tion of a new bridge. From this cost comparison, it was decided that the above three bridges should be totally reconstructed. The other ten bridges listed in Table 4.4-1 should also be reconstructed, because of their unfavorable horizontal alignment and effective width.

Table 4.4-1 Evaluation of Existing bridge characteristics

Bridge	Horizontal Alignment	Effective Width	Structure
Patuni	N	N	N
Challa	N	N	N
Cascada	P	N	N
Alto Choro-2	N	N	N
Pto Leon	N	N	N
Cajones	N	N	N
Chojña	P	N	N
San Silverio	N	N	N
Yara (*)	P	P	P
San Lorenzo	N	N	P
Espiritu	N	N	P
Carrasco	N	N	P
Avaroa	N	N	N

* Bridge Length = 180.75 m
P Complying with the criteria
N Not complying with the criteria

It was concluded that thirteen (13) bridges , at all locations except Yara listed in the previous section , are to be newly constructed including a new bridge at Point A.

4.4.2 Exact Location and Scale of New Bridges

The optimum location of the above thirteen (13) bridges was examined in two groups, categorized depending upon the situation of the horizontal alignment of the road before and after each bridge:

1) Group I - The approach road is almost perfectly aligned.
   (The Cascada, Cajones, and Chojna bridge)

The location of each bridge is almost automatically determined by the horizontal alignment of the road. As a result, each new bridge is constructed at almost the same location of the existing bridge. The length of each bridge is given as follows:

Cascada bridge	18.5m	(see Fig. 4.4-2)
Cajones bridge	25.0m	(see Fig. 4.3-1 and Fig. 4.3-2)
Chojna bridge	22.0m	(see Fig. 4.4-3)

2) Group II - The approach road is not aligned with the bridge axis.
   (Point A, Patuni, Challa, Alto Choro-2, San Silverio, San Lorenzo, Espiritu, Carrasco, and Avaroa bridge)

All of the bridges in this group are located in the valley. Constructing a bridge in the deep section of the valley makes the bridge length short, but the volume of earthworks required for the approach road becomes large. Therefore, to find the optimum, or the most economical, construction site for such a bridge, <u>a sum of the costs for the bridge and for the approach road at each alternative location must be estimated and evaluated.</u>

Furthermore, these bridges are likely to be curved bridges, and in that case the radius of horizontal curvature (R) should be as follows:

a) Bridges constructed by the scaffolding method
   : <u>R is 50m or greater.</u>

b) Bridges constructed by other methods
   (cantilever method, extraction and sliding method, etc.)
   : <u>R is 100m or greater.</u>

In addition, the structure of a curved bridge should be <u>slab type</u> with a high torsional rigidity, <u>or a box girder type</u> from the viewpoint of structural characteristics. Furthermore, <u>a continuous structure</u> is desirable for the following reasons:

a) To maintain torsional rigidity
b) To reduce twisting moments
c) To prevent negative reactions

Keeping the above conditions in mind, the alternative bridge locations were compared and examined with the purpose of determining the most economical location. The results of the examination for each bridge are presented in Table 4.4-2 (1) through 4.4-2 (9) and Table 4.4-3.

However, the results of examination for the Avaroa bridge were not attached for the following reasons:

Avaroa bridge is across the Mula Jihuata river, which descends sharply cascading like a waterfall. In addition, the embankment slope is very steep (around 50 degrees) and therefore, it is significantly difficult to construct piers. Consequently, it is obviously advantageous to span the river with a simple girder bridge around the location of the existing bridge. (See, Fig. 4.4-4)

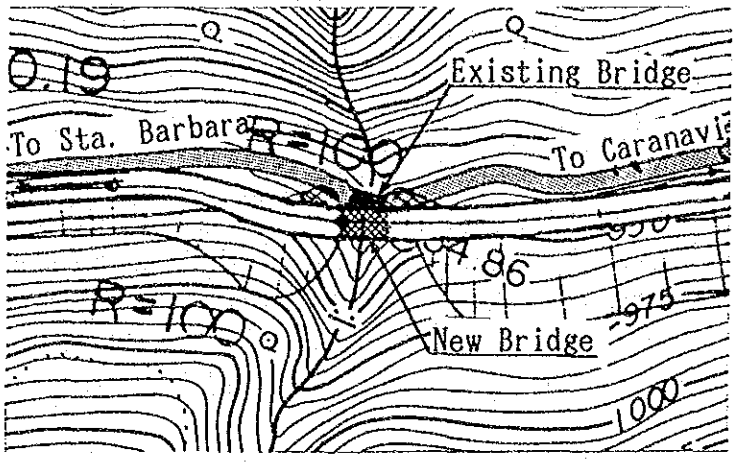


Fig 4.4-2 Location of Cascada Bridge

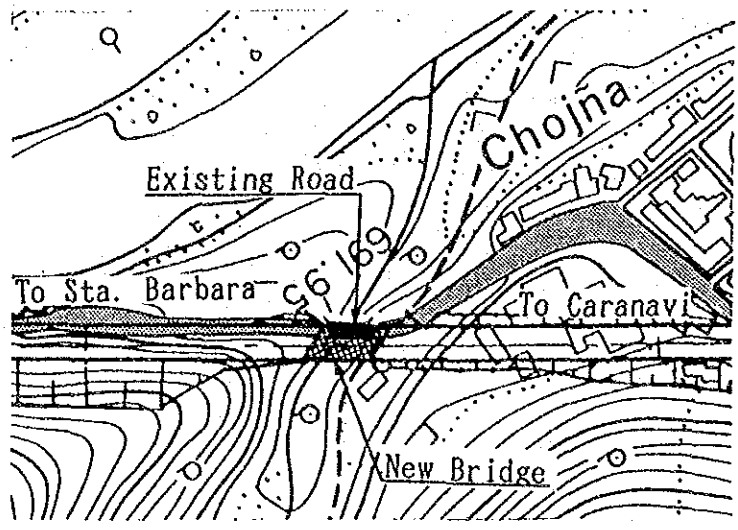


Fig 4.4-3 Location of Chojña Bridge

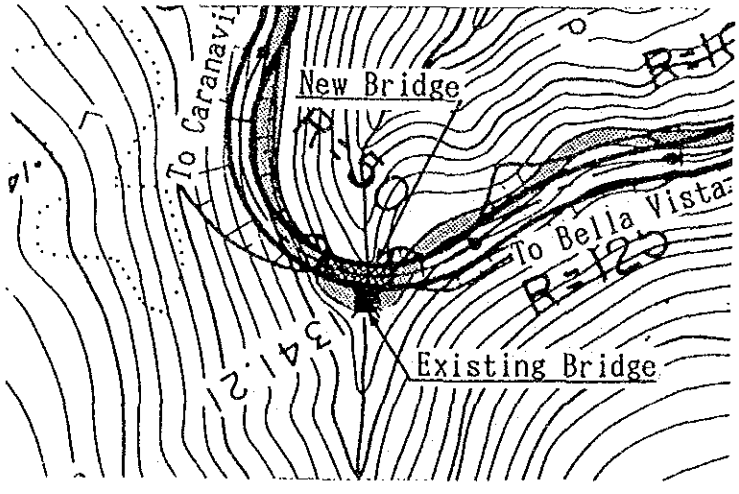
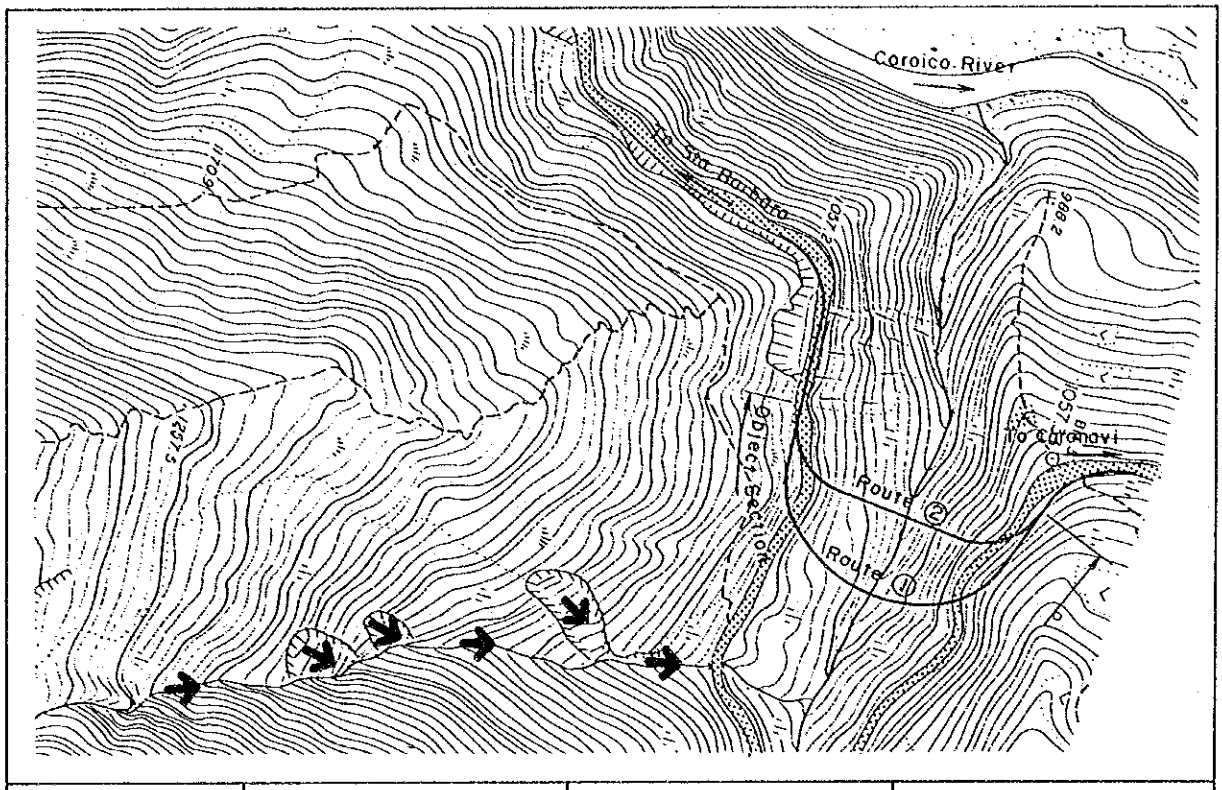


Fig 4.4-4 Location of Avaroa Bridge

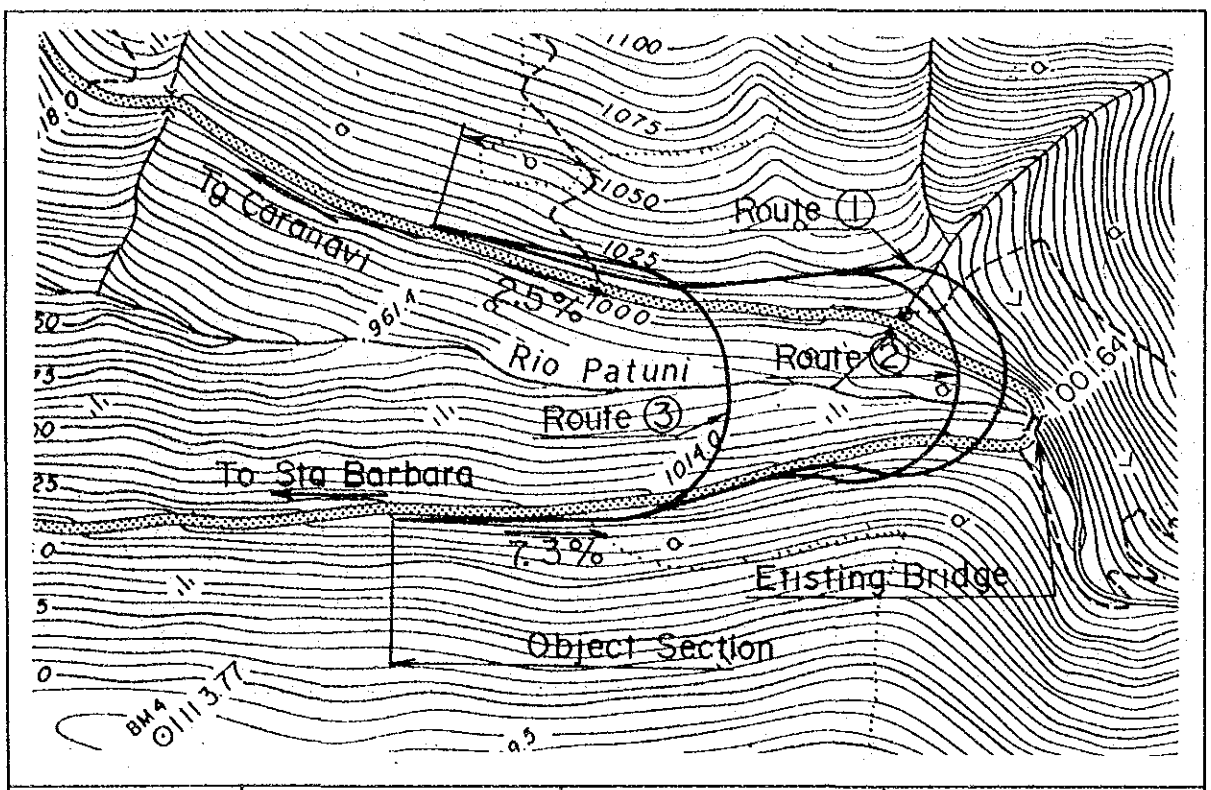
Table.4.4- 2 (1) Point A



Route	①	②	③
Length of Route	300m	220m	
Radius of Curvature	50-100-50m	50- -50m	
Volume of Excavation	34600m³	48300m³	
Bridge Length	130m	135m	
Depth of Valley	45m	50m	
Ease of Construction	Not Easy	Easy	
Rough Cost Estimation unit:$1000	$ 2230 (1.1)	$ 2090 (1.0)	
Selected Route	Route 2		
Reasons for Selection	1. Almost same bridge length 2. Small volume of excavation 3. Direct route and easy construction 4. Almost equal cost		

Brackets "( )" indicate the expansion rate,compared with the lowest route.

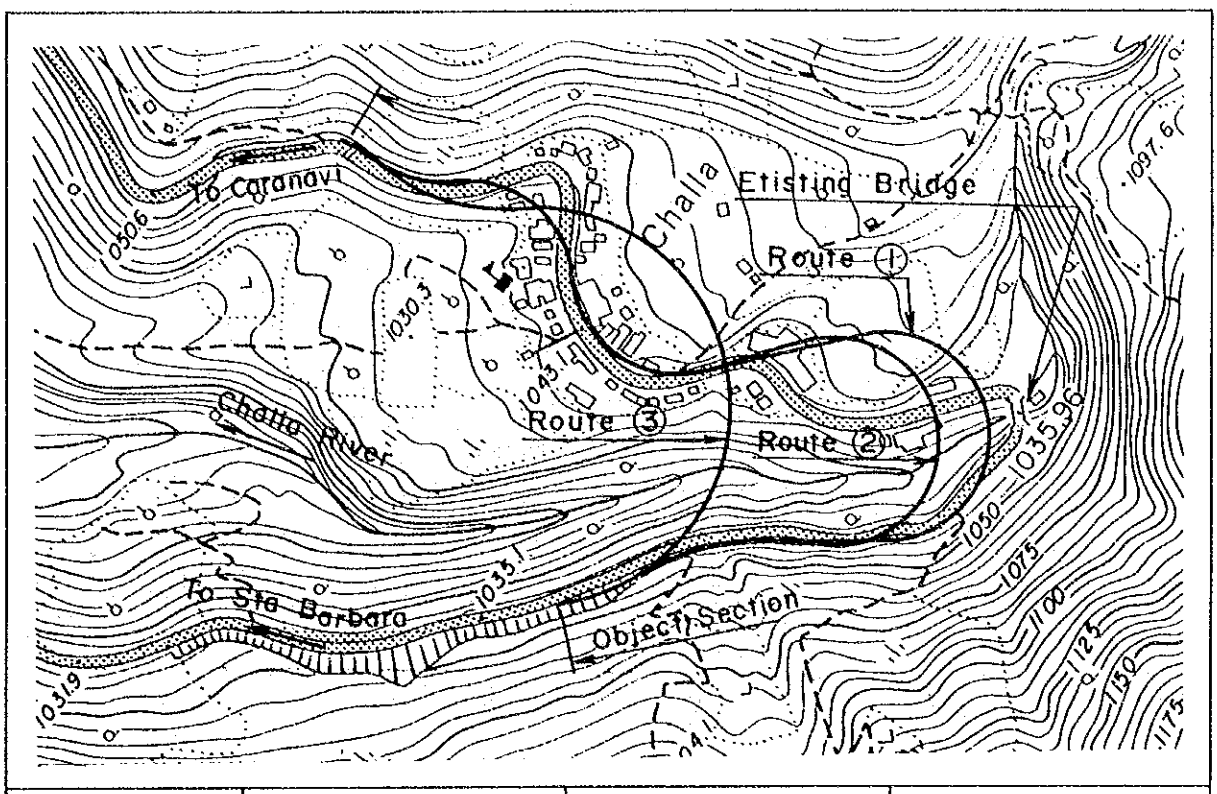
Table 4.4-2(2) Patuni Bridge



Route	①	②	③
Length of Route	640m	395m	380m
Radius of Curvature	50m	50m	50-100-50m
Volume of Excavation	86100m³	39100m³	17200m³
Bridge Length	30m	45m	90m
Depth of Valley	10m	12.5m	30m
Ease of Construction	Easy	Easy	Not Easy
Rough Cost Estimation unit:$1000	$ 1490 (1.3)	$ 1100 (1.0)	$ 1660 (1.5)
Selected Route	Route 2 (45m downstream from the existing bridge)		
Reasons for Selection	1. Small volume of excavation 2. Lowest cost 3. Same ease of construction as route 1		

Brackets "( )" indicate the expansion rate,compared with the lowest route.

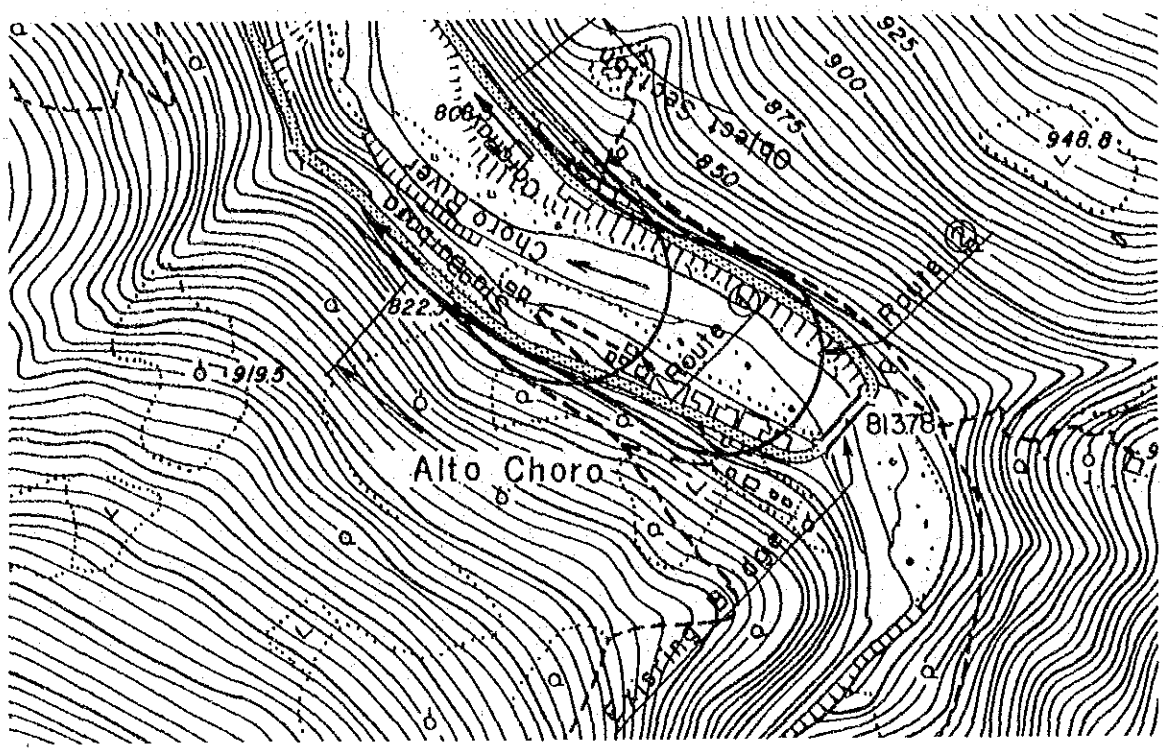
Table 4.4- 2 (3) Challa Bridge



Route	①	②	③
Length of Route	620m	580m	400m
Radius of Curvature	50m	50m	100m
Volume of Excavation	35740m³	22360m³	21310m³
Bridge Length	30m	60m	110m
Depth of Valley			
Ease of Construction	Easy	Easy	Not Easy
Rough Cost Estimation unit:$1000	$ 750 (1.0)	$ 880 (1.17)	$ 1670 (2.23)
Selected Route	Route 1 (15m downstream from the existing bridge)		
Reasons for Selection	1. Shortest bridge length 2. Lowest cost 3. Easy construction work		

Brackets "( )" indicate the expansion rate,compared with the lowest cost.

