnel muck or residual earth, and
d) To maintain the natural environmental condition

The preliminary designs and drawings involved are shown in Drawing FS-10.

3) Countermeasure on the northern side of Dalton Pass

The Route crosses eight small tributaries of Sta Fe River. Seven streams with less than 0.3 sq.km., considered more as gullies or rills develop on the northern side of Dalton Pass. However, these extremely small streams, triggered by the exploitations of the Route may lead to the outbreak of debris flow. The structure of the road across these streams were carefully designed based on the results of investigation, as shown in Table 5-3.6. Meanwhile, in the main course of Sta. Fe River, one sabo consolidation dam, three consolidation works and ground sills were planned as well as revetment works along the course of the river. These construction works generally are called "channel work". In this case, the purpose of the channel work is as follows:

a) To support and stabilize the parcel of land along the planned route
b) To protect the planned road from sediment flow disaster of Sta. Fe River
c) To dispose tunnel muck or residual earth
d) To maintain the natural environmental conditions

4) Method of Computation on Maximum Flood Discharge

In the objective areas the gradient along the small streams is confined to more than 1/100. The propagation velocity (w) can be assumed as w + 3.5 m/sec based on Kraven's formula. The concentration time of flood, therefore, is easily determined with such assumption that ground surface water will take around 20 minutes until it enters the stream course. That is:

$$T = L/60w + 20 = L/120 + 20 \quad \ldots\ldots (1)$$

where:

T = Concentration time of flood (minute)
L = Length of the stream (m)
w = Propagation velocity of flood (m/sec)

Corresponding to the value T thus obtained, the average intensity of rainfall within the concentration time (r) can be read on the C-curve in Fig. 5-3.8 or Fig. 5-3.9.

Then, the maximum flood discharge is to be computed by the following equation:

$$Q = \frac{1}{3.6} f \cdot r \cdot A = q \cdot A \quad \ldots\ldots (2)$$

where:

Q = Maximum discharge of flood (m^3/sec)
q = Specific flood discharge (m^3/sec/Km^2)
A = Catchment area (Km^2)
r = Average rainfall intensity (mm/hr)
f = Run-off coefficient (0.65 – 0.75)

Upon applying the above equation (1) the concentration time (T) will be 30 to 40 minutes at most, as the length of the stream (L) is limited mostly within 2 km. as far as small streams are concerned. Even in the case of rather big streams with a stream length of around 10 km., likewise, the concentration time will be confined from 50 to 80 minutes at the most. Hence, noting the length of the stream, the concentration intensity of rainfall practically can be assumed in accordance with the stream length as follows:

In the case of stream length $L \leqslant 9000$ m ..... $r = 135$ mm

In the case of stream length $9000 < L \leqslant 15{,}000$ m $r = 95$ mm

The above-mentioned value $r = 135$ mm corresponds to the intensity of 30 minutes at the return period of 25 years, while $r = 95$ mm to that of 60 minutes at the same return period.

Thus, specific flood discharge (q) in the equation (2) can be given, assuming run-off coefficient $f = 0.75$ as follows:

In the case, $L \leqslant 9000$ m, $q = 1/3.6 \times 95 \times 0.75 = 28$ $m^3/sec/km$

In the case, 9000 m $< L \leqslant 15{,}000$ m, $q = 1/3.6 \times 95 \times 0.75 = 20$ $m^3/sec/km^2$

The obtained result suggests that the following rough estimation of flood discharge could be applied, that is:

$$Q = (20 \sim 28) \cdot A \quad (m^3/sec)$$

Nevertheless, in such case that the concentration time is considered to be more than 90 minutes, the above simple method is not appropriate to apply to. In computing the maximum flood discharge along the main course of Digdig River or Sta. Fe River the average intensity of rainfall within the concentration time could be computed by using the curve C as shown in Fig. 5-3.8 or Fig. 5-3.9.
Assuming such proper daily rainfall (R_{24}) the amount of rainfall in a certain hour (R) can be obtained easily from the ratio(R/R_{24}). Then, the average rainfall intensity (r) is:

$$r = R/T \quad (mm/hr)$$

Accordingly, the maximum discharge of flood (Q) can be computed by the application of equation (2).