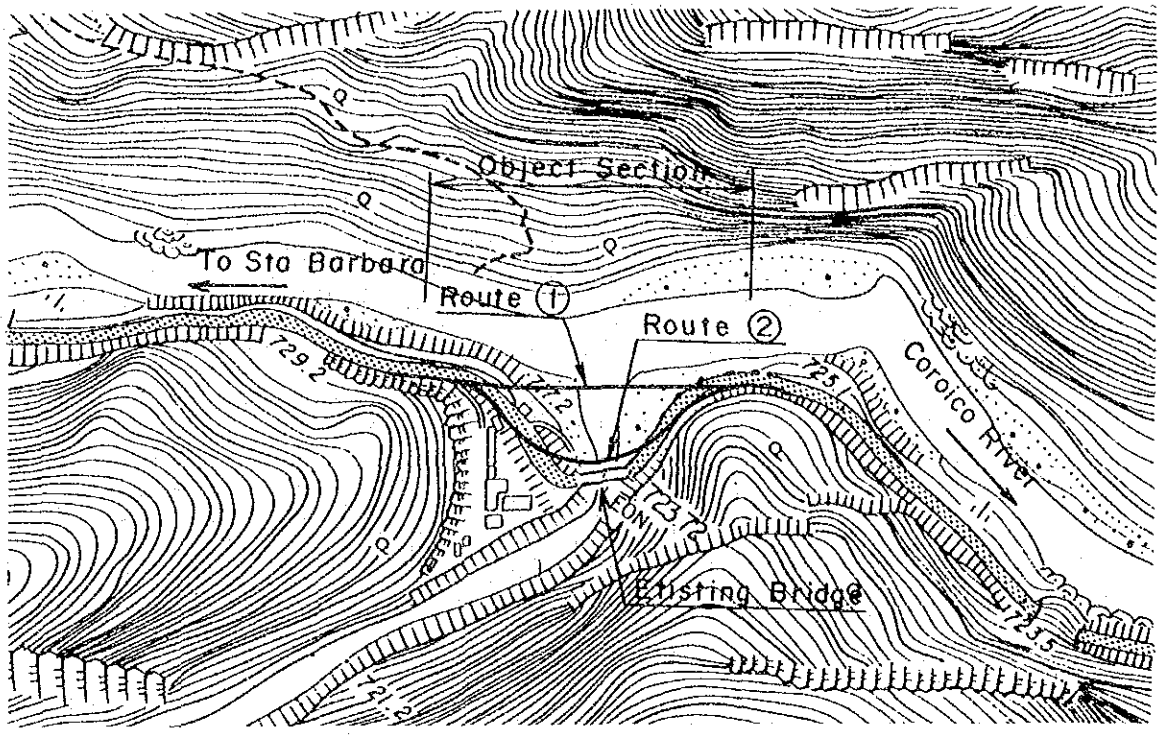
Table 4.4-2 (4) Alto Choro(2) Bridge



Route	①	②	③
Length of Route	300m	460m	
Radius of Curvature	50m	50m	
Volume of Excavation	25770m³	35900m³	
Bridge Length	70m	50m	
Depth of Valley	12m	8m	
Ease of Construction	Not Easy	Easy	
Rough Cost Estimation unit:$1000	$ 980 (1.02)	$ 960 (1.00)	
Selected Route	Route 2 (20m downstream from the existing bridge)		
Reasons for Selection	1. Gentle vertical gradient (Refer to Note below) 2. Easy construction work 3. Almost equal cost Note: The proposed vertical gradient of the new bridge becomes more gentle when it is as near the existing bridge as possible, because of the steep vertical gradient of the existing road as shown in ( )of the above Figure.		

Brackets "( )" indicate the expansion rate,compared with the lowest cost.

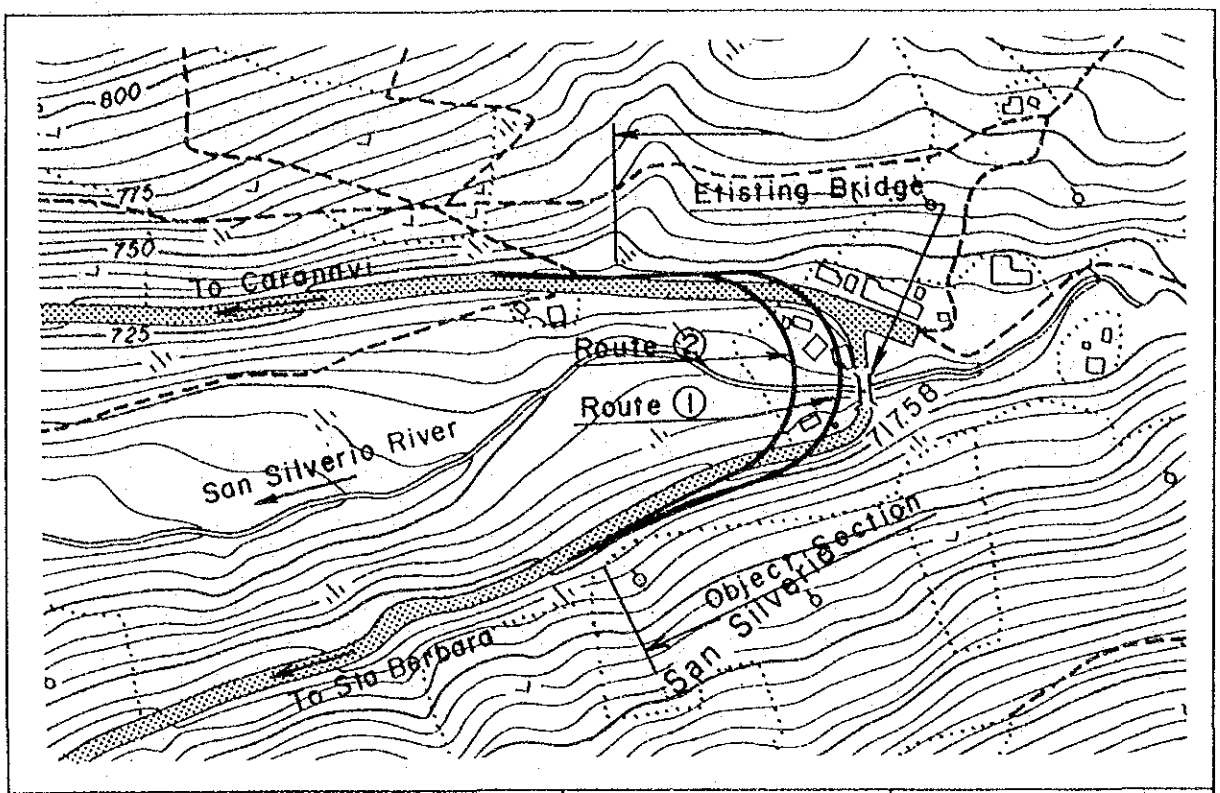
Table 4.4-2 (5) Pto Leon Bridge



Route	①	②	③
Length of Route	160m	180m	
Radius of Curvature		50m	
Volume of Excavation	8647m³	23060m³	
Bridge Length	55m	30m	
Depth of Valley			
Ease of Construction	Easy	Not Easy	
Rough Cost Estimation unit:$1000	$ 550 (1.00)	$ 610 (1.11)	
Selected Route	Route 1		
Reasons for Selection	1. Small volume of excavation 2. Smooth horizontal road alignment 3. Easy construction work besause of shallow water 4. Almost equal cost		

Brackets "( )" indicate the expansion rate,compared with the lowest cost.

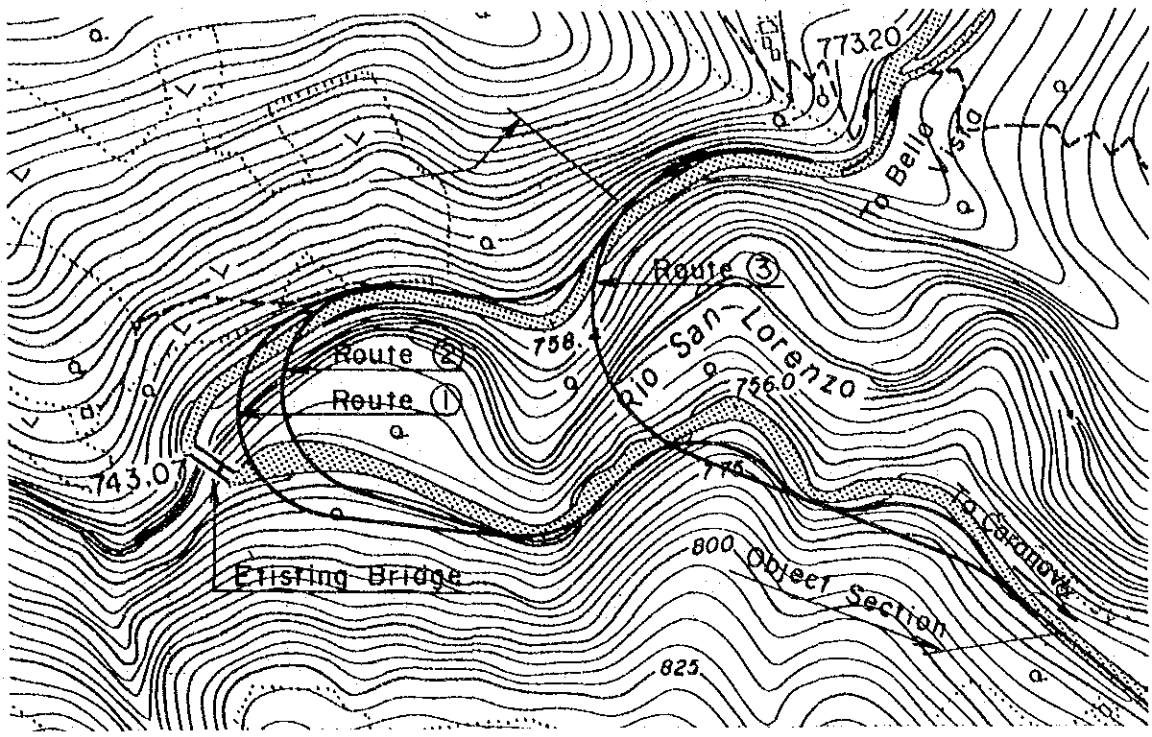
Table.4.4-2 (6) San Silverio Bridge



Route	①	②	③
Length of Route	280m	240m	
Radius of Curvature	50m	50m	
Volume of Excavation	9980m³	4264m³	
Bridge Length	40m	50m	
Depth of Valley	12.0m	13.0m	
Ease of Construction	Easy	Easy	
Rough Cost Estimation unit:$1000	$ 540 (1.03)	$ 520 (1.0)	
Selected Route	Route 2 (30m downstream from the existing bridge)		
Reasons for Selection	1. Small volume of excavation 2. No removal of houses 3. Almost equal cost 4. No problem for construction work		

Brackets "( )" indicate the expansion rate,compared with the lowest cost.

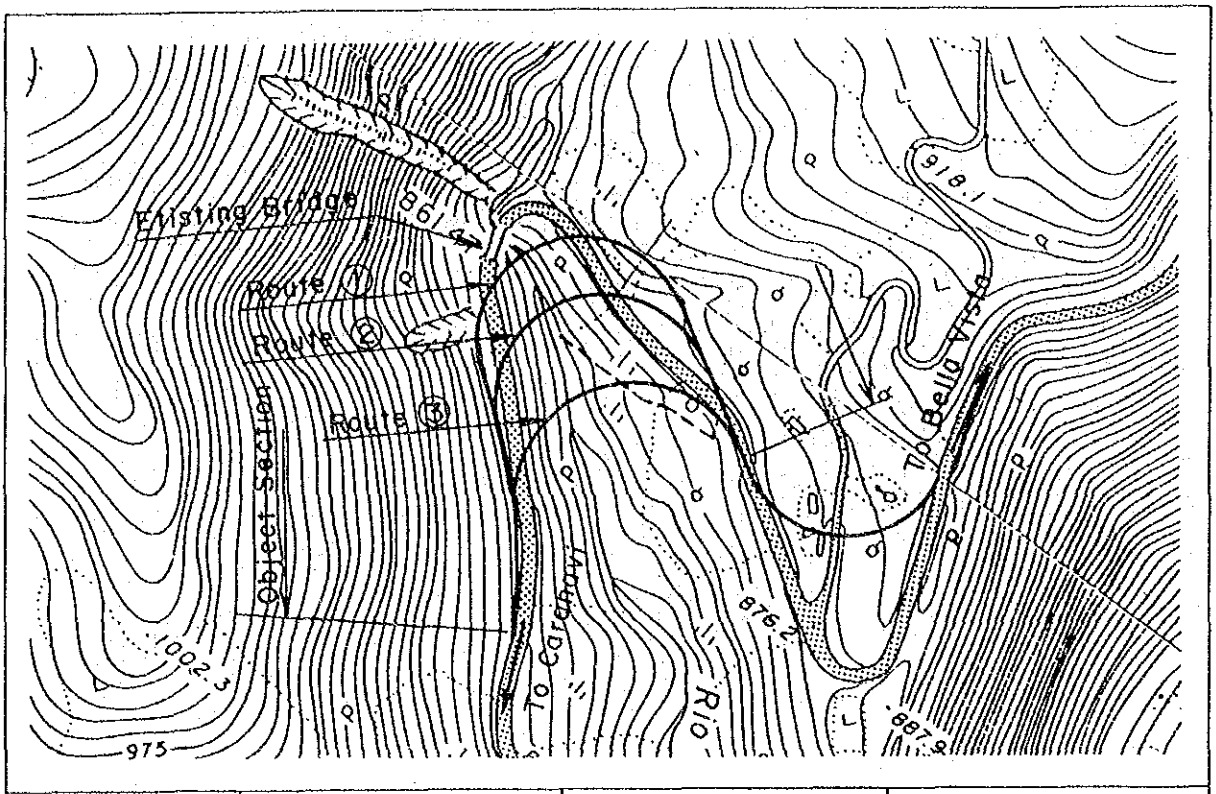
Table 4.4- 2 (7) San Lorenzo Bridbe



Route	①	②	③
Length of Route	800m	760m	430m
Radius of Curvature	50m	50m	100m
Volume of Excavation	174600m³	112660m³	67340m³
Bridge Length	50m	60m	130m
Depth of Valley	15m	15m	30m
Ease of Construction	Easy	Easy	Not Easy
Rough Cost Estimation unit:$1000	$ 2470 (1.31)	$ 1880 (1.00)	$ 2510 (1.34)
Selected Route	Route 2 (50m downstream from the existing bridge)		
Reasons for Selection	1. Lowest cost 2. Smaller volume of excavation than route 1 3. Easy construction work like route 1		

Brackets "( )" indicate the expansion rate,compared with the lowest cost.

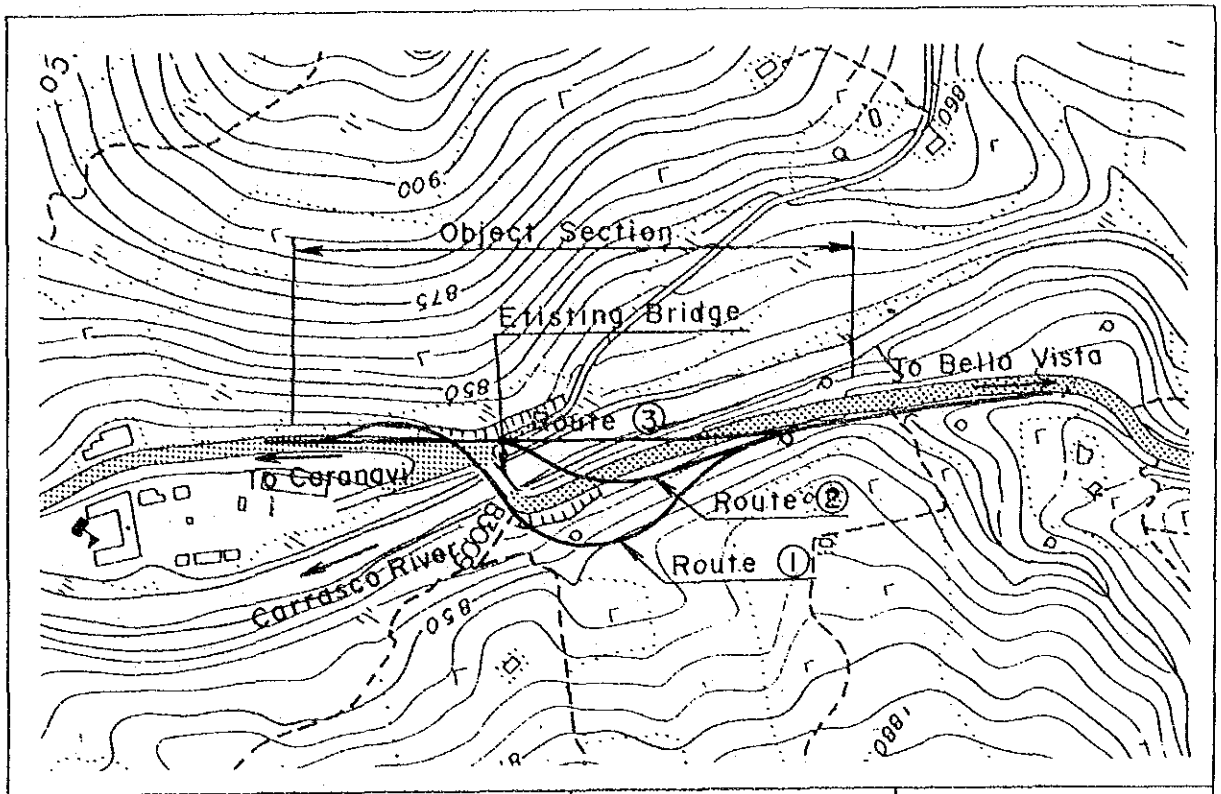
Table 4.4- 2 (8) Espiritu Bridge



Route	①	②	③
Length of Route	360m	300m	215m
Radius of Curvature	50m	50m	100m
Volume of Excavation	31290m³	10350m³	2400m³
Bridge Length	35m	50m	100m
Depth of Valley	15m	15m	20m
Ease of Construction	Easy	Easy	Not Easy
Rough Cost Estimation unit:$1000	$ 730 (1.13)	$ 640 (1.00)	$ 1430 (2.20)
Selected Route	Route 2 (40m downstream from the existing bridge)		
Reasons for Selection	1. Lowest cost 2. Smaller volume of excavation than route 1 3. Same ease of construction as route 1		

Brackets "( )" indicate the expansion rate,compared with the lowest cost.

Table 4.4-2 (9) Carrasco Bridge



Route	① *	②	③
Length of Route	310m	280m	265m
Radius of Curvature		60-60m **	
Volume of Excavation	41100m³	5470m³	4290m³
Bridge Length	23m	30m	65m
Depth of Valley	5.0m	5.0m	5.0m
Ease of Construction	Not Easy	Easy	Easy
Rough Cost Estimation unit:$1000	$ 540 (1.45)	$ 370 (1.00)	$ 490 (1.32)
Selected Route	Route 2 (30m downstream from the existing bridge)		
Reasons for Selection	1. Lowest cost 2. Smalle volume of excavation 3. Easy construction work		

Note : * Use of the existing bridge
** S-Curve
Brackets "( )" indicate the expansion rate,compared with the lowest cost.

Table 4.4-3 Summary of Required New Bridges

Name of Bridge	Name of River	Location of Bridge	Length of Bridge	Horizontal Curvature
Point (A)	---	250 m downstream from exist. brid.	L=132.5 m	R=50- -50 m
Patuni	Patuni	45 m downstream from exist. brid.	L=40 m	R=50 m
Challa	Challa	15 m downstream from exist. brid.	L=20 m	R=50 m
Cascada	Cala Cala	the same location as exist. brid.	L=18.5 m	R=1200 m
Alto Choro	Choro	20 m downstream from exist. brid.	L=50 m	R=50 m
Pto. Leon	Quitacarzon	30 m downstream from exist. brid.	L=75 m	straight
Cajones	Cajones	the same location as exist. brid.	L=25 m	R=400 m
Chojña	Chojña	the same location as exist. brid.	L=22 m	straight
San Silverio	San Silverio	30 m downstream from exist. brid.	L=50 m	R=50 m
San Lorenzo	San Lorenzo	50 m downstream from exist. brid.	L=52 m	R=50 m
Espiritu	Espiritu	40 m downstream from exist. brid.	L=52 m	R=50 m
Carrasco	Carrasco	20 m upstream from exist. brid.	L=30 m	R=60 m S-curve
Avaroa	Mula Jihuata	15 m downstream from exist. brid.	L=25 m	R=50 m

4.5 Facilities Required for Disaster Prevention

In this section, studies on the selection of optimum countermeasures for the classified disasters are presented. The studies were conducted in accordance with the following flowchart (Fig.4.5-1). Each study consideration is described in the section numbered on the right hand side of each box.

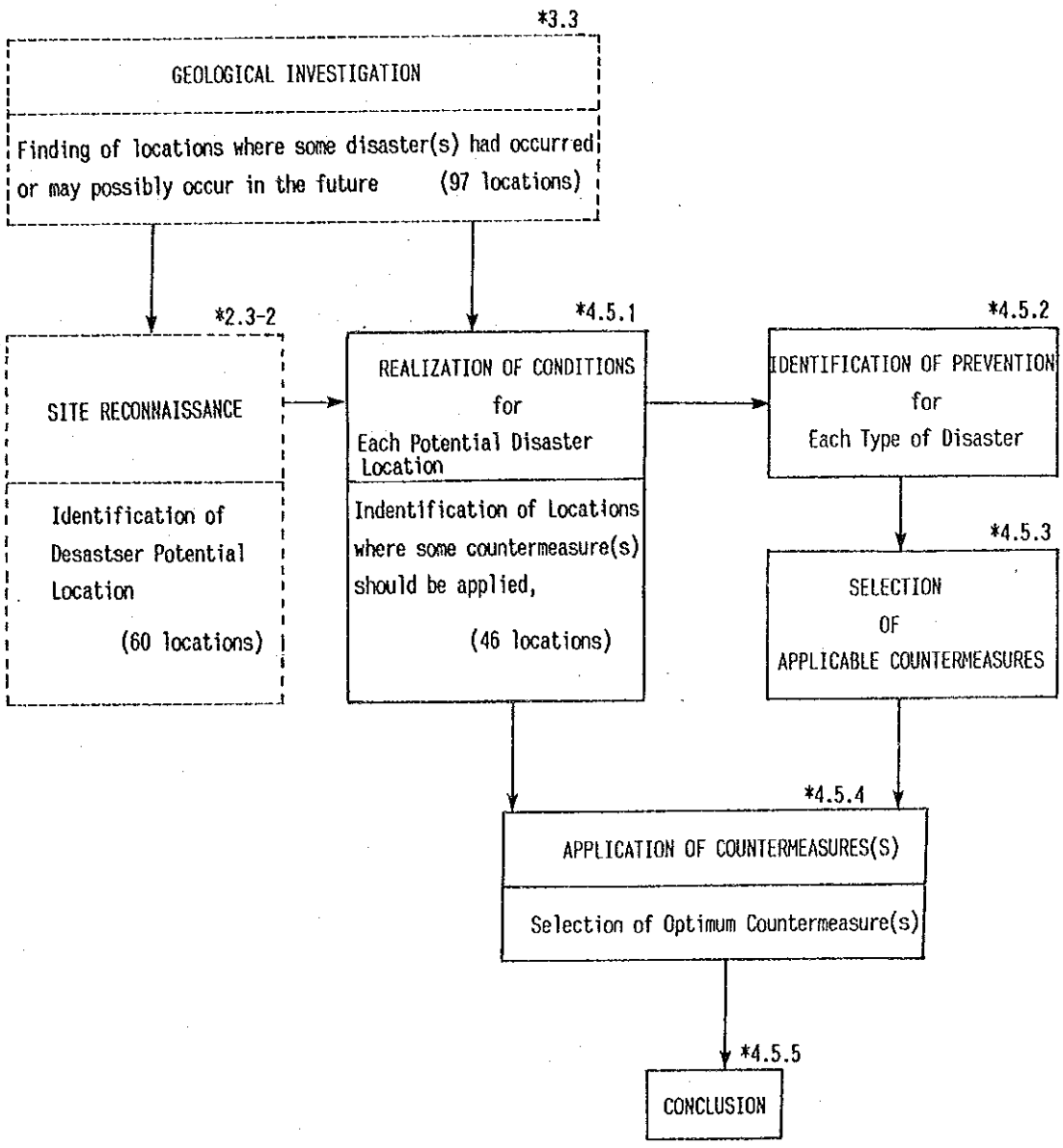


Fig. 4.5-1 Methodology for Countermeasure Selection

As presented in Fig. 4.5-1, prior to starting this section, the studies indicated with in the dotted line were conducted. The purpose was to identify potential disaster locations.

That is;

At first, as described in Chapter 3 GEOLOGICAL INVESTIGATION, all places where some disaster had occurred or thought to possibly occur in the future were selected and graded by stability as follows:

Grade	I	: stable	37 locations
	II	: unstable when it rains	41 locations
	III	: unstable	19 locations
		Total	97 locations

Next, as described in 2.3.2, Results of Site Reconnaissance, after evaluating the condition and the features at each location, the followings were confirmed and identified:

- Since the Grade I location were confirmed to be evidently stable, it is considered that they don't have any disaster potential.
- The Grade II and III locations were confirmed to have a disaster potential.

Thus, it was nominated that Grade II and III locations corresponded to potential disaster points.

In section 4.5.1, Realization of Conditions, the locations where some countermeasures should be applied, are identified out of the potential disaster points, that is to say, Grade II and Grade III locations.

4.5.1 Realization of Conditions for Each Potential Disaster Location

The location of each identified potential disaster point is illustrated on Fig. 4.5-2 and those conditions are clarified using geological investigation and site reconnaissance as presented in Table 4.5-1.

In the third column of the table , stability grades for the locations are indicated. For Grade II it is assumed that a disaster may occur during heavy rain and for Grade III it is assumed that a disaster may occur frequently regardless of whether.

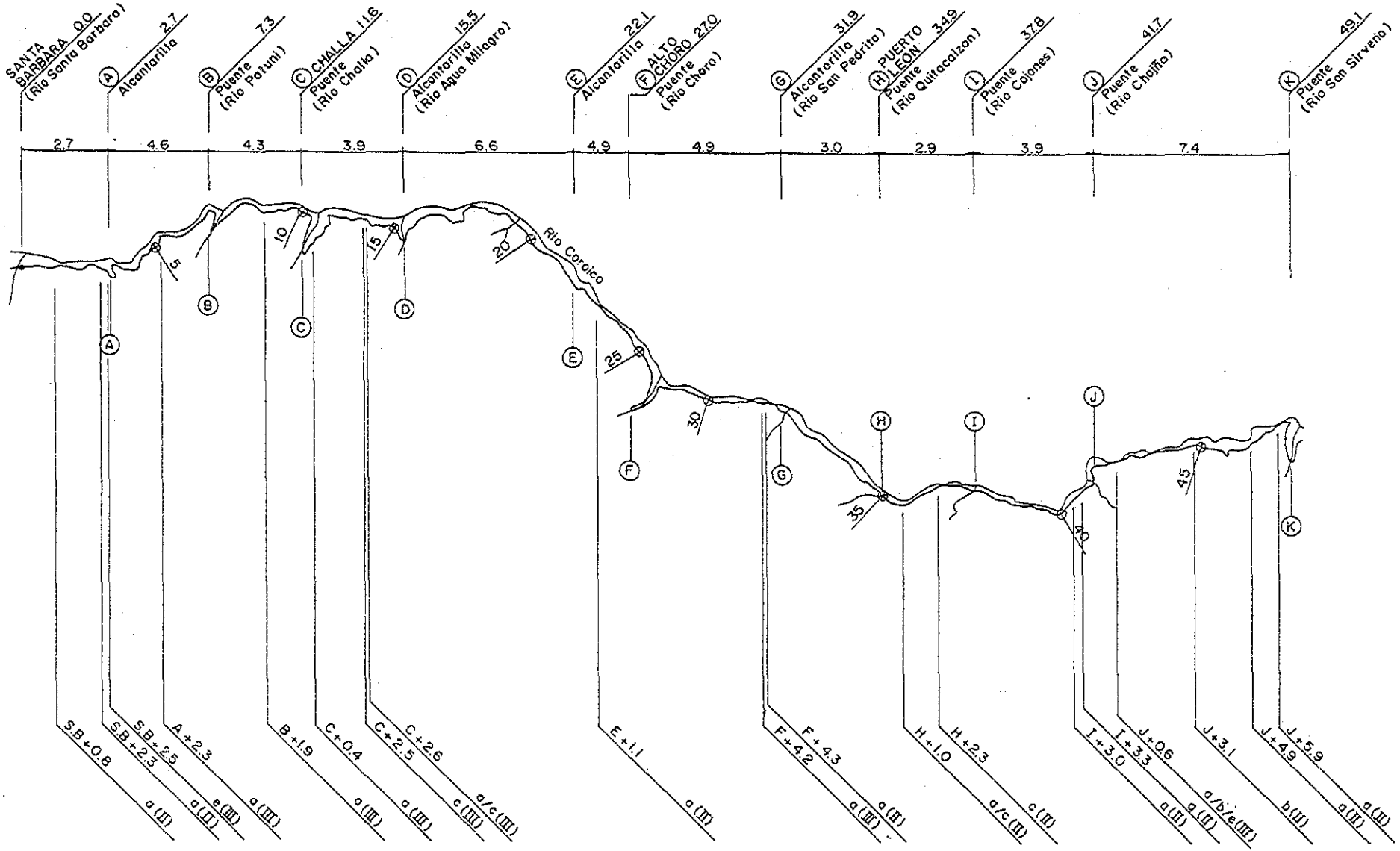


Fig. 4.5-2(1) POTENTIAL DISASTER LOCATIONS

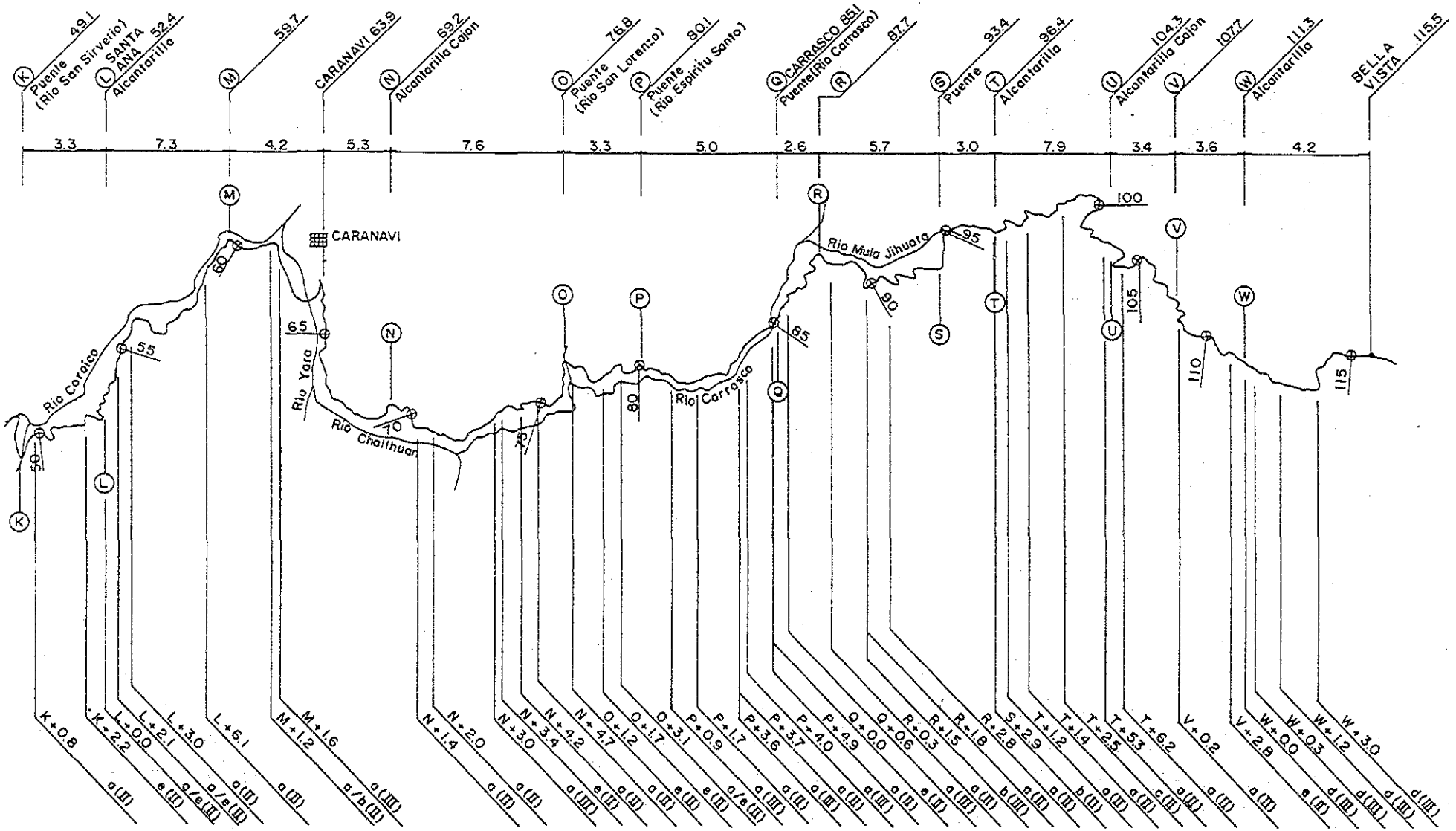


Fig. 4.5-2(2) POTENTIAL DISASTER LOCATIONS

Table 4.5-1 Conditions of Potential Disaster Location-(1)

Station		Type of Disaster 3)					Stability 5)	Material	Site Condition				Slope Gradient			Remarks
Existing 2) Alignment	New Alignment	a	b	c	d	e	Grade		Seepage 6) Water	Scale of 7) Boulders	Clearance 8)	Area of 9) debris/ earthflow	Natural	Cut	Aft. Improv.	
S.B +0.8	No. 0+700	●					II	Soil w/gravel	—				45°	60°	1.5:1	(56°)
S.B +2.3	No. 2+200	●					II	-ditto-	—				45°	60°	1.5:1	
S.B +2.5* 1)	—					●* 4)	III	-ditto-	—			L	30°	—	1.5:1	A better detour than the existing road was found.
A +2.3	No. 4+375	●					II	unsound rock	—				40°	45°	2.0:1	(63°)
B +1.9	No. 8+100	●					III	-ditto-	—				60°	80°	2.0:1	
C +0.4	No.10+900	●					III	soil w/gravel	—				40°	70°	1.5:1	
C +2.5	No.12+600			●			III	unsound rock	—	B	—		60°	—	2.0:1	
C +2.6	No.12+780	●		▲			III	-ditto-	—	B	—		60°	70°	2.0:1	
E +1.1	No.21+200	●					II	-ditto-	—				—	—	2.0:1	
F +4.2	No.29+500	●					III	soil w/gravel	—				35°	55°	1.5:1	
F +4.3	No.29+600	●					II	unsound rock	—				—	—	2.0:1	
H +1.0	No.33+700	●		▲			II	-ditto-	○	C	—		—	60°	2.0:1	
H +2.3	No.35+560			●			II	-ditto-	—	B	—		—	70°	2.0:1	
I +3.0	No.38+740	●					II	soil w/gravel	—				30°	60°	1.5:1	
I +3.3	No.39+ 30	●					III	unsound rock	—				40°	65°	2.0:1	
J +0.6	No.40+300	●	▲*			▲	III	-ditto-	—			L	60°	90°	2.0:1	Disaster "b" would be solved by cut slope protection work.
J +3.1*	—		●*				II	soil w/gravel	—				40°	55°	1.5:1	The location would be avoided by means of improvement of horizontal alignment.
J +4.9	No.44+400	●					II	-ditto-	—				60°	55°	1.5:1	
J +5.9	No.45+230	●					II	unsound rock	—				60°	55°	2.0:1	
K +0.8	No.47+520	●					II	-ditto-	—				40°	55°	2.0:1	

Notes: 1) The location indicated with an asterisk on left side of this table, means the location where would not be nesessary to consider any countermeasures for the reason described in the remarks.
2) Stations indicated according to nominal of milestones on the existing road defined in "2.3.2(1)".
3) a:Cut slope failure, b:Embankment slope failure, c:Rock fall, d:Landslide, e:Debris/earth flow. ●:Main disaster. ▲ : Subordinate disaster(S)
4) The type of disaster indicated with an asterisk, means the disaster which would not be necessary to consider any countermeasures for the reason described in the remarks.
5) II :Unstable when it rains, III :Unstable
6) The location indicated with a circle, means the location where remarkable seepage water would occur. The location indicated with a triangle, means the location where little seepage water would occur.
7) The condition should be consider for rock falls. Each location shall be graded by the scale of boulders which would fall. A:Large scale, B:Medium Scale, C:Small scale
8) The condition should be consider for rock falls. 1:The spot where suffifient road side clearance is possible, 2:the spot where sufficient clearance is possible on top of the slope
9) The condition should be consider for debris/earth flow. L:The spot where large debris/earth flow area would be considered, S:The spot where small debris/earth flow area would be considered.

Table 4.5-1 Conditions of Potential Disaster Location-(2)

Station		Type of Disaster 3)					Stability 5)	Material	Site Condition				Slope Gradient			Remarks
Existing Alignment 2)	New Alignment	a	b	c	d	e	Grade		Seepage Water 6)	Scale of Boulders 7)	Clearance 8)	Area of debris/ earthflow 9)	Natural	Cut	Aft. Improv.	
K +2.2	No.49+210					●	II	Soil w/gravel	△			S	15°	55°	1.5:1	
L +0.0*1)	—	*4) ▲				*●	II	soil	—			L	50°	45°	1.0:1	The location would be avoided by means of improvement of vertical alignment.
L +2.1*	—	*▲				*●	II	-ditto-	—			L	45°	60°	1.0:1	(45°) -ditto-
L +3.0	No.52+200	●					II	-ditto-	—				20°	60°	1.0:1	
L +6.1	No.55+500	●					II	soil w/gravel	—				45°	50°	1.5:1	
M +1.2	No.57+750	●	*▲				II	unsound rock	—				40°	50°	2.0:1	Disaster "b" would be solved by means of improvement of horizontal alignment.
M +1.6	No.58+200	●					III	soil	—				20°	—	1.0:1	
N +1.4	No.66+ 0	●					II	unsound rock	—				30°	60°	2.0:1	
N +2.0	No.66+500	●					II	soil w/gravel	—				30°	60°	1.5:1	
N +3.0	No.68+440	●					III	soil	—				25°	60°	1.0:1	
N +3.4	No.68+820					●	II	-ditto-	△			S	25°	70°	1.0:1	
N +4.2*	—	*●					II	-ditto-	—				—	—	1.0:1	The location would be avoided by means of improvement of horizontal alignment.
N +4.7	No.70+100	●					II	-ditto-	—				—	65°	1.0:1	
O +1.2	No.71+800					●	II	unsound rock	—			S	—	—	2.0:1	
O +1.7*	—					*●	II	soil	—			S	—	—	1.0:1	The location would be avoided by means of improvement of vertical alignment.
O +3.1	No.73+890	*▲				●	II	-ditto-	—			S	—	—	1.0:1	Disaster "a" would be solved by means of improvement of horizontal alignment.
P +0.9	No.75+570	●					III	unsound rock	—				—	63°	2.0:1	
P +1.7	No.76+320	●					II	-ditto-	—				60°	—	2.0:1	Spalling of slope surface would occur.
P +3.6	No.77+800	●					III	soil	△				15°	—	1.0:1	
P +3.7	No.78+100	●					II	-ditto-	—				20°	50°	1.0:1	

Notes:
1) The location indicated with an asterisk on left side of this table. means the locations where would not be nesessary to consider any countermeasures for the reason described in the remarks.
2) Stations indicated according to nominal of milestones on the existing road defined in "2.3.2(1)".
3) a:Cut slope failure.b:Embankment slope failure.c:Rock fall.d:Landslide.e:Debris/earth flow. ●:Main disaster. ▲ : Subordinate disaster(S)
4) The type of disaster indicated with an asterisk.means the disaster which would not be necessary to consider any countermeasures for the reason described in the remarks.
5) II :Unstable when it rains.III :Unstable
6) The location indicated with a circle. means the location where remarkable seepage water would occur.The location indicated with a triangle. means the location where little seepage water would occur.
7) The condition should be consider for rock falls.Each location shall be graded by the scale of boulders which would fall.A:Large scale.B:Medium Scale.C:Small scale
8) The condition should be consider for rock falls.1:The spot where suffifient road side clearance is possible.2:the spot where sufficient clearance is possible on top of the slope
9) The condition should be consider for debris/earth flow.L:The spot where large debris/earth flow area would be considered.S:The spot where small debris/earth flow area would be considered.

Table 4.5-1 Conditions of Potential Disaster Locations-(3)

Station		Type of Disaster[5]					Stability[6]	Material	Site Condition				Slope Gradient			Remarks
Existing[2] Alignment	New Alignment	a	b	c	d	e	Grade		Seepage[6] Water	Scale of[7] Boulders	Clearance[8]	Area of[9] debris/ earthflow	Natural	Cut	Aft. Improv.	
P +4.0	No.78+600	●					III	unsound rock	—				20°	40°	2.0:1	
P +4.9	No.79+500	●					II	soil	—				35°	45°	1.0:1	
Q +0.0[1]	—					*●[4]	II	-ditto-	—			S	40°	60°	1.0:1	The location would be avoided by means of improvement of horizontal alignment.
Q +0.6	No.80+350	●					III	-ditto-	—				40°	60°	1.0:1	
R +0.3	No.82+600	●					II	soil w/gravel	—				40°	60°	1.5:1	
R +1.5*	—		*●				III	unsound rock	—				45°	60°	2.0:1	The location would be avoided by means of improvement of horizontal alignment.
R +1.8	No.84+350	●					II	soil w/gravel	—				—	—	1.5:1	
R +2.8*	—	*●					II	-ditto-	—				50°	80°	1.5:1	The location would be avoided by means of improvement of horizontal alignment.
S +2.9*	—		*●				II	unsound rockl	△				70°	—	2.0:1	The location would be avoided by means of improvement of vertical alignment.
T +1.2*	—	*●					II	soil w/gravel	△				—	—	1.5:1	-ditto-
T +1.4*	—	*●					II	-ditto-	—				35°	60°	1.5:1	-ditto-
T +2.5	No.92+900			●			II	unsound rock	—	B	—		60°	80°	2.0:1	
T +5.3*	—	*●					II	-ditto-	○				55°	70°	2.0:1	The location would be avoided by means of improvement of vertical alignment.
T +6.2*	—	*●					II	soil w/gravel	○				70°	—	1.5:1	-ditto-
V +0.2	No.101+475	●					II	unsound rock	—				—	40°	2.0:1	
V +2.8	No.103+190					●	II	soil w/gravel	—			S	20°	—	1.5:1	
W +0.0	No.104+ 20				●		III	soil	—				10°	45°	1.0:1	
W +0.3	No.104+670				●		III	-ditto-	—				10°	45°	1.0:1	
W +1.2	No.105+840				●		III	-ditto-	—				10°	45°	1.0:1	
W +3.0	No.107+500				●		III	-ditto-	△				15°	—	1.0:1	
No. of Spots		34	0	3	4	5										46:Total of Location Requiring Contermeasure(S)

Notes: 1) The location indicated with an asterisk on left side of this table, means the location where would not be nesessary to consider any countermeasures for the reason described in the remarks.
2) Stations indicated according to nominal of milestones on the existing road defined in "2.3.2(1)".
3) a:Cut slope failure.b:Embankment slope failure.c:Rock fall.d:Landslide.e:Debris/earth flow. ●:Main disaster. ▲ : Subordinate disaster(S)
4) The type of disaster indicated with an asterisk.means the disaster which would not be necessary to consider any countermeasures for the reason described in the remarks.
5) II :Unstable when it rains.III :Unstable
6) The location indicated with a circle, means the location where remarkable seepage water would occur.The location indicated with a triangle, means the location where little seepage water would occur.
7) The condition should be consider for rock falls.Each location shall be graded by the scale of boulders which would fall.A:Large scale.B:Medium Scale.C:Small scale
8) The condition should be consider for rock falls.1:The spot where suffifient road side clearance is possible.2:the spot where sufficient clearance is possible on top of the slope
9) The condition should be consider for debris/earth flow.L:The spot where large debris/earth flow area would be considered.S:The spot where small debris/earth flow area would be considered.

In other words, it can be said that Grade III locations have a higher disaster probability than Grade II. In the fourth column, slope material for each location is indicated. These were classified into four kinds of material on the basis of results from site reconnaissance. These were, soil, soil with gravel, unsound rock and sound rock. The slope gradients after improvement in the sixth column were determined corresponding to each kind of slope material as follows:

soil	- 1.0 : 1
soil with gravel	- 1.5 : 1
unsound rock	- 2.0 : 1
sound rock	- 4.0 : 1

All of the type f disasters, (fractured zone in disaster potential locations) are accompanied with a type a (slope failure), or type c (rock fall refer to Table 3.3-1), and prevention of these disasters would be possible by means of countermeasures for the main type of disaster. Hence, type f disasters were excluded beforehand from Table 4.5-1.

Identification of Locations where some countermeasures should be applied

All type b disaster locations can be avoided by means of horizontal or vertical alignment improvements. Furthermore, all of the subordinate type b disasters could also be avoided by means of horizontal alignment improvement or cut slope protection work. Thus, it would not be necessary to consider any countermeasures for type b disasters, if aligment improvements were made.

The places where landslides may possibly occur in the future are concentrated with in the Bella Vista region. Since this potential area is spread throughout vicinity, it would be impossible to find a route to construct a detour with the purpose of avoiding a possible damage by a landslide in the future. In addition to this, it has been confirmed in the course of the site investigation, that all potential locations along the existing road have been stabilized at present and have little possibility of suffering a serious landslide in the near future.

Considering these two points, the improvement of the road in this region should be carried out by widening and the

adjustment of the existing road alignment, as well as installing some appropriate preventive facilities against landslides.

Four locations where large scale debris/earth flow may possibly occur were found along the existing road. These locations are as follows:

- Santa Bårbara + 2.5 km (debris/earth flow is main disaster)
- Point J + 0.6 km (debris/earth flow is subordinate disaster)
- Point L + 0.0 km (debris/earth flow is main disaster)
- Point L + 2.1 km (debris/earth flow is main disaster)

Out of these locations above, Point L + 0 km and Point L + 2.1 km can be eliminated hereafter since route relocation to improve the vertical alignment in this area is indispensable and the relocated road, as a result, will pass far away from these two points.

With regards to the location of Santa Bårbara + 2.5 km, a better detour than the existing road was found. On the contrary, no detour for avoiding the location of Point J + 0.6 km was found, due to the steep topography.

A further, six "type A" disaster locations can be also avoided by means of horizontal or vertical alignment improvement.

As a conclusion, 46 locations were identified as places where some countermeasures should be applied.

4.5.2 Identification of Prevention for Each Type of Disaster

(1) Type "a" Disaster (Slope Failure)

As described in 2.3.2, "Results of Site Reconnaissance" and chapter 3, "Geological Investigation", all of the slope failures along the existing road are assumed to be surface failures. Thus, this type of disasters could be prevented by the means of some slope protection work(s) including retaining walls.

(2) Type "c" Disaster (Rock Fall)

All of the rock falls in locations requiring countermeasure(s) are assumed to have been caused by open cracks in the rocks. Thus, the disaster could be prevented by the means of some so called catch works.

(3) Type "d" Disaster (Landslide)

As a land slide (in general) occurs slowly, it is not difficult to find evidence of its beginning. Constant careful observation of the ground is the most important factor for decreasing road damage resulting from it. Further more, it would be indispensable to prevent permeation of ground water into sub-soil considering it is a principal cause of landslides.

(4) Type "e" Disasters (Debris/earth Flow)

Debris deposits evidently exist on the upstream side of the debris flow locations. However, since it is rather difficult to forecast the volume of the deposit which flows down from one disaster, catch work(s) which catch the entire debris deposit should be installed along the stream bed for prevention of this type of disaster.

4.5.3 Selection of Applicable Countermeasures

On the basis of identification of prevention for each type of disaster described in 4.5.2, the applicable countermeasures which have been practically adopted in other similar road construction, were selected. The applicable range of these countermeasures are presented in Table 4.5-2.

(1) Applicable Countermeasures for Type "a" Disasters

As presented in Table 4.5-2(1), the applicable ranges are indicated by stability grade, slope material, applicability for seepage water, slope gradient and slope height/length.

As described in 4.5.1, Realization of Conditions,grade III indicates that the possibility of the disaster is higher than grade II. Thus, the slope protection work which has a

firmer structure should be adopted for grade III locations. The other applicable ranges for each slope protection work were determined according to the structural characteristics and from practical experiences.

(2) Applicable Countermeasures for Type "c" Disasters

The applicable range for each of the catch works are presented in Table 4.5-2(2). As described in the table, each application mainly depends on the scale of the boulders.

(3) Applicable Countermeasure for Type "d" Disasters

Subsurface drainage work should be adopted to prevent the permeation of ground water into the sub-soil, considering the conditions for each specified landslide area.

(4) Applicable Countermeasures for Type "e" Disasters

For the locations where small debris/earth flow is considered, gabions or concrete dam(s) should be adopted depending on the probability of a disaster.

On the other hand, for Point J + 0.6 km it was considered difficult to find a detour because of the steep topography, although the spot has a considerably wide disaster potential area.
Considering the scale and the conditions, a flow shed should be installed, in order to completely avoid any damage by the flow.

4.5.4 Application of Countermeasure(s)

For the purpose of deriving applicable countermeasures according to the ranges specified in Table 4.5-2, a flowchart as shown in Fig. 4.5-3 was recommended.
The practical application of these countermeasure was conducted spot by spot according to the following procedures:

1) Derivation of applicable countermeasures using Fig. 4.5-3
2) Comparison of the derived countermeasures from the view point of cost and/or topographical condition

Table. 4. 5-2 (1) Summary of Applicable Countermeasures for Type "a" Disaster

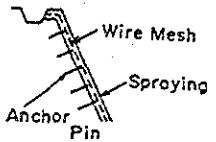
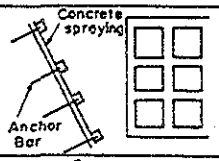
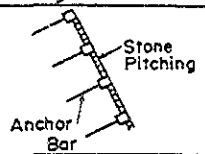
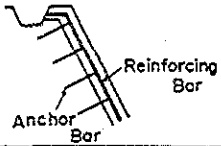
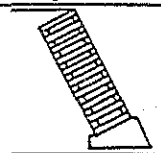
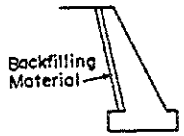
Type of Countermeasure	Illustration	Applicable Grade	Applicabe Material	Applicability for seepage water	Applicable Slope Gradient	Limit of Slope Height	Limit of Slope Length	Remarks
Type 1: Concrete Spraying		II	for unsound rock or sound rock	Not applicable for any seepage water	practically gentler than 4.0:1	—	—	It is most economical in all slope protection works. Thus, it should be applied. if the spot is adaptable these conditions.
Type 2: Stone Masonry Retaining wall		II	for soil or talus layer	Applicable for little seepage water	3.3:1 2.5:1 2.0:1	3.0m 5.0m 7.0m	— (height)	Adaptable to the spot where is advantageous from topographical view point.
Type 3: Concrete Crib with concrete spraying and Anchoring		II / III	for soil with gravel or unsound rock	Applicable for little seepage water	practically gentler than 2.0:1	— (length)	Practically less than 13.0m for a slope	
Type 4: Concrete Crib with stone pitching and Anchoring		II / III	for soil or soil with gravel or unsound rock	Applicable for remarkable seepage water	practically gentler than 1.0:1	— (length)	Practically less than 13.0m for a slope	Adaptable to the spot where remarkable seepage water would exist.
Type 5: Concrete Pitching and Anchoring		II / III	for soil or talus layer	Applicable for little seepage water	practically gentler than 1.0:1	— (length)	Practically less than 10.0m	
Type 6: Grid Type Concrete Retaining Wall		II / III	for soil or soil with gravel or unsound rock	Applicable for remarkable seepage water	3.3:1 → 2.5:1 → 2.0:1 →	18.0m 22.5m 30.0m	— (height)	Adaptable to the spot where is advantageous from topographical view point.
Type 7: Supported Type Concrete Retaining Wall		III	for soil or talus layer	Applicable for little seepage water	practically gentler than 2.0:1	practically lower than 15.0m	— (height)	- ditto -

Table. 4. 5-2 (2) Summary of Applicable Countermeasures for Type C Disaster

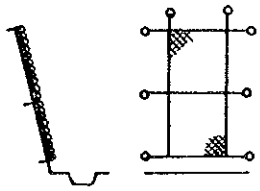
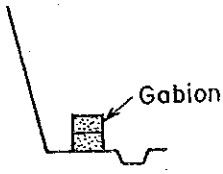
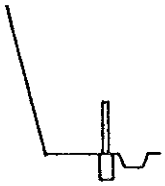
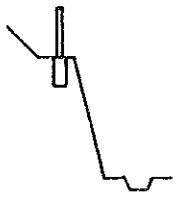
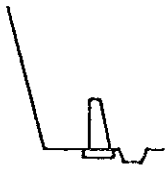
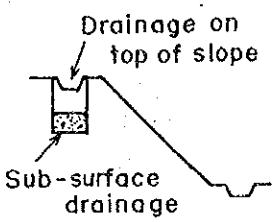
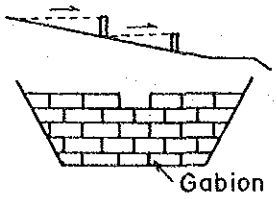
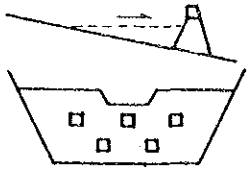
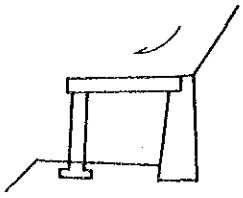
Type of Countermeasure	Illustration	Applicable Grade	Applicable Site Concition
Type 8: Catch Ditch		II	The spot where small scale of boulders would fall, and sufficient road side clearance is possible.
Type 9: Catch Netting		II / III	The spot where medium/small scale of boulders would fall, or spalling of slope surface (type A) would occur.
Type 10: Gabion Catch Wall		II	The spot where small scale of boulders would fall, or spalling of slope surface (type A) would occur.
Type 11: Catch Fence installed at road side		II / III	The spot where midium scale of boulders would fall.
Type 12: Catch Fence installed at top of slope		III	The spot where large scale of boulders would fall, and sufficient clearanceis possible on top of the slope.
Type 13: Concrete Catch Wall		III	The spot where large scale of boulders would fall.

Table. 4. 5-2 (3) Summary of Countermea for Pyped, Type d or Type e Disaster

Type of Countermeasure	Illistration	Applicable Grade	Applicable Site Concition
Type 14: Sub-surface Drainage for Lamdslide		II / III	Applicable to the spot where little seepage water would occur.
Type 15: Gabion Dam for Debris/earth flow		II	Applicable to the spot where debris/earth flow would occur when it rains.
Type 16: Concrete Dam for Debris/earth flow		III	Applicable to the spot where frequent debris/earth flow would occur.
Type 17: Debris/earth flow shed		II / III	Applicable to the spot where installation of clam(s) would be difficult becaure of steep and narrow stream bed and sufficient road side clearance would not be secured.

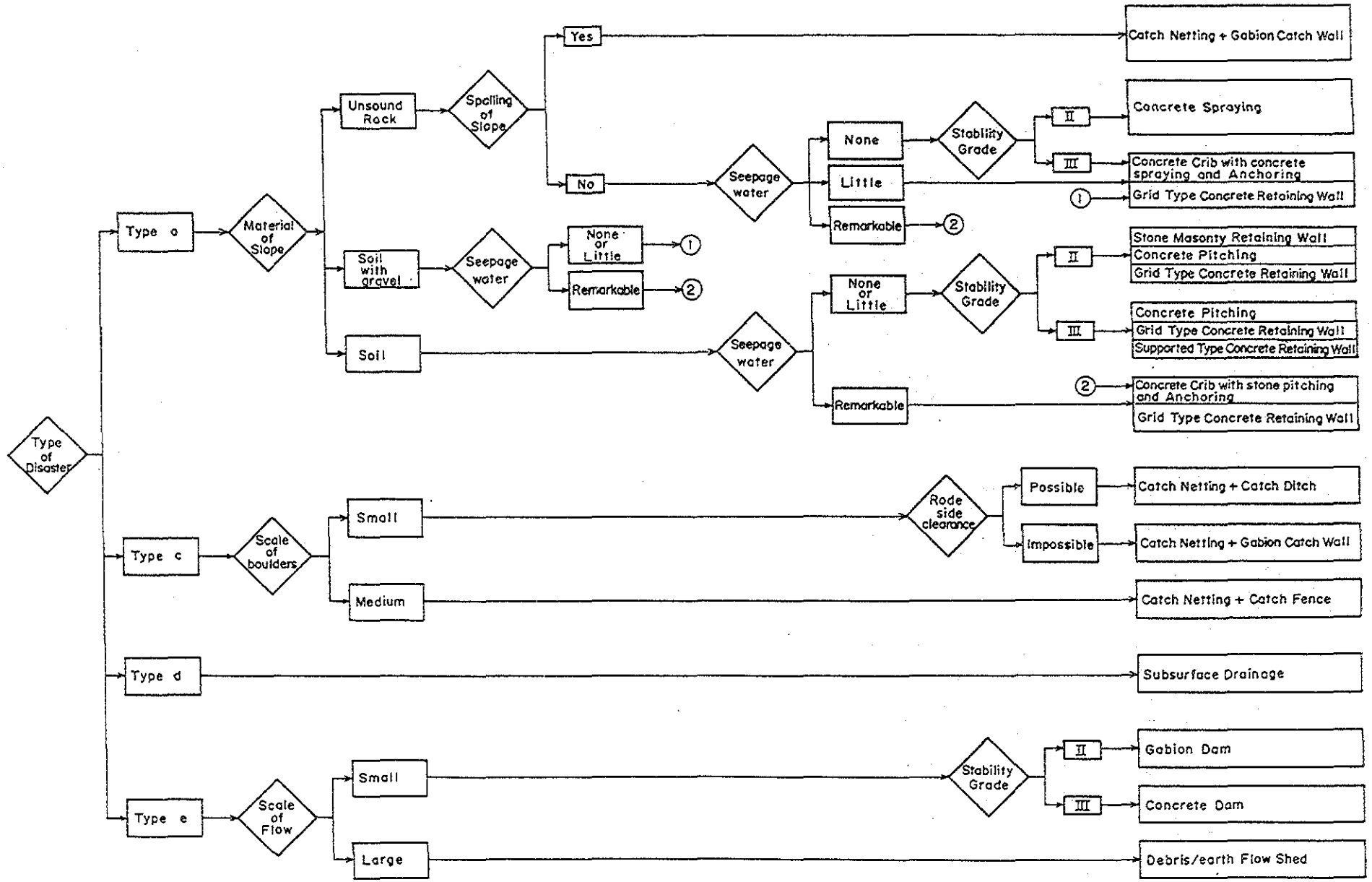


Fig. 4.5-3 APPLICATION OF COUNTERMEASURE (S)

3) Selection of optimum countermeasures

The results are tabulated in Table 4.5-3. Furthermore, detailed descriptions regarding the comparison and selection are presented in Appendix 4-4.

4.5.5 Conclusion

As a result of the selection of the optimum countermeasures, 11 types out of all the possible countermeasures were adopted. These countermeasures and the number used are tabulated in Table 4.5-4. In conclusion, 5-type slope protection work for slope failure, 3-type catch work for rock fall, 1-type prevention work for landslide, and 1-type catch work and 1-type prevention work, for debris/earth flow were adopted respectively.

Table 4.5-3 (1) Comparison of Countermeasure(s)

Spot												
	1) Station on Existing Alignment	S.B+0.8	S.B+2.3	A +2.3	B +1.9	C +0.4	C +2.5	C +2.6	E +1.1	F +4.2	F +4.3	H +1.0
	Station on New Alignment	No.0+700	No.2+200	No.4+375	No.8+100	No.10+900	No.12+600	No.12+780	No.21+200	No.29+500	No.29+600	No.33+700
2) Disaster type and the Grade		A(II)	A(II)	A(II)	A(III)	A(III)	C(III)	A/C(III)	A(II)	A(III)	A(II)	A/C(II)
Type 1	Concrete Spraying			●			● 5)		●		●	
Type 2	Stone Masonry Retaining Wall											
Type 3	Concrete Crib with concrete spraying and Anchoring	△ ● 3)	●		●	●		●		●		
Type 4	Concrete Crib with stone pitching and Anchoring											△
Type 5	Concrete pitching and Anchoring											
Type 6	Grid Type Concrete Retainig Wall	●	△		△	△		△		△		●
Type 7	Supported Type Concrete Retaining Wall											
Type 8	Catch Ditch											
Type 9	Catch Netting											
Type 10	Gabion Catch Wall											●
Type 11	Catch Fence installed at road side							●				
Type 12	Catch Fence installed at top pf slope											
Type 13	Concrete Catch Wall											
Type 14	Sub-surface Drainage for Landslide											
Type 15	Gabion Dam for Debris/earth Flow											
Type 16	Concrete Dam for Debris/earth Flow											
Type 17	Debris/earth Flow Shed											
4) Selection Factor		C	T	-	T	T	-	T	-	T	-	C/T
Remarks (Appendix No.)		4-(1)	4-(2)		4-(3)	4-(4)	Peculiar location	4-(5)		4-(6)		4-(7)

Notes: 1) Stations indicated accoding to nominal of milestones on the existing road defined in "2.3.2(1)".
2) A:Cut slope failure, C:Rock fall, D:Landslide, E:Debris/earthflow, II:Unstable when it rains, III:Unstable
3) ●:Optimum Countermeasure(S). △:Possible Countermeasure(S)
4) C:Cost, T:Topographical Condition
5) Due to steep Topographical condition, the stable slope gradient would not be applied to this location. Thus, Concrete Spraying should be applied to prevent the rock fall instead of Catch Netting + Catch Fence.

Table 4. 5-3 (2) Comparison of Countermeasure (s)

Spot												
	1) Station on Existing Alignment	H +2.3	I +3.0	I +3.3	J +0.6	J +4.9	J +5.9	K +0.8	K +2.2	L +3.0	L +6.1	M +1.2
	Station on New Alignment	No.35+560	No.38+740	No.39+30	No.40+300	No.44+400	No.45+230	No.47+520	No.49+210	No.52+200	No.55+500	No.57+750
2) Disaster type and the Grade		C(II)	A(II)	A(III)	A/E(III)	A(II)	A(II)	A(II)	E(II)	A(II)	A(II)	A(II)
Type 1	Concrete Spraying						●	●				●
Type 2	Stone Masonry Retaining Wall									●		
Type 3	Concrete Crib with concrete spraying and Anchoring		●	△		●					●	
Type 4	Concrete Crib with stone pitching and Anchoring											
Type 5	Concrete pitching and Anchoring									△		
Type 6	Grid Type Concrete Retainig Wall		△	●		△				△	△	
Type 7	Supported Type Concrete Retaining Wall											
Type 8	Catch Ditch											
Type 9	Catch Netting	●										
Type 10	Gabion Catch Wall											
Type 11	Catch Fence installed at road side	●										
Type 12	Catch Fence installed at top pf slope											
Type 13	Concrete Catch Wall											
Type 14	Sub-surface Drainage for Landslide											
Type 15	Gabion Dam for Debris/earth Flow								●			
Type 16	Concrete Dam for Debris/earth Flow											
Type 17	Debris/earth Flow Shed				● 3)							
4) Selection Factor		-	T	C	-	T	-	-	-	C/T	T	-
Remarks (Appendix No.)			4-(8)	4-(9)		4-(10)				4-(11)	4-(12)	

Notes: 1) Stations indicated according to nominal of milestones on the existing road defined in "2.3.2(1)".
2) A:Cut slope failure. C:Rock fall. D:Landslide. E:Debris/earthflow. II:Unstable when it rains. III:Unstable
3) ●:Optimum Countermeasure(S). △:Possible Countermeasure(S)
4) C:Cost. T:Topographical Condition

Table 4.5-3 (3) Comparison of Countermeasure(s)

Spot														
		[1] Station on Existing Alignment	M +1.5	N +1.4	N +2.0	N +3.0	N +3.4	N +4.7	O +1.2	O +3.1	P +0.9	P +1.7	P +3.6	P +3.7
		Station on New Alignment	No.58+200	No.66+ 0	No.66+500	No.68+440	No.68+820	No.70+100	No.71+800	No.73+890	No.75+570	No.75+320	No.77+800	No.78+100
[2] Disaster type and the Grade			A(III)	A(II)	A(II)	A(III)	E(II)	A(II)	E(II)	E(II)	A(III)	A(II)	A(III)	A(II)
	Type 1	Concrete Spraying		●										
	Type 2	Stone Masonry Retaining Wall						△						●
	Type 3	Concrete Crib with concrete spraying and Anchoring			●						●			
	Type 4	Concrete Crib with stone pitching and Anchoring												
	Type 5	Concrete pitching and Anchoring	●			△		●					●	△
	Type 6	Grid Type Concrete Retainig Wall	△		△	●		△			△		△	△
	Type 7	Supported Type Concrete Retaining Wall	△			△							△	
	Type 8	Catch Ditch												
	Type 9	Catch Netting										●		
	Type 10	Gabion Catch Wall										●		
	Type 11	Catch Fence installed at road side												
	Type 12	Catch Fence installed at top pf slope												
	Type 13	Concrete Catch Wall												
	Type 14	Sub-surface Drainage for Landslide												
	Type 15	Gabion Dam for Debris/earth Flow					●		● [3]	●				
	Type 16	Concrete Dam for Debris/earth Flow												
	Type 17	Debris/earth Flow Shed												
[4] Selection Factor			T	-	T	C/T	-	T	-	-	T	-	T	C/T
Remarks (Appendix No.)			4-(13)		4-(14)	4-(15)		4-(16)			4-(17)		4-(18)	4-(19)

Notes: 1) Stations indicated accoding to nominal of milestones on the existing road defined in "2.3.2(1)".
2) A:Cut slope failure, C:Rock fall, D:Landslide, E:Debris/earthflow, II:Unstable when it rains, III:Unstable
3) ●:Optimum Countermeasure(S), △:Possible Countermeasure(S)
4) C:Cost, T:Topographical Condition

Table 4.5-3 (4) Comparison of Countermeasure (s)

Spot		[1] Station on Existing Alignment	P +4.0	P +4.9	Q +0.6	R +0.3	R +1.8	T +2.5	V +0.2	V +2.8	W +0.0	W +0.3	W +1.2	W +3.0
		Station on New Alignment	No.78+600	No.79+500	No.80+350	No.82+400	No.84+350	No.92+900	No.101+475	No.103+190	No.104+ 20	No.104+670	No.105+840	No.107+500
[2] Disaster type and the Grade			A(III)	A(II)	A(III)	A(II)	A(II)	C(II)	A(II)	E(II)	D(III)	D(III)	D(III)	D(III)
	Type 1	Concrete Spraying							●					
	Type 2	Stone Masonry Retaining Wall		●										
	Type 3	Concrete Crib with concrete spraying and Anchoring	●			△	●							
	Type 4	Concrete Crib with stone pitching and Anchoring												
	Type 5	Concrete pitching and Anchoring		△	△									
	Type 6	Grid Type Concrete Retainig Wall	△	△	●	●	△							
	Type 7	Supported Type Concrete Retaining Wall			△									
	Type 8	Catch Ditch												
	Type 9	Catch Netting						●						
	Type 10	Gabion Catch Wall												
	Type 11	Catch Fence installed at road side						●						
	Type 12	Catch Fence installed at top pf slope												
	Type 13	Concrete Catch Wall												
	Type 14	Sub-surface Drainage for Landslide									● [3]	●	●	●
	Type 15	Gabion Dam for Debris/earth Flow								●				
	Type 16	Concrete Dam for Debris/earth Flow												
	Type 17	Debris/earth Flow Shed												
[4] Selection Factor			T	C/T	C/T	C	T	-	-	-	-	-	-	-
Remarks (Appendix No.)			4-(20)	4-(21)	4-(22)	4-(23)	4-(24)							

Notes: 1) Stations indicated accoding to nominal of milestones on the existing road defined in "2.3.2(1)".
2) A:Cut slope failure. C:Rock fall. D:Landslide. E:Debris/earthflow. II:Unstable when it rains. III:Unstable
3) ●:Optimum Countermeasure(S). △:Possible Countermeasure(S)
4) C:Cost. T:Topographical Condition

Table 4.5-4 Adopted Countermeasures

Adopted Countermeasure	Number of Countermeasures		
	Grade II location	Grade III location	Total
Type 1 Concrete Spraying	8	1*	9
Type 2 Stone Masonry Retaining Wall	3	-	3
Type 3 Concrete Crib with Concrete Spraying and Anchoring	7	6	13
Type 5 Concrete Pitching and Anchoring	1	2	3
Type 6 Grid Type Concrete Retaining Wall	3	3	6
Type 9 Catch Netting	3	0	3
Type 10 Gabion Catch Wall	2	0	2
Type 11 Catch Fence installed at road side	2	1	3
Type 14 Sub-surface Drainage for Landslide	0	4	4
Type 15 Gabion Dam for Debris/earth Flow	5	-	5
Type 17 Debris/earth Flow Shed	0	1	1
Total number of Countermeasures	34	18	52
Number of Locations having two countermeasures	5	1	6
Total number of Spots requiring countermeasures	29	17	46

* It was adopted to a peculiar location.(Refer to Table 4.5-3(1))

5. PRELIMINARY DESIGN

5. PRELIMINARY DESIGN

The surveys conducted for the purpose of preliminary design can generally be classified in to four groups: preliminary road design, preliminary structural design, construction planning and maintenance planning. The preliminary design includes estimation of the construction quantities.

The preliminary design was done using 1:5,000 scale topographical maps based on topographic surveys conducted by the survey group of the JICA Study Team in September and October, 1989.

Route proposed in the preliminary design was based on the optimum route selected from the comparison study of the possible routes described in Section 4.
Major tasks undertaken in the preliminary design are as follows.

5.1 Preliminary Road Design

The preliminary road design mainly consists of horizontal and vertical alignment, cross section, pavement and drainage design.

These were carried out, on the basis of the results gained from precise field investigations, topographical and geological surveys, and surveys on existing structures. As for the fundamental design conditions, the design criteria mentioned in Section 4 have been employed.

5.1.1 Horizontal and Vertical Alignment Design

For the horizontal and vertical alignment design, the entire length of the planned route (108.63 Km) was divided into six sections according to topographical and geological characteristics:

a) Santa Barbara,(No.0+0) - Point(F),(No.25+300)--Section 1
b) Point(F),(No.25+300) - Point(K),(No.46+760)----Section 2
c) Point(K),(No.46+760) - Caranavi,(No.60+0)------Section 3
d) Caranavi,(No.60+0) - Point(Q),(No.79+550)------Section 4
e) Point(Q),(No.79+550) - Point(V),(No.101+300)---Section 5

f) Point(V),(No.101+300) - B/Vista,(No.108+630)---Section 6

(1) Santa Barbara - Point (F) Section

Major design tasks undertaken for this section are as follows:

a) Since the topography on the valley side of the existing road was so steep as to make widening of the valley side by means of embankment filling impossible, the road widening was extended along the mountain side.

b) Since the dimensions of probable disasters occurring at Point (A) is so great as mentioned in Section 3 and 4, it makes the used of simple protection works difficult. Hence a new route was selected to bypass the existing route.

c) Although the gradient of the existing road in the No.0+200 - No.6+300 exceeded 7% at some subsections, the gradient was adjusted to 7% using cut and fill of the existing road surface.

d) Since the gradient of existing road subsections other than those described in c) above was 7% or less, the gradient of the planned road was set according to the gradient of the existing roads

(2) Point (F) - Point (K) Section

a) Compared with the above-mentioned road section, the topography in this section is characterized by a smaller difference in elevation. However, construction of a new road is still difficult since less land is available for road construction due to the proximity of the existing road and the Coroico River. Consequently, the route for this section, like the previous section, was planned by considering possible widening of the mountain side.

b) As considered in section 4.3.3, two tunnels were planned in the No.35+500 - No.36+500 subsection. The cross sections of the tunnels and other details are described in 5.3.1.

c) Since the existing road in No.39+800 - No.40+100 subsection meanders with a number of small curvatures, sight distance is not good. The road pass through the village of Chojña, causing problems such as community segregation and frequent traffic accidents. Since any horizontal alignment in accordance with the existing route would result in an undesirable location for the Chojña Bridge, as well as an undesirable alignment for the road, the new road was planned so that it ran near one end of the village, bypassing the ridge No.40 with a radius of curvature of R=50.

d) A swamp in the No.42 - No.44+400 subsection occupies relatively flat terrain. Since this project produces a large amount of surplus soil, a route was proposed assuming the use of spoil banks as fill for the road. (locations of fill: No.42+200, No.42+700, No.43+700, No.44+300)

e) Since the gentle gradient of the existing road does not pose any problem with respect to the geometric design standard, the vertical alignment was planned in accordance with the existing road.

(3) Point (K) - Caranavi Section

a) Although the topography in the No.54+50 - No.57+800 subsection is steep, the remaining area is relatively flat. Since a considerable amount of surplus soil is expected from the project, as mentioned above, embankment structure will be utilized wherever possible in the planning horizontal alignment.

b) A new route was adopted for the No.49+200 - No.51+700 subsection according to the results of the study explained in section 4.3.4.

c) The route for the No.58 - No.60+100 subsection was planned by aligning the center lines of the existing and new roads since the improvement of the road and the Yara Bridge was already complete.

d) Since there were no major problems with the gradients of the existing roads, the vertical alignment of all routes other than those to be newly laid out were planned base on those of the existing ones.

e) The vertical alignment of new route section was designed so as to maximize the amount of fill used.

(4) Caranavi - Point (Q) Section

a) The existing road in the No.60+100 - No61+200 subsection goes generally toward the town of Caranavi. Since there is a difference in elevation of 30 m between the planned and existing roads at the connection point with the Yara Bridge, it is impossible to connect the existing road to the Yara Bridge directly. The horizontal alignment in this section was planned for a gradient of 6.5%, and the route has been determined so that it suits the topography.

b) The newly selected route as explained in section 4.3.5 was adopted for the No.73+200 - 600 subsection. Subsections with a steep topography except the above two, have been planned so that the road is widened on the mountain side.

c) Where the topography was not steep and utilize fill was possible, the widening of the road was planned on the valley side. These subsections include No.64+630, No.66+200, No.69+600, No.73+900, No.77+600, and No.78+850.

d) Although there were some problems concerning the gradient of the road in two of the subsections, one was solved by creating a new route to replace the existing one. The gradient in the other subsection is great, but the specific limits of i=8% and L=400 m can be met by utilizing cut and fill along the existing road. Since the road sections directly connecting with these subsections are downslope, vehicles are able to quickly regain speed.

e) Since the gradient of the remaining subsections does not exceed 7%, these subsections was planned based on

the gradient of the existing road.

(5) Point (Q) - Point (V)

a) As shown in Fig 5.1-1, the topography in this section is such that areas in the No.79.500 - No.89 subsection are not steep but those in the No.89 - No.101+350 subsections and very steep. In subsections where No.84+300 - No.85+500 and No.89 - No.101+350 the vertical alignment was undesirable, the alternative route, which had been selected as a result of comparison explained in 4.3.6, was adopted in planning the horizontal alignment. The vertical alignment has been planned so that surplus soil is minimized by using as much embankment volume as possible as filling.

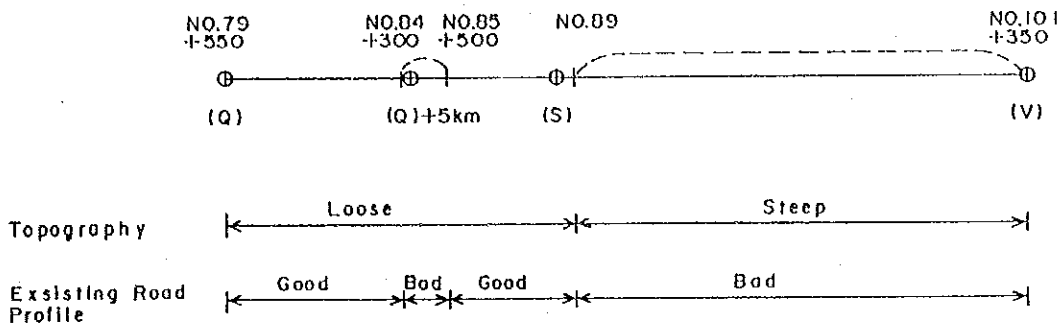


Fig. 5.1-1 Topography and Profile Condition of Existing Road

b) Since the No.79+550 - No.84+300 and No.85+500 - No.89 subsections are not steep, the widening of the road has been planned using embankment structure on the valley side wherever possible. For example, fill with a depth of 15 to 20 m has been planned for subsections No.80+100, No.80+950 - No.81+400, No.85+650, No.86+300, No.87, and No.87+950. Since the gradient of the remaining subsections does not exceed 7%, these subsections was planned based on the gradient of the existing road.

(6) Point (V) - Bella Vista

a) The No.101+350 - No.103 subsection is steep. If this subsection is to be widened on the valley side, the slope needs to be protected by means of embankment structure or other protective structures. Therefore, a horizontal alignment was planned based on the alignment of the existing road, so that the road is widened on the mountain side and so that cutting can be minimized.

b) Since the subsection between No.103 and the end of the road is not steep, plans was made so that the road can be widened on the valley side by utilizing embankment wherever possible.

c) The vertical alignment of the overall section was planned in accordance with that of the existing road since the vertical alignment of the existing route does not pose any major problems.

5.1.2 Cross Section Design

(1) Road Width

The width of the road was planned in accordance with the specified in 4.1.

(2) Slope gradient

The slope gradient of cut and embankment is decided according to the types of filling material used and the geological characteristics of grounds to be cut.
Gradient of slope shown in Table 5.1-1 was used in the design.

Table 5.1-1 Standard Gradient of Slope

Type of Slope Method		Gradient of Slope(H/V)
Fill (high-quality material)		1.5(H)/1(V)
Cut	Hard rock	1/4
	Soft rock	1/2
	Weathered rock	1/1.5
	Earth	1/1

Much of the surplus soil produced during cutting the slope is suitable for being used as a high quality material of embankment.
The slope gradient of the fill, therefore, was set at 1.5/1.

(3) Berms

a) Berms are provided to stabilize slopes, to reduce the power of surface water flow over the slopes, and the minimize the erosion.

b) Basically, built-up berms was provided where the height of fill is 8 m or greater. These berms are provided at every 5 m elevation point from the top of embankment slope and are 1.5 m wide.

c) Generally, along the slopes of cut areas or weathered rock there are 1.5 m wide berms at every 5 m elevation point. For a slope made of soft rock, a 1.0 m wide berm is provided every 10 m in height.

d) No berm is provided on a slope composed of hard rock since it does not have an effect on the stability and erosion of the slope.

(4) Cross Section Design

The cross section of the existing ground conditions was prepared with the paper location. The cross section points were determined at about 100m intervals required for the calculation of soil volumes. Under this cross section, the

cross section design was planned based on the above criteria (1) to (3).

1) Retaining Walls for Embankment

a) Since the construction cost of roads is greatly affected by the size and number of retaining walls and similar structures required, the cross section of the road was planned so that the number of retaining walls can be minimized.

b) The use of retaining walls was included in the cross section plan only when earth works cannot furnish acceptable solutions in special topographical areas, or when the use of structures other than earth works can bring about more economical and safer solutions.

The type and height of walls used are shown in Table 5.1-2.

Table 5.1-2 Type and Height Range of Retaining Wall

Type		height Range (m)
Gravity		0 ~ 3
Masonry (fill)		0 ~ 5
Grid	Single	5 ~ 7.5
(H/V = 1/2.5)	Double	7.5 ~ 15
	Triple	15 ~ 22

2) Cut Slopes

In the cases where the use of the standard gradient of slope is expected to result in extremely large volumes of excavation or long cut slopes, the following special slope protection works was employed for the purpose of economy and safety:

a) For earth slopes:
Masonry retaining walls on cut sections
Gradient of slope n = 1/2
Height of wall H 7.0 m

b) For weathered rock slopes:

Shotcrete

Gradient of slope	$n = 1/2$
Height of slope	$H \leqq 10$ m

c) For soft rock slopes:

Shotcrete

Gradient of slope	$n = 1/4$
Height of slope	$H \leqq \infty$

5.1.3 Pavement Design

The pavement layer thicknesses have been estimated according to the "AASHTO Guide for Design of Pavement Structures, 1986". The method is summarized in Fig.5.1-2. Each design consideration is described in the clause numbered on the right hand side of each box.

(1) Analysis Period

As the improvement work is considered to be finished in 2000 according to the analysis described in Chapter 7, Overall Evaluation and Implementation Program, the analysis period in the pavement design is assumed to be from 2001 to 2010.

(2) Design Traffic

The accumulated future traffic volumes classified by specific vehicle type between 2001 and 2010 are taken as the required Design Traffic for the pavement design, and are summarized in Table 5.1-3 from the results of "Traffic Study" described in Chapter B of the Report, Volume II.

Table 5.1-3 Accumulated Traffic Volumes by Vehicle Type (2001 - 2010)

Section	Vehicle Type				(Vehicles)	
	Passenger Car	Bus	Light Truck	Medium Truck	Heavy Truck	Total
Sta. Barbara - Caranavi	272,175	154,889	1,116,271	172,920	2,644,552	4,360,807
Caranavi - Bella Vista	58,870	18,031	210,107	31,735	1,425,347	1,744,090

(3) Axle Load for Each Vehicle Type

In this design method, the axle load distribution for each vehicle type must be established in order to derive the necessary load equivalence factors for the conversion of mixed traffic, to its 18-kip equivalent single axle loading (ESAL). However, since this information is not available

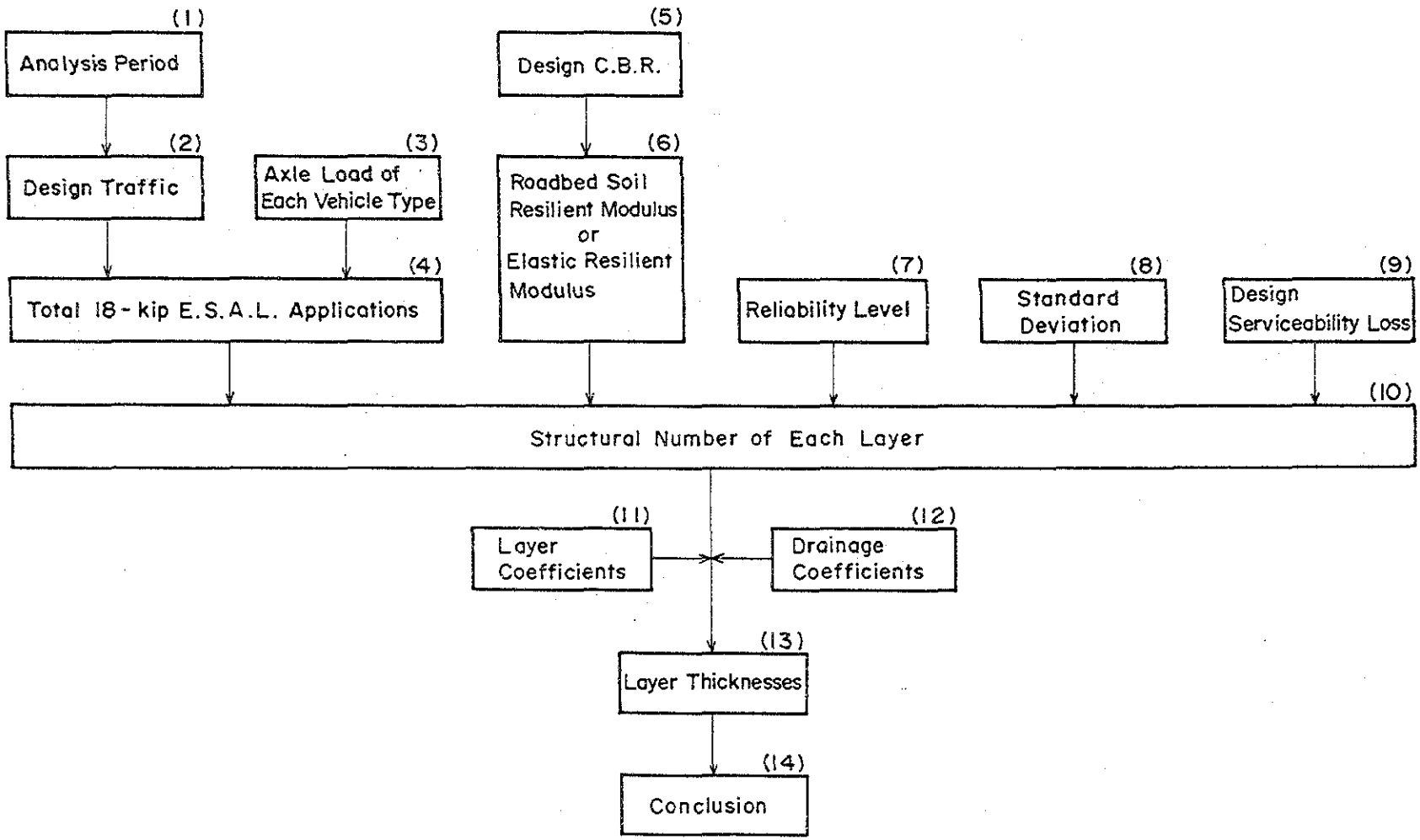
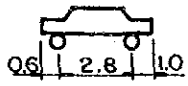
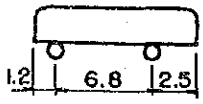
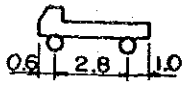
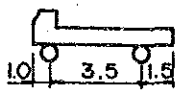
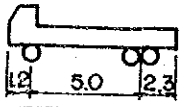


Fig. 5.1-2 Pavement Design Method

in Bolivia, representative values obtained from similar trunk roads in Japan, were utilized as being the axle load distributions. (Table 5.1-4)

Table 5.1-4 Axle Load Distribution for Each Vehicle Type

Vehicle Type	Average Vehicle Weight	Vehicle Model	Ratio of Axle Load		Axle Load Distribution	
			Front Wheel	Rear Wheel	Front Wheel	Rear Wheel
Passenger Car	1.30 (t)		0.501W + 0.03	0.498W - 0.03	S (t) 0.6813	S (t) 0.6174
Bus	13.80		0.376W - 0.464	0.624W + 0.464	S 4.7248	S 9.0752
Light Truck	3.60		0.231W + 0.76	0.769W - 0.76	S 1.5916	S 2.0084
Medium Truck	6.20		0.182W + 1.38	0.818W - 1.38	S 2.5084	S 3.6916
Heavy Truck	17.00		0.109W + 3.22	0.891W - 3.22	S 5.073	T 11.927

Note : W = Vehicle Weight, S = Single Axle, T = Tandem Axle

(4) Total 18-kip ESAL Applications

The cumulative total of 18-kip equivalent single axle loads for the analysis period were estimated on the basis of the Design Traffic and load equivalency factors (ESAL factors) as follows.

Table 5.1-5 Estimation of Total 18-kip ESAL Applications

Section	Vehicle Type	Design Traffic(A)	ESAL Factor(B)	Design ESAL(A) X (B)
Sta. Barbara - Caranavi	Passenger Cars	272,175	0.0004	109
	Buses	154,889	1.598	247,513
	Light Trucks	1,116,271	0.004	4,465
	Medium Trucks	172,920	0.044	7,608
	Heavy Trucks	2,644,552	0.553	1,462,437
Total				1,722,132
Caranavi - Bella Vista	Passenger Cars	58,870	0.0004	24
	Buses	18,031	1.598	28,813
	Light Trucks	210,107	0.004	840
	Medium Trucks	31,735	0.044	1,396
	Heavy Trucks	1,425,347	0.553	788,217
Total				819,290

Note : ESAL Factors were derived from Appendix 5-1(1).

The design ESAL must be converted into total 18-kip ESAL equivalents, classified by direction and lanes. To this end, the following equation was used.

$$W_{18} = D_D \times D_L \times W_{18}$$

where,

D_D = a directional distribution factor, expressed as a ratio, that accounts for the distribution of ESAL units according to direction.
The factor is generally 0.5 for most roadways.

D_L = a lane distribution factor, expressed as a ratio, that accounts for the distribution of traffic when two or more lanes are available in

one direction. The project road will be one lane in one direction. Thus, the factor is to be 1.0

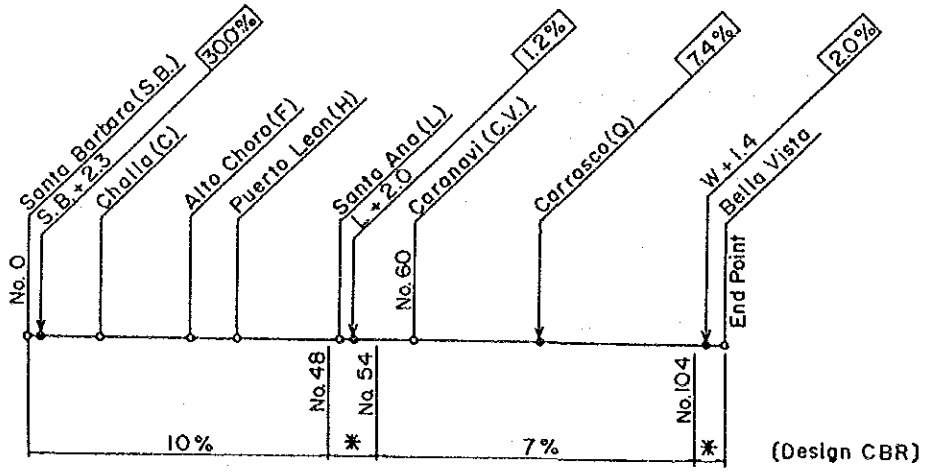
W_{18} = the cumulative two-directional 18-kip ESAL units predicted for a specific section of high way during the analysis period. (design ESAL) The results were as follows:

Section	Total 18-kip ESAL Application
No.0 (Sta. Barbara) - No.60 (Caranavi)	861,066
No.60 (Caranavi) - End Point(Bella Vista)	409,645

(5) Design CBR

For the purpose of estimating design CBR values for the road subbase soil, laboratory tests were carried out by samplings at four survey points which possess typical soil characteristics for the project site. The results are indicated in Fig.5.1-3.

Assuming that a similar CBR value is found in the vicinity of each survey point, design CBR values for similar sections were determined. These are also presented in Fig.5.1-3.



* The section requiring displacement of roadbed soil

Fig. 5.1-3 Estimation of Design CBR

Since the sections marked with an asterisk indicate the lower CBR value of 1.2 or 2.0, the displacement of the roadbed soil should be adapted for material cost saving. A CBR value after displacement, that is, the value to be adopted as the design CBR, is normally estimated by the following equation:

$$CBRm = \left(\frac{(Dd-20) \times CBRd^{1/3} + 20 \times CBRo^{1/3}}{Dd}\right)^3$$

where,

Dd = Amount of displacement (cm)
CBRm = CBR value after displacement
CBRd = CBR value of materials for displacement
CBRo = CBR value of existing roadbed soil

Assuming that the displacement (Dd) is 100 cm and the CBR value for displaced material is 10% (which is considered to be available on site), the CBR value after displacement was estimated as follows:

Section	No.48 - No.54	No.104 - End point
CBR of existing roadbed soil	1.2 %	2.0 %
CBR of displaced materials	10.0 %	10.0 %
displacement	100.0 cm	100.0 cm
CBR after displacement	7.0 %	7.0 %

To the contrary, one section was found to indicate a remarkably higher value, (30%). It is considered that this value is excessively high to be used as a CBR value for the roadbed soil. It would be appropriate to adopt a value of 10% in view of design safety.

(6) Roadbed Soil Resilience Modulus / Elastic Resilience Modulus

Roadbed Soil Resilience Modulus

The equation for converting CBR value to the roadbed soil resilience modulus(M_R) are as follows:

M_R (psi) = 1,500 X CBR(%)

Thus, estimated roadbed resilience modulus was:

CBR = 10 : M_R = 1,500 X 10 = 15,000 psi

CBR = 7 : M_R = 1,500 X 7 = 10,500 psi

Elastic Resilience Modulus for Base Layers

For aggregate base layers, the elastic resilience modulus (E_{sb}) is a function of the stress state within the layer and is normally given by the relationship:

$$E_{sb} = K_1 \times \theta_{sb}^{K_2}$$

where,

θ = stress state,

K_1, K_2 = are regression constants which are a function of the material characteristics.

Values for the stress state within the base vary with the sub-grade modulus and thickness of the surface layer. Assuming that the surface layer is 4 to 6 inches, the value would be 15.0.

Values of K_1 and K_2 for the base materials vary with the material quality and moisture conditions as follows:

Moisture Condition	K_1*	K_2*
Dry	6,000 - 10,000	0.5 - 0.7
Damp	4,000 - 6,000	0.5 - 0.7
Wet	2,000 - 4,000	0.5 - 0.7

* Range in K_1 and K_2 is a function of material quality.

Considering the availability of the material on site, the following values were adopted :

$$K_1 = 8{,}000 \quad , \quad K_2 = 0.6$$

Thus, the estimated elastic resilience modulus was :

$$E_{sb} = 8{,}000 \times 15^{0,6} = 40{,}620 \text{psi}$$

Elastic Resilience Modulus for the Subbase Layers

For granular subbase layers, the elastic resilience modulus (E_{SB}) is affected by the stress conditions in a fashion similar to that for the base layer and is normally given by the following relationship:

$$E_{sb} = K_1 \times \theta_{Sb}^{k_2}$$

Values for the stress conditions within the sub-base vary with the thickness of the surface layer. Assuming that the surface layer thickness is greater than 4 inches, the value would be 5.0.

Values of K1 and k2 for the sub-base materials vary with the material quality and moisture conditions as follows:

Moisture Condition	K_1*	K_2*
Dry	6,000 - 8,000	0.4 - 0.6
Damp	4,000 - 6,000	0.4 - 0.6
Wet	1,500 - 4,000	0.4 - 0.6

* Range in K_1 and K_2 is a function of material quality.

Considering the availability of the material on site, the following values were adopted :

$$K_1 = 8{,}000 \quad , K_2 = 0.6$$

Thus, the estimated elastic resilience modulus (E_{sb}) was :

$$E_{sb} = 8{,}000 \times 5.0^{0.6} = \underline{21{,}012 \text{ psi}}$$

(7) Reliability Level

For this design method, the following values are recommended.

Table 5.1-6 Suggested levels of reliability for various functional classifications.

Functional Classification	Recommended Level of Reliability	
	Urban	Rural
Interstate and other freeways	85 - 99.9	80 - 99.9
Principal Arterial roads	80 - 99	75 - 95
Collectors	80 - 95	75 - 95
Local roads	50 - 80	50 - 80

Note : Results based on a survey in the AASHTO Pavement Design Task

The project road is one of the trunk roads in Bolivia, and is located in rural area, so it is considered to be classified in between "Interstate Highway" and "Principal Road" of the case in the United States.
After several times of discussion with SNC engineers on this matter, the reliability level was concluded to be set at 85% in the Study.

(8) Standard Deviation

A value of 0.45 is generally adopted for the overall standard deviation (S_o) for flexible pavements.

(9) Design Serviceability Loss

The following equation is normally used to define the design serviceability loss (PSI).

$$PSI = P_o - P_t$$

where,

P_o = initial serviceability
P_t = terminal serviceability index

Since the time at which a given pavement reaches its terminal serviceability depends on the traffic volume it is subjected to and the original or initial serviceability (P_o), some considerations must also be given to the selection of P_o. (It should be recognized that the P_o values observed for the AASHO Road Test were 4.2 for flexible pavements.)
Selection of the terminal serviceability index (P_t) is based on the lowest index that can be tolerated before rehabilitation, resurfacing, or reconstruction becomes necessary. An index of 2.5 or higher is suggested for the design of major highways.

Thus, the overall design serviceability loss was:

$$PSI = 4.2 - 2.5 = \underline{1.7}$$

(10) Structural Number of Each Layer

In this design method, a nomogram for estimating structural numbers(SN) is recommended. The nomogram allows each structural number to be estimated by applying the above mentioned values. These values are; total 18-kip ESAL applications, the roadbed soil resilient modulus or elastic resilient modulus, reliability level, standard deviation and design serviceability loss. The estimations for each layer are described in appendix 5-1. Furthermore, for a more accu-rate estimation, the values were also estimated using a method from the "AASHTO INTERIM GUIDE". Comparing both methods, a difference was found in the value for the roadbed in the section between No.0 (Santa Barbara) and No.48. With regard to this section, the higher conservative value was adopted for design safety. Those procedures are also described in Appendix 5-1. As a result, the following values were adopted.

Table 5.1-7 Structural Number of Each Layer

Section / Layer	No.0 (Santa Barbara) - No.48	No.48 - No. 60 (caranavi)	No.60 (Caranavi) - End (Bella Vista)
Design CBR of Sub-grade	10.0 %	7.0 %	7.0 %
Roadbed Soil (Subgrade)	2.9	3.0	2.7
Subbase Course	2.3	2.3	2.0
Base Course	1.7	1.7	1.5

Note : The estimation is described in Appendix 5-1(2).

(11) Layer Coefficients

Average values were for layer coefficients, adopted for materials used in the AASHTO Road Test. These values are as follows:

Asphalt concrete surface (a_1) = 0.44
Crushed stone base (a_2) = 0.14
Sandy gravel sub-base (a_3) = 0.11

(12) Drainage Coefficients

Table 5.1-8 shows the recommended values for drainage coefficients (mi) as a function of drainage quality and the percentage of time during a year that the pavement structure would normally be exposed to moisture levels approaching saturation.

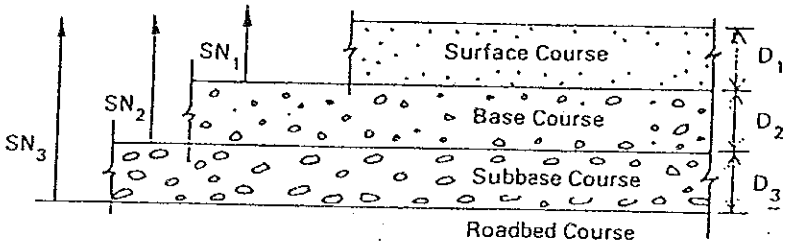
Table 5.1-8 Recommended m_i values for modifying structural layer coefficients of untreated base and sub-base materials in flexible pavements.

Drainage Quality	Percentage of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1 - 5%	5 - 25%	Greater Than 25%
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.00 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

In this design, assuming that quality of drainage is good and the percentage of time that pavement structure is exposed to moisture levels approaching saturation will be restricted to 5 to 25%, a value of 1.0 was adopted.

(13) Layer Thickness

In this design method, layer thicknesses are determined using the following equations:



$$D^*_1 \geq \frac{SN_1}{a_1}$$

$$SN^*_1 = a_1 D^*_1 \geq SN_1$$

$$D^*_2 \geq \frac{SN_2 - SN^*_1}{a_2 m_2}$$

$$SN^*_1 + SN^*_2 \geq SN_2$$

$$D^*_3 \geq \frac{SN_3 - (SN^*_1 + SN^*_2)}{a_3 m_3}$$

1) a, D, m and SN are defined in the text and are the minimum required values.
2) An asterisk with D or SN indicates that it represents the value actually used, which must be equal to or greater than the required value.

The determined thickness of each layer on each section were as follows:

Table 5.1-9 Determined Layer Thicknesses

Section / Layer	No.0(Santa Barbara) - No.48	No.48 - No.60(Caranavi)	No.60(Caranavi) - End Point(Bella Vista)
Surface	4 inches	4 inches	4 inches
Base	6	6	6
Subbase	6	6	6

Note: The determinations are presented in Appendix 5-1(3).

(14) Conclusion

With regard to surface course, an asphalt macadam method would be considered other than the above hot-mixed asphalt concrete paving. However, since the service life of the asphalt macadam is shorter, the adoption is not recommendable to the project road which the remarkable traffic increase is estimated, from a economical viewpoint.
Thus, the hot-mixed asphalt concrete paving should be adopted to the project road.

Furthermore, an idea on the implementation schedule that the project road will be left with unpaved surface for at least a few years after completion of the earthwork operation for the Project has been paid attention to in the Study from the following reasons:

a) The road section between Cotapata and Santa Barbara (connected with the project road at Santa Barbara) is planned to be an asphalt macadam road.
Therefore, the road surface between Santa Barbara and Bella Vista might be sufficient to be unpaved, considering the consistency and continuity of road conditions between Cotapata-Santa Barbara and Santa Barbara-Bella Vista for some time.

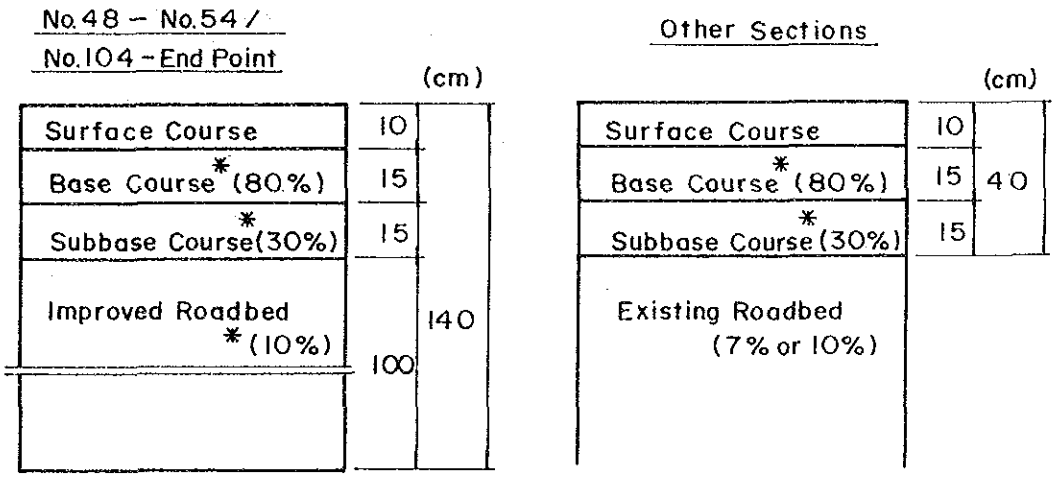
b) It is generally considered that it takes a few years for newly built slopes to be stabilized. If the project road opens with the asphalt concrete surface, the paved

surface might be damaged in a few years by construction machinery activities like bulldozers to remove debris, rocks, etc. occurred by the slope failure.

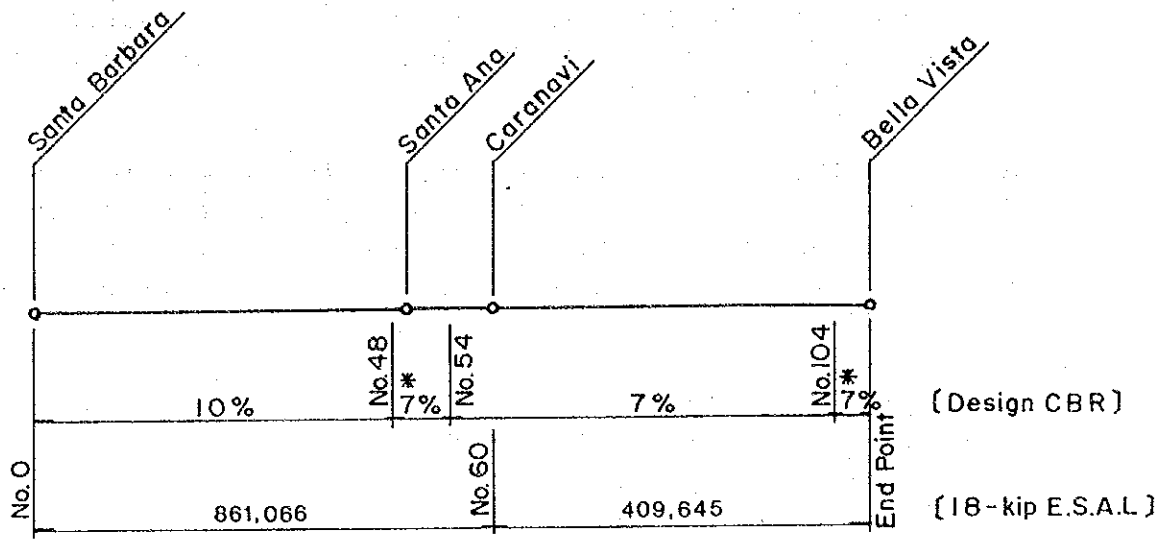
However, in this case that the Project is implemented as a stagewise construction, the procurement of the project fund would become difficult due to the twice procedures of the financial arrangement with a fund resource for the same project.

As a result of discussion with SNC for this matter, it was concluded that the asphalt concrete pavement should be constructed consecutively or together with the earthwork in view of the smooth promotion of the Project.

In conclusion, the following design pavement structures are recommended;



* Materials to be adopted
Surface - Hot-mixed Asphalt Paving
Base - Graded Crashed Stone having a CBR value of 80%
Sub-base - Sandy Gravel having a CBR value of 30%



* The value obtained by means of displacement

Fig. 5.1-4 Recommended Design Pavement Structures

5.1.4 Drainage Design

(1) Hourly Rainfall Intensity

Since no data was available on the hourly rainfall intensity in and around the study area, the cross section, high water level and gradient of four rivers were obtained by the topographic survey, whose catchment areas were measured using National Basic Map (S=1/50,000). The rainfall intensity was then estimated from these data. The results of estimation of catchment area and measurements for each river are given in Table 5.1-10.

Table 5.1-10 Catchment Areas, Cross-Sectional Areas and Gradients of Rivers

Name of River	Catchment Area (A Km^2)	Cross-Sectional Area of Flow(a m^2)	Wetted Perimeter (S m)	Gradient (%)
Patuni River	8.0	20.3	18.6	13
Challa River	9.0	17.9	14.8	14
Chorro River	135.5	182.4	51.2	2
San Pedrito	2.5	10.3	10.9	5

From the above data, the velocity and volume of flow were calculated using Manning's equation. From the relationship between the volume of flow and the catchment area, a design rainfall intensity can be estimated using the runoff equation. Results of the estimation are summarized in Table 5.1-11.

Table 5.1-11 Estimated Design Rainfall Intensity

Name of River	Flow Velocity (V m/sec)	Runnoff volume (Q m^3/sec)	Design Rainfall Intensity (I mm/h)
Patuni River	9.55	193.87	145.4
Challa River	10.62	190.10	126.7
Chorro River	8.25	1504.80	66.9
San Pedrito	5.38	55.41	133.0

Manning's Equation:

$$V = 1/n\ R^{2/3}\ I^{1/2}$$
$$Q = V \times a$$

where,
- V: flow velocity (m3/sec)
- R: hydraulic radius a/s (m)
- a: cross-sectional area of flow (m_2)
- s: wetted perimeter (m)
- I: gradient (%)
- n: coefficient of roughness (0.04)
- Q: flow volume (m^3/sec)

Runoff Equation:

$$Q = 1/(3.6 \times 10^{-6}) \times C \times I \times A \text{---------(i)}$$

where,
- Q: runoff=flow (m3/sec)
- c: runoff coefficient (0.6)
- I: design rainfall intensity (mm/hr)
- A: catchment area (m2)

From Equation (i) above, the design rainfall intensity is

$$I = (3.6 \times 10^{-6} \times Q)/(C \times A)$$

The hourly rainfall intensity was estimated by calculating back, using correction values for the time of concentration in the equations.

Name of River	L	B	Average Flow Velocity (V' Km/hr)	Time of Con-centration (T mim)	Correction Values (K)	Hourly Rainfall Intensity (I_0 mm/hr)
Patuni	5.40	1.74	36.5	8.8	2.0	72.7
Challa	4.50	1.54	37.8	7.1	2.0	63.4
Chorro	17.50	2.24	21.0	50.0	1.1	60.8
San Pedrito	2.90	0.88	35.2	4.9	2.0	66.5

Average Hourly Rainfall Intensity I_0 = 65 mm/hr

The average flow velocity in the catchment area:

$$V' = 72(H/L)^{0.6}$$

V': average flow velocity (km/hr)
L : horizontal distance along the river channel from the uppermost point to the observation point (km)
H : sectional head for L (km)

Time of concentration:

$$T = (L/V') \times 60$$

T: time of concentration (min)

Correction value:

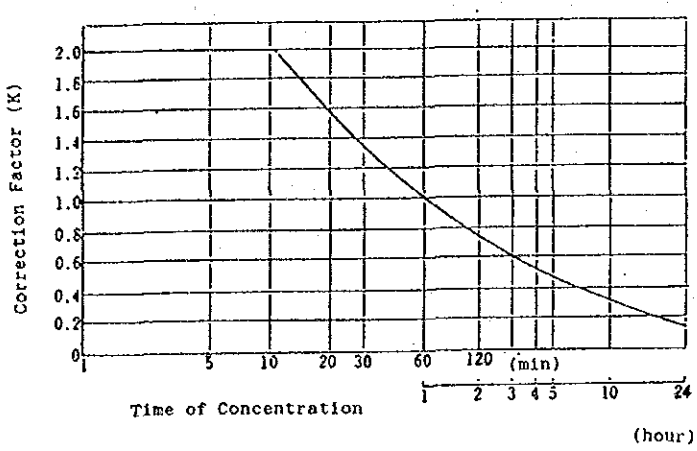


Fig. 5.1-5 Correction Factors for Different Times of Concentration

From the above, the hourly rainfall intensity of I_0=65 mm/hr was adopted for this project.

(2) Drainage Structures

1) Types of Structures and Scope of Application

Any drainage structure to be built across the road must be designed in accordance with the discharge capacity of the drainage facilities involved, and the actual runoff.

In this section, economical drainage structures for places where flow is moderate, except for the planned bridge construction sites, were selected. The following types of

structures which are commonly used in Bolivia were compared:

a) Corrugated metal pipe culvert
b) Concrete pipe culvert
c) Single box culvert
d) Double box culvert

As a result of the comparative evaluation as shown in Fig. 5.1-6, the types of structures and the scope of their application were determined as follows:

Flow (Q m3/sec)	Drainage Structure
$0 \leqq Q < 24$	Concrete pipe culvert
$24 \leqq Q < 100$	Single box culvert
$100 \leqq Q$	Double box culvert

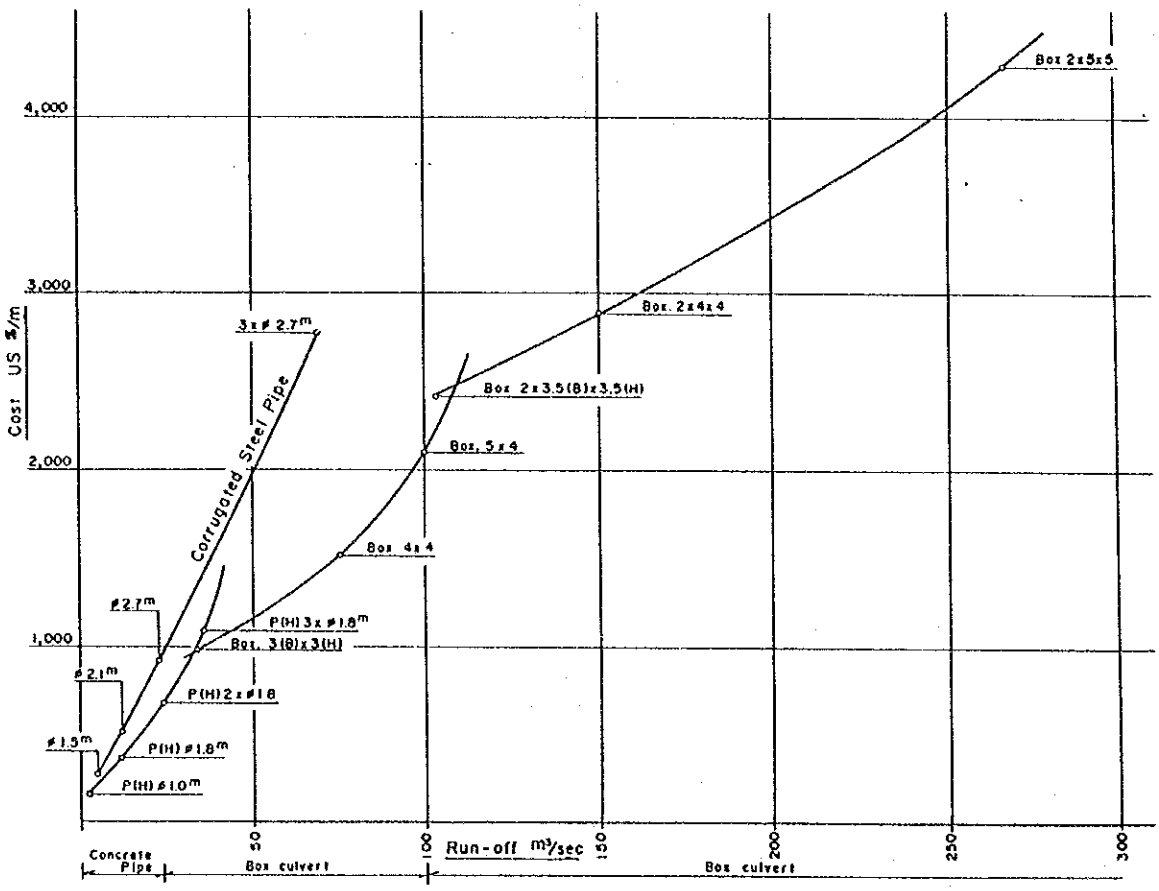


Fig. 5.1-6 Comparison of Run-off and Construction Cost

2) Catchment Area and the Size of Drainage Structures

The relationship between the catchment area and the flow was calculated so as to determine the catchment area and the size of drainage structures required. The discharge capacities for different sized structures were calculated, and the relationship between the size of structures and the catchment area was determined from a specific chart.
The relationship between the catchment area and the flow was calculated under the following conditions, using the runoff equation given in (1) above.

C: runoff coefficient (0.6)
I: design rainfall intensity ($I = K I_0 = 2 \times 65$ mm/hr =130 mm/hr)
(If the catchment area (A) $\leqq$ 6.0 km2, time of concentration is 10 minutes or less, thus K = 2.)

The results of calculation are summarized in below.

Catchment Area (A Km^2)	Flow (Q m^3/sec)
0.5	10.8
2.0	43.3
4.0	86.7
6.0	130.0

The relationship between the catchment area and the flow is illustrated in Fig. 5.1-7.
Discharge capacities calculated using Manning's equation for different sizes of structures are as follows:

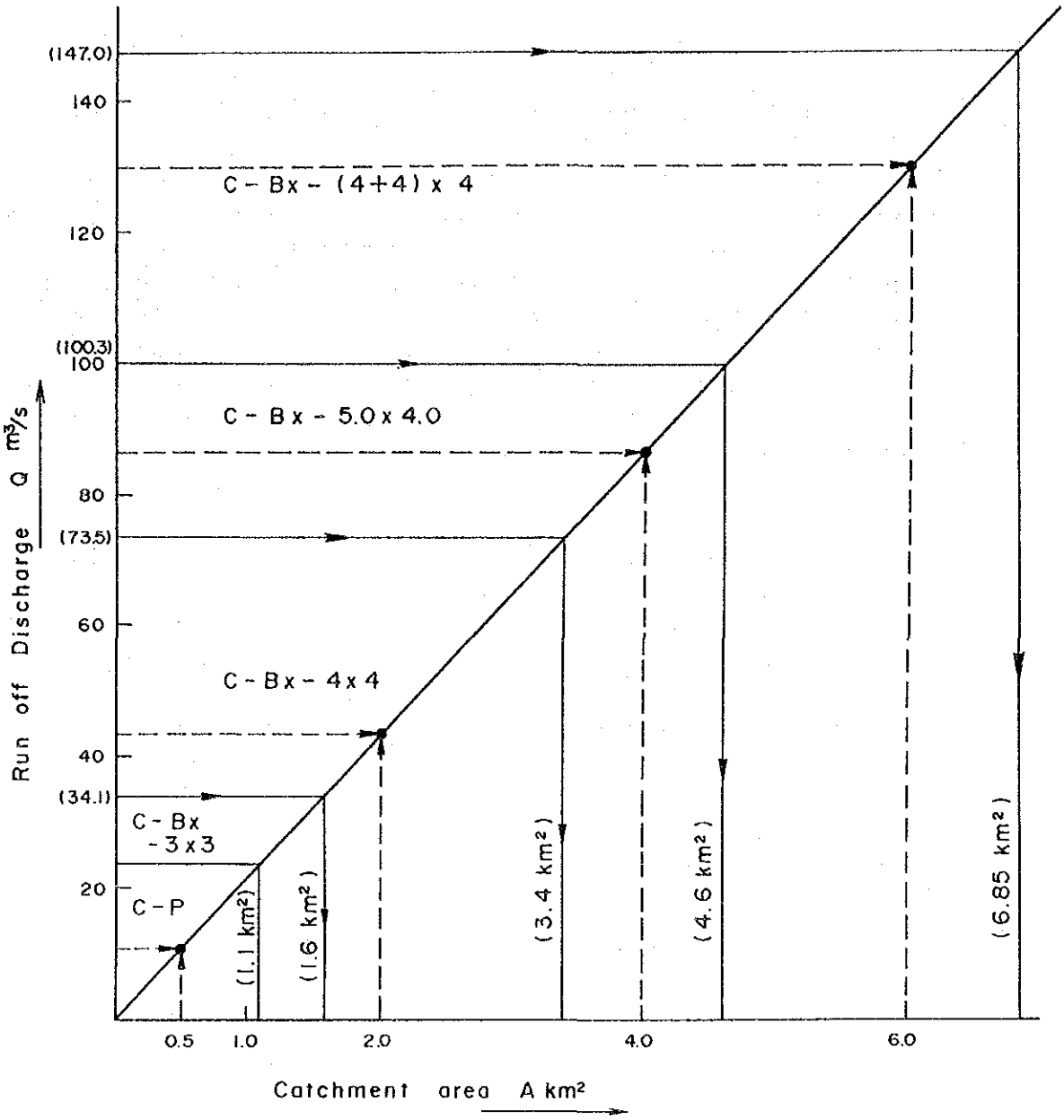


Fig. 5.1-7 Relationship between the catchment area and the flow

Size of Box	Cross-Sectional Area of Flow (a m2)	Discharge Capacity (Q m3/sec)
3.0 X 3.0	7.2	34.1
4.0 X 4.0	12.8	73.5
5.0 X 4.0	16.0	100.3
(4.0+4.0) X 4.0	25.6	147.0

Note : - An 80% depth of the cross-sectional area of flow is assumed.
- The Coefficient of roughness (n) is 0.015.
- Gradient (i) is 4%.

From Fig. 5.1-7, the relationship between the size of structures and the catchment area was determined as follows:

Structure	Catchment Area (a Km^2)
Pipe culvert	0 < a < 1.1
Box culvert 3.0 X 3.0	1.1 < a < 1.6
Box culvert 4.0 X 4.0	1.6 < a < 3.4
Box culvert 5.0 X 4.0	3.4 < a < 4.6
Box culvert (4.0 + 4.0) X 4.0	4.6 < a < 6.85

3) Drainage Structure Plan

In planning the arrangement of the box culverts, the drainage area for each swamp was measured and the following plan was formulated based on the calculations in 2) above. In remaining area, run-off water was not so much. Therefore, it was considered that installation of pipe culverts with a diameter of 100 cm every 250 m interval along the side ditches and etc.

Planned Box Culvert Locations

Station	Catchment Area (Km^2)	Cross Section (m)
No.14 + 160	1.65	4.0 X 4.0
No.20 + 540	1.30	3.0 X 3.0
No.28 + 145	1.10	3.0 X 3.0
No.30 + 180	2.50	4.0 X 4.0
No.43 + 700	1.53	3.0 X 3.0
No.49 + 210	1.28	3.0 X 3.0
No.49 + 685	1.20	3.0 X 3.0
No.64 + 660	1.18	3.0 X 3.0
No.96 + 490	1.15	3.0 X 3.0
No.97 + 225	1.18	3.0 X 3.0
No.98 + 420	1.42	3.0 X 3.0

(3) Other Drainage Structures

The failure of earth structures such as cutting and embankment is often directly caused by water. Possible causes of such failure include the scouring or erosion of slopes due to surface water flow and landslides caused by water infiltration. Drainage structures planned here are intended to secure the safety of all earth structures.

As shown in Fig. 5.1-8, there are many types on drainage of a road. Depending on the water to be drained, these types can be classified as surface drainage, underground drainage, slope drainage, or the drainage of backfilled structures. Drainage works planned for the study are as follows.

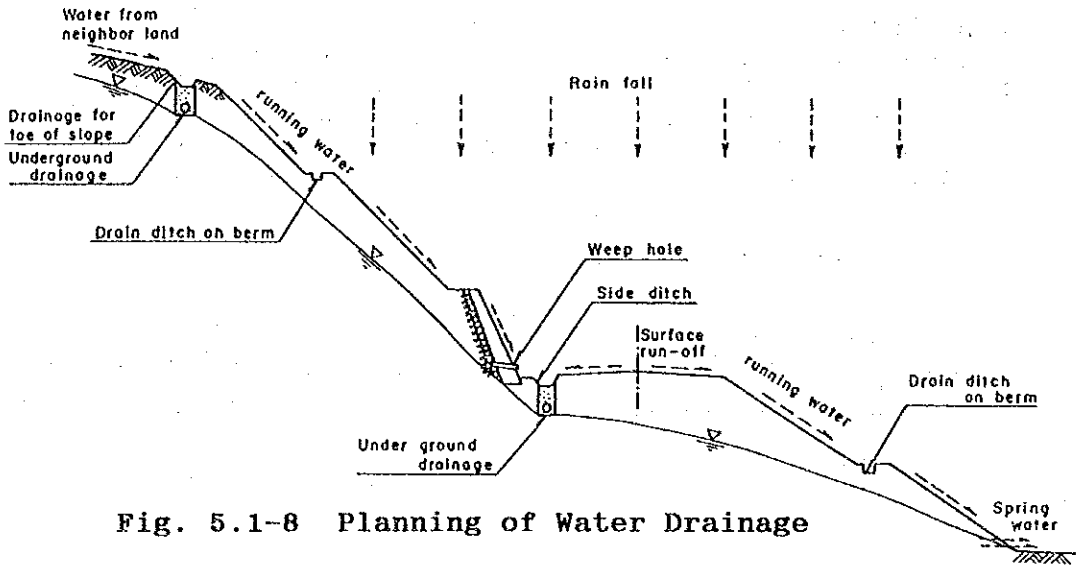


Fig. 5.1-8 Planning of Water Drainage

1) Gutters on Cut Slopes

Gutters on cut slopes collect flow from the cut slopes, surface flow from the road surface and seepage water to prevent scouring of the toes of slopes and slope failure due to erosion. Gutters on cut slopes are also provided along the entire length of cut section to prevent infiltration into the base and subgrade, thus maintaining good pavement conditions.

2) Ditches at the Top of Slopes

Ditches are made along the top of all slopes so as to prevent storm water and seepage water from flowing into the lower slopes.

3) Berm Ditches

The amount of storm water flowing down the slopes increases as the slopes become larger. For earth slopes, making ditches on the berms placed every five meters, effectively prevents the erosion of large slopes. Therefore, each earth berm has a ditch provided.

4) Storm Water Sewers

In places where there is significant seepage flow or where the groundwater level is high, it is likely that slope failure will result in the form of landslides. In addition, seepage water may permeate road bases and subgrades, causing extreme damage to the pavement.
Storm water sewers are provided in such places as to lower the groundwater level and maintain the slopes and pavement in a good condition.

5) Drainage from Backfill for Structures

The accumulation of groundwater and other water behind retaining walls could jeopardize safety, resulting in the failure of structures. Structures therefore will be provided with drain holes.

5.2 Preliminary Bridge Design

5.2.1 General

In previous section 4.3.8, the site of bridges and bridge length to be selected on the study road were examined.
The result of the above mentioned examinations are shown in table 5.2-1. The location map of bridge is shown in Fig.5.2-1.

Table 5.2-1 Selected Bridge name and length

Bridge Name	Bridge Length (m)	Curved Bridges
1) Point (A) Bridge	132.5	
2) Patuni Bridge	40	O
3) Challa Bridge	20	O
4) Cascada Bridge	18.5	
5) Alto Choro Bridge	50	O
6) Pto. Leon Bridge	75	
7) Cajones Bridge	25	
8) Chojña Bridge	22	
9) San Silverio Bridge	50	O
10) San Lorenzo Bridge	52	O
11) Espiritu Bridge	52	O
12) Carrasco Bridge	30	
13) Avaroa Bridge	25	O

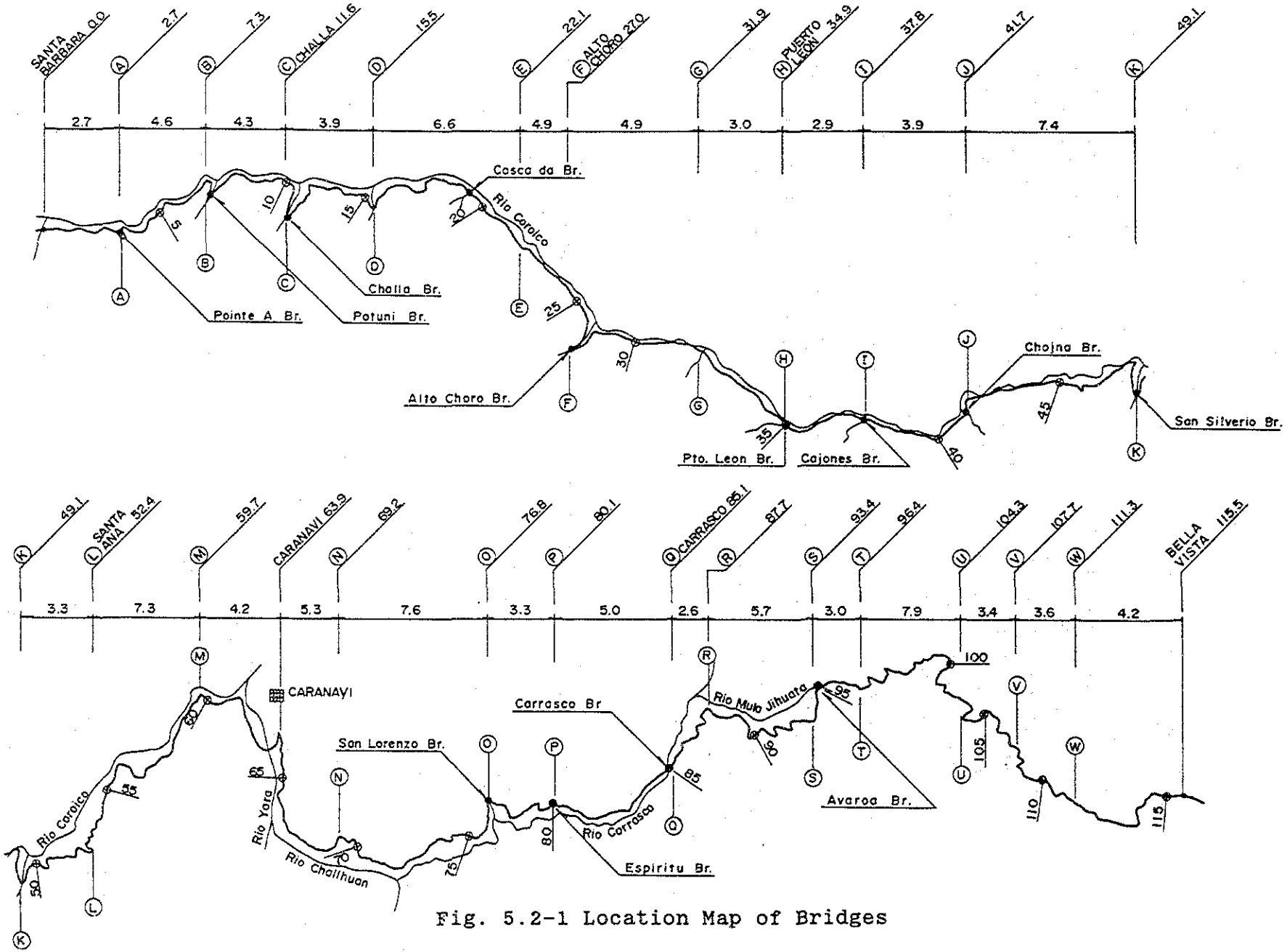


Fig. 5.2-1 Location Map of Bridges

5.2.2 Type of Superstructure

(1) Type of Bridge

As shown in table 5.2-1, seven(7) bridges among the thirteen(13) are curved. These curved bridges should be of a slab type with a high torsional rigidity,or a box girder type from the viewpoint of structural characteristics. In addition, a continuous structure is desirable from the following reasons:

a) To maintain torsional rigidity
b) To reduce twisting moments
c) To prevent negative reaction

(2) Span Arrangement

Spans arrangement for the bridge lengths determined by the method explained in section 4.3.8 (see table 5.2-1) is derived subject to the following conditions:

1) Bridges with a length of 30m or less should be single span.

2) In view of the mechanical properties, spans for curved bridges should be set so that the intersection angle ( ) for a single span is 30 degrees or less (in addition, spans should be equal wherever possible).

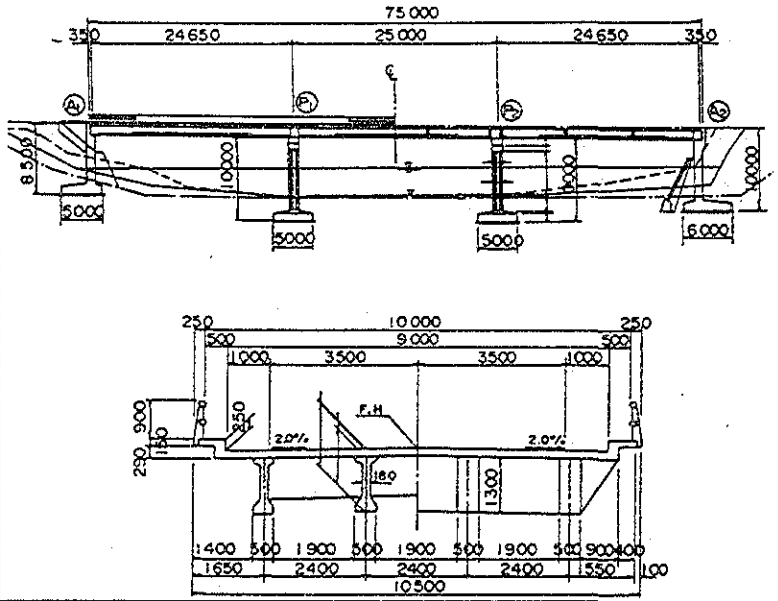
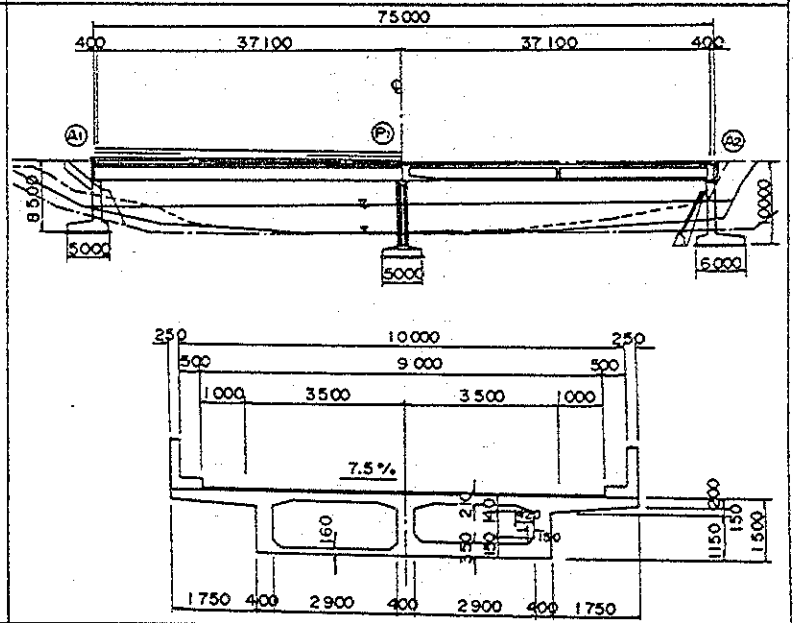
3) Bridges spans other than those described in a) or b) above (Point A and Pto. Leon) should be determined by a separate preliminary study (see Table 5.2-2, Table 5.2-7 (3)).

a) The span arrangement of Point(A) bridge is determined as 26.0m + 80.0m + 26.5m considering the type of bridge, economic aspects and geographical condition of bridge site.

b) As a result of the examination, the three span bridge was adopted for the Pto. Leon bridge by the following reasons:

- The construction cost of the three span bridge is

Table 5.2-2 Determinatiion of Spans (Pto Leon)

			Prestressed Concrete Composite Girder Bridge (3-Span)		Two-Span Continuous Box Girder Bridge	
Cross Secion						
Span (m)			24. 65+25. 0+24. 65		37. 15+37. 15	
			Superstructure	Substructure	Superstructure	Substructure
	Concrete	m³	262	498	393	425
	Form	m²	866	677	1, 977	558
	Rainforcement Bar	t	32. 1	23. 4	66. 8	19. 0
	PC-Cable	t	7. 4	–	13. 8	–
Rough Cost Estimation			554, 000$ (1. 00)		692, 000$ (1. 25)	
Evaluation			– This is the type of bridge most commonly used in Bolivia. – Lead time is short. – This type is more economical than the two-span continuous box girder bridge.		– Although a considerable number of box girder bridges have been built, most of them are long-span bridges. – Lead time is long. – This type is less economical than the composite girder bridge.	
			○		×	

cheaper than that of the two span bridge.

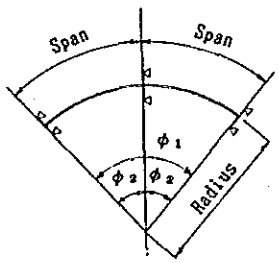
- The construction method of the three span bridge is much easier than that of the two span bridge since, a pier of three span bridge can be avoided from the center of water flow.

Table 5.2-3 shows the length of each bridge, along with the determined span arrangement.

Table 5.2-3 Length and Span of Each Bridge

Name of Bridge	Type of Bridge ※	Bridge Length(m)	Effective Width	Span (m)	Angle (Curved Bridges)	
					φ1	φ2
Point A	S	132.5	7.3～9.5	25.6 +80.0+26.1	—	—
Patuni	C	40.0	10.4	19.65+19.65	45 °02'04"	22 °31'02"
Challa	C	20.0	10.4	19.3	22 °06'58"	—
Cascada	S	18.5	9.0	17.7	—	—
Alto Choro	C	50.0	9.5	24.60+24.60	56 °29'37"	28 °11'22"
Pto. Leon	S	75.0	7.3	24.65+25.0+24.65	—	—"
Cajones	S	25.0	9.0	24.3	—	—"
Chojña	S	22.0	9.0	21.3	—	—"
San Sirverio	C	50.0	9.5	24.60+24.60	56 °29'37"	28 °11'22"
San Lorenzo	C	52.0	9.5	25.60+25.60	58 °47'08"	29 °20'08"
Espiritu	C	52.0	9.5	25.60+25.60	58 °47'08"	29 °20'08"
Carrasco	S	30.0	12.0	29.3	—	—
Avaroa	C	25.0	10.4	24.3	27 °50'45"	—

※ "S" and "C" above stand for "straight" and "curved" repectively



The bridge length is classified into four groups, such as the bridge length 18m - 22m, 25m - 30m (straight Bridge) 25m - 30m(curve bridge) and over 80m.
The preliminary design of bridge is carried out based on the above mentioned four groups.

(3) Type of Superstructure

As shown in Table 5.2-4, each type of superstructure has its own standard span based on its own structural characteristics and the economic conditions. Table 5.2-5 compares the different types of bridges in groups considered in this project. These have been determined using the criteria given in Table 5.2-4, while taking into consideration the following matters;

a) Availability of local material
b) Experience of construction method and maintenance in Bolivia
c) Economical aspects

As a result of a study, the type of bridges in each group is chosen and this is shown in Table 5.2-4 to Table 5.2-7.

Table 5.2-4 Standard Span of Bridges

Type		Span (50m, 100m, 150m)	Suitability for Carved Structure (main Structure)
Steel Bridge	Simple composite girder		○
	Continuous I-girder		○
	Simple box girder		○
	Continuous box girder		○
	Continuous truss		×
	Arch		×
Concrete Bridge	Pretensioned girder*		×
	PC hollow slab		○
	PC simple T-girder*		×
	PC composite girder		×
	Simple box girder		○
	Continuous box girder (staging method)		○
	Continuous box girder (cantilever method)		○
	RC T-girder		×

Table 5.2-5 Comparison of bridges in Different Groups

Groups	Span (m)	Name of Bridges	Type of Bridges *	Bridges Length (m)	Type of Comparison Bridges
I	ℓ=18~22	Cascada	S	18.5	PC T-Girder
		Chojna	S	22.0	PC Composite Girder
		Patuni **	S	40.0	
		Challa **	S	20.0	
II	ℓ=25~30	Pto.Leon	S	75.0	
		Cajones	S	25.0	PC Composite Girder
		Carrasco	S	30.0	PC box Girder
III	ℓ=25~30	Alto Choro	C	50.0	
		San Silverio	C	50.0	PC Follow Slab
		San Lorenzo	C	52.0	PC box Girder
		Espiritu	C	52.0	(Uniform Section)
		Avaroa	C	25.0	
IV	ℓ=80~90 (Center Span)	Point A	S	132.5	Steel Arch Bridge PC box Girder (Ununiform Section)

Note: * "S" and "C" above stand for "straight" and "curved" respectively.

** Since these bridges have small spans and shifts (1.0m or less) and thus can be adapted to curved sections using slabs, they are planned as straight bridges.

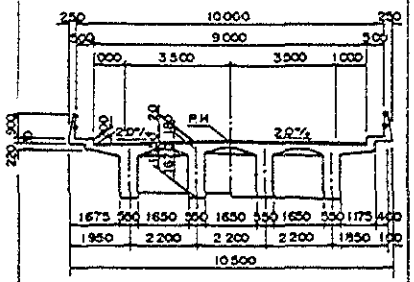
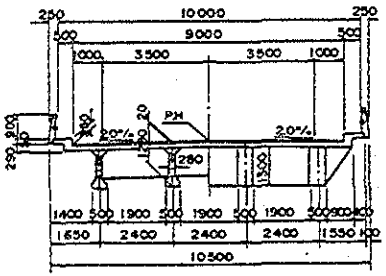
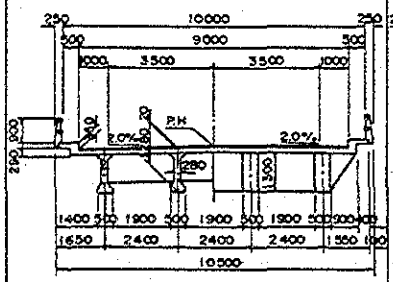
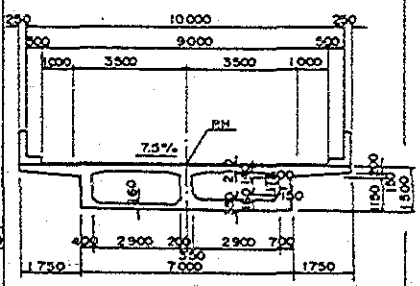
Table 5.2-6 Type of Superstructure in Different Groups

Group	Span	Type of Bridges※	Type of Superstructure
I	18m~22m	S	PC Composite Girder
II	25m~30m	S	PC Composite Girder
III	25m~30m	C	PC Box Girder(Umiform)
IV	ℓ≧80m	S	PC Box Girder(Unumiform)

※ : "S" and "C" above : stand for "Straight" and "Curved" respectively.

PC : Prestressed Concrete

Table 5.2-7(1) Composition of Different Types of Bridges

			Group I		Group II	
			RC T-Girder Bridge*	PC Composite Bridge	PC Composite Bridge	PC Box Girder Bridge (uniform section)
Cross Secion						
Span (m)			19.4		29.60	
(m)			9.0		9.00	
	Concrete	m³	128	68	122	170
	Form	m²	543	280	473	777
	Rainforcement Bar	t	18.000	12.300	19.060	28.900
	PC-Cable	t	–	1.474	3.520	3.672
Rough Cost Estimation (Superstructure)			95,000$ (1.20)	79,000$ (1.00)	153,000$ (1.00)	191,000$ (1.25)
Evaluation			×	○	○	×
			An economic comparison revealed that the PC composite bridge was most economical when the span was 20 m to 30 m. Therefore, all straight bridges with a length of 30 m or less will utilize PC composite girders.			

* RCT: reinforced concrete T-girder bridge, PC: prestressed concrete

Table 5.2-7(2) Group IV Comparison of Different Types of Bridges (Point A)

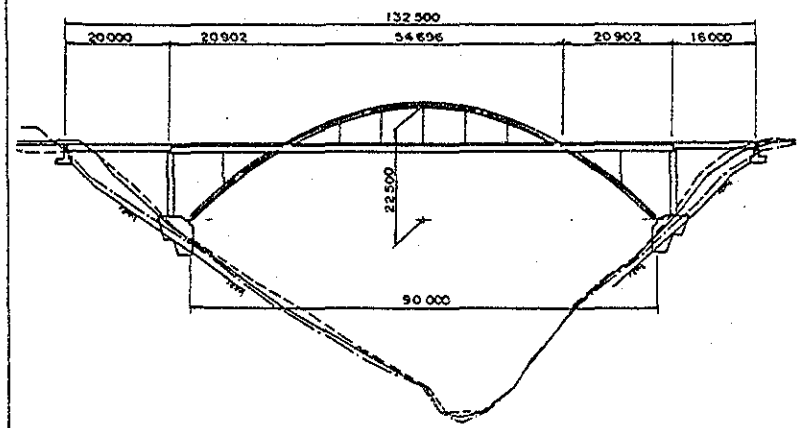
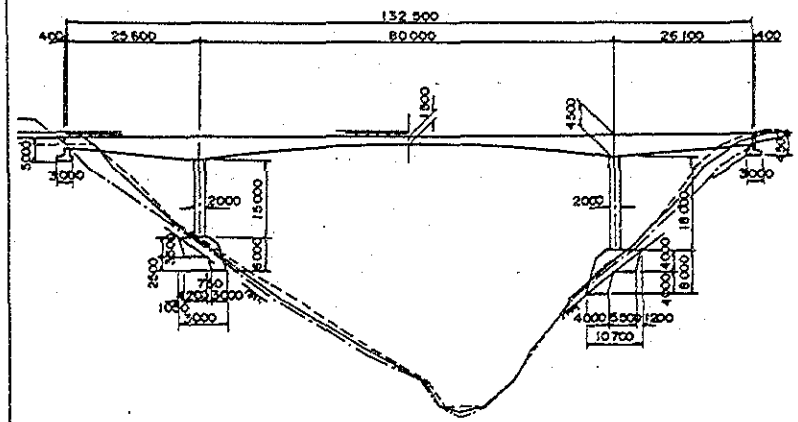
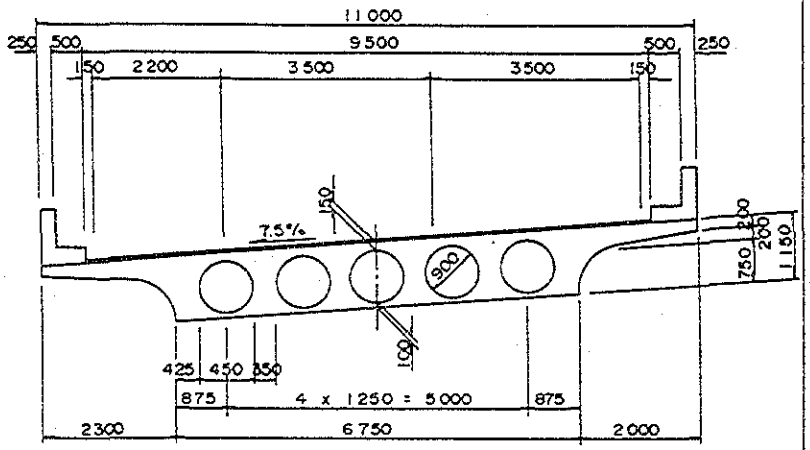
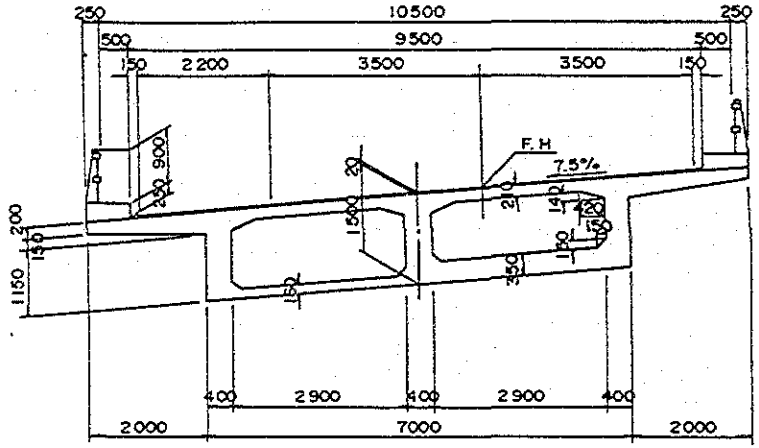
			Midheight-Deck Steel Arch Bridge		These-Span Continuous Rigid Frame Bridge	
Side View or Cross Section						
Span (m)			Arch Span 90. 0		25. 6+80. 0+26. 1	
Horizontal carvature (m)			50−∞−50		50−∞−50	
			Superstructure	Substructure	Superstructure	Substructure
	Concrete	m³	205	1, 911	1, 023	1, 488
	Stell	m³	420. 7	−	−	−
	PC-Cable	t	−	−	56. 3	−
	Rainforcement Bar	t	43. 1	23. 4	122. 8	66. 2
Rough Cost Estimation			2, 340, 000$ (1. 37)		1, 710, 000$ (1. 00)	
Evaluation			− Commonly built in V-shaped valleys and has aesthetic advangate over types of bridges. − No bridge of this type has been built in Bolivia to date. Cable construction requires a higher level of technology and skill. − Bridges materials (steel) need to be imported. which makes this type less economical.		− Commonly used with long-span concrete structures. − Aconsiderable number of bridges of this type have been built in Bolivia. This type of bridges offers fewer construction problems. − More economical than the steel arch bridge.	
			×		○	

Table 5.2-7(3) Group III Comparison of Different Types of Bridges (curved brige, ℓ=25-30m)

			PC Hollow Slab Bridge	PC Box Girder Bridge (Uniform Section)
Side View or Cross Section				
Span (m)			24.65+24.65	
Horizontal carvature (m)			50.0	
	Concrete	m³	314	288
	Form	m²	712, 716	1,350
	Rainforcement Bar	t	34.540	48.960
	PC-Cable	t	7.850	
Rough Cost Estimation (Superstructure)			361,000$ (1.01)	358,000$ (1.00)
Evaluation			– Although this is the type commonly used for curved briges, only a small number of bridges of this type were ever built in Bolivia. – Construction is slightly more complex than that of the box girder type. – Is economically similar to that of the PC box girder bridge.	– This is one of the types commonly used for curved bridges. – This type of bridge is most commonly found in Bolivia, and is easier to build than the PC hollow slab bridge. – Is economically similar to that of the PC hollow slab bridge.
			×	○

(4) Curbs, Railing and Pavement

Wall-panel railing was adopted since the road section in this project is in a mountainous region and curbside accidents are likely. At present, the wall-panel railing is in service at the Sta. Barbara Bridge and Yara Bridge.

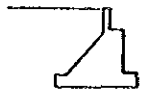
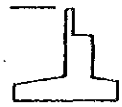
Pavement should be concrete (2.0cm thick) which has traditionally been used in Bolivia.

5.2.3 Type of Substructure

(1) Abutments

Simple structures were employed as the abutments, and the type of abutment has been determined, according to height, shown in Table 5.2-8.

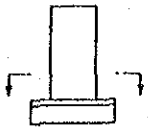
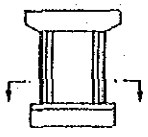
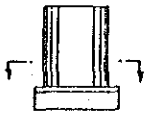
Table 5.2-8 Type of Abutments

Height (h)	Type of Abutments	Cross Section
h<5m	Gravity-Type Autment	
5≦h <10m	Inverted T-shaped Autment	

(2

Piers are divided into two categories, those on the land and those in rivers. Considering the flow velocity and drift, oval cross sections were adopted for piers in the rivers, while rectangular cross sections were chosen for those on the land from the viewpoint of easy construction method. Type of pier is shown in Table 5.2-9.

Table 5.2-9 Type of Pier

	Type	Cross Section	Remarks
On Land			Point A Br.
In the Water Flow			Patuni Br. Pt. Leon Br.
			Alto Choro Br. San Silverio Br. San Lorenzo Br. Espiritu Br.

(3) Foundations

Field investigations and geological surveys revealed that there were exposed or near-surface bedrock layers at the construction sites. Therefore, in this project, spread foundations are to be used for all sites assuming that good bearing layers near the ground surface are present.

5.2.4 Preliminary Design of Bridges

(1) Design Conditions

The design of the bridges conforms to AASHTO "Standard Specifications for Highway Bridges." The main design conditions are as follows:

a) Load : Trailer load (HS20)
b) Width : See Figs. 5.2-2 to 5.2-4.
c) Foundations : Spread foundations
d) Materials : Specified design strength of concrete(ck):
PC members ck = 350 Kg/cm^2
PC members ck = 210 Kg/cm^2
PC steel cable : G270
Reinforcing bars : Grade60
(fg = 4,200 Kg/cm^2)

Note : "PC" and "RC" above stand for "prestressed concrete" and "reinforced concrete" respectively.

(2) Preliminary Design

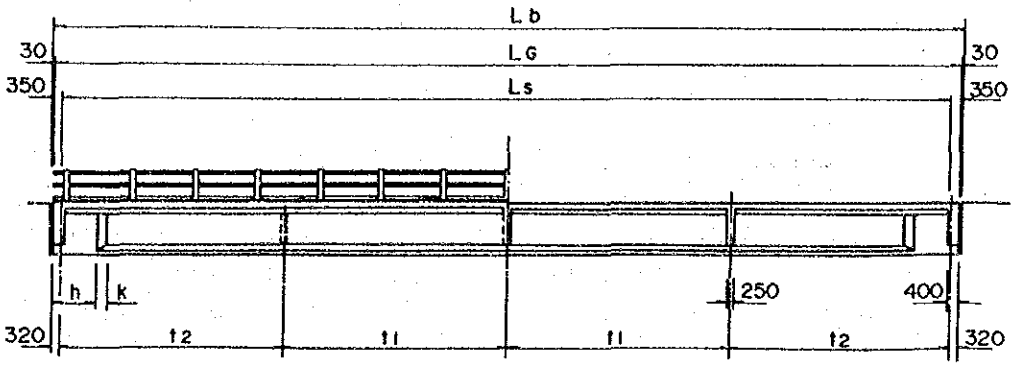
The type of superstructure employed in this project is the one most commonly used in Bolivia. In the preliminary design, cross sections and other details were determined solely on the bases of historical and other data available in the country without performing structural or other calculation. Figs.5.2-2 to 5.2-4 show the cross sections that were determined. PC composite girder bridge Group I and II are the most popular bridge. The BPR girder type is adopted for the shape and size of girder in this study. The BPR girder is developed and recommended by the Bureau of Public Road, AASHOTO and Prestressed Concrete Institute.

The PC Box girder bridge(Group III) is adopted for long span bridge. The girder sections maintained as equal girder sections type considering the constructions method of timbering and minimum main girder height is 1.5 meters.
The size of the other structures is followed the Alto. Beni Bridge as shown in Fig.5.2-3 to Fig.5.2-4.
The center span of Points A bridge (Group III) is 80.0 meters and girder height of center and end of span are 1.5m and 4.5m respectively. The web is adopted as diagonal web due to the reinforced concrete web.

5.2.5 Construction Plan

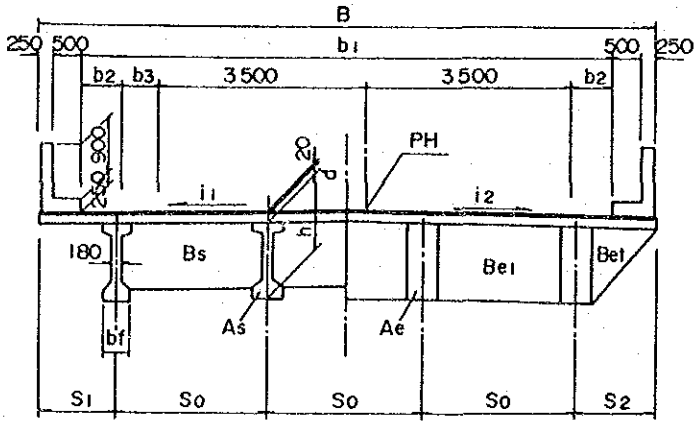
Methods to be used for erecting the structure were chosen according to the topographical and geological characteristics, river conditions and the type of bridge to be used at each individual construction site. All construction sites have good foundations, and the river flow during the dry season is low. However, due to the steep topography at the construction sites and the possibility of erecting the structures during the wet season, the beam staging method, in which girders are fabricated on-site, is to be used for all bridges except for one at Point A (see Fig.5.2-5).

The valley at Point A is 50m deep, and the side spans are relatively small compared with the center span.
Therefore, the side spans will be erected first using the staging method, and then the center span will be erected using the cantilever method while employing the side spans as counterweights. In order to prevent overturning during



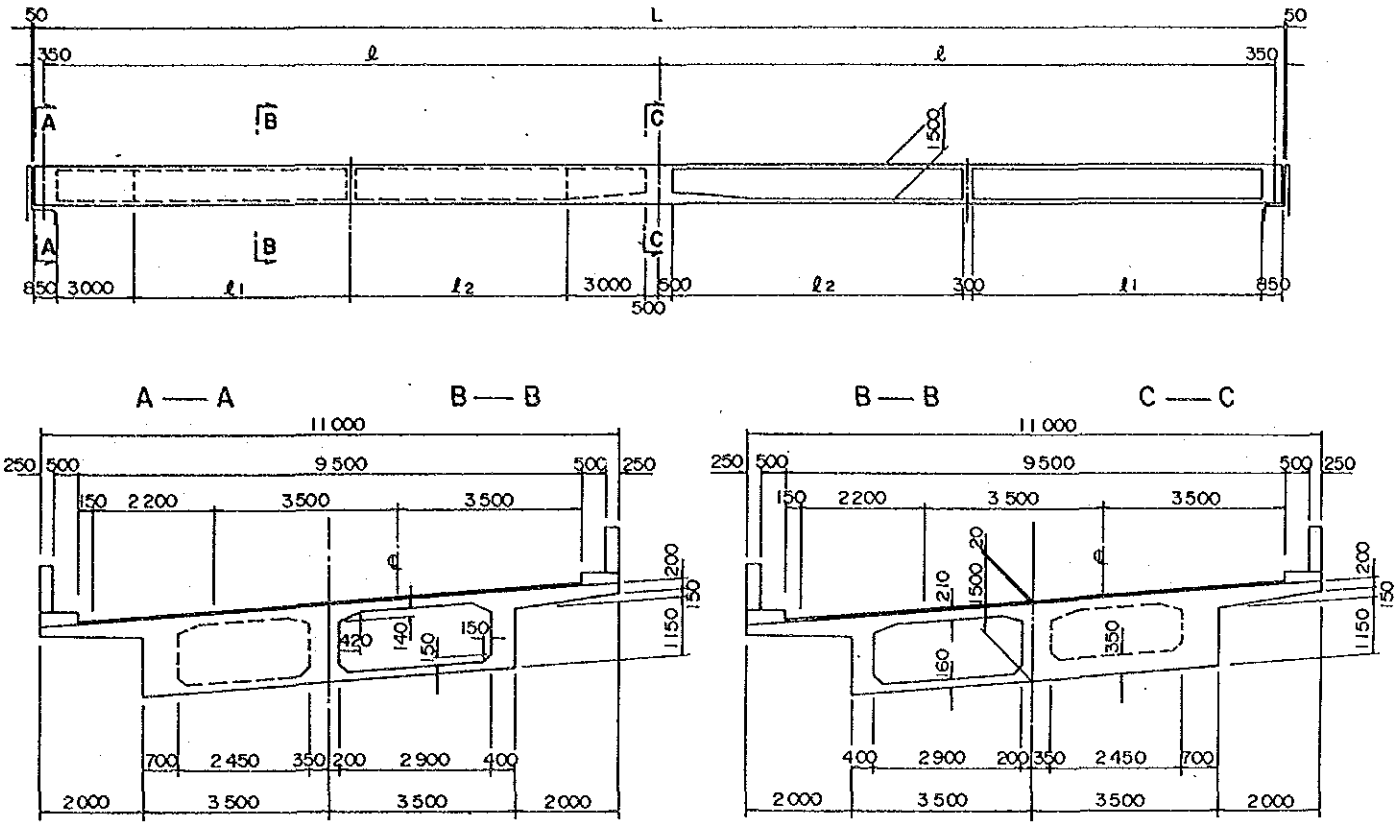
(Unit : m)

Name of Bridges	Total Bridge Length (L_B)	Girder Length (L_G)	Span (L_S)	Cross Beam				K
				T_1	T_2	h_t	m	
Patuni	40,000	19.94×2	19.65×2	6,400	6,450	0.770	2	0.40
Challa	20,000	19.94	19.3	6,400	6,450	0.770	2	0.40
Cascada	18,500	18.44	17.8	6,140	6,150	0.770	2	0.40
Pto.Leon	75,000	24.94×3	24.65×3	6,000	6,150	0.975	3	0.40
Cajones	25,000	24.94	24.3	6,000	6,150	0.975	3	0.40
Chojña	22,000	21.94	21.3	7,100	7,100	0.870	2	0.40
Carrasco	30,000	29.94	29.3	7,350	7,300	1.115	3	0.50



Name of Bridges	Width				Spaceing of Girder			Slab	Girder		
	B	b_1	b_2	b_3	S_0	S_1	S_2	d	h	b_f	n
Patuni	11.90	10.40	0.60	2.00	2.40			0.19	1.10	0.51	5
Challa	11.90	10.40	0.60	2.00	2.40	0.64~ 1.804	1.660 ~ 0.753	0.19	1.10	0.51	5
Cascada	10.50	9.00	1.00	–	2.60	1.35	1.35	0.19	1.10	0.51	4
Pto.Leon	8.80	7.30	0.15	–	2.20	1.10	1.10	0.18	1.50	0.51	4
Cajones	10.50	9.00	1.00	–	2.60	1.35	1.35	0.19	1.50	0.51	4
Chojña	10.50	9.00	1.00	–	2.60	1.35	1.35	0.19	1.20	0.51	4
Carrasco	13.00	12.00	2.50	–	2.60	1.30	1.30	0.19	1.70	0.56	5

Fig. 5.2-2 Dimension of Superstructure for Group I and II Bridges



Name of Bridge	Bridge Length	Center of Girder (m²)			
		L	ℓ	ℓ₁	ℓ₂
Alto Choro	50.000	48.038	23.669	8.400	8.269
San Silverio	50.000	48.038	23.669	8.400	8.269
San Lorenzo	52.000	50.778	25.039	9.100	8.939
Espiritu	52.000	50.778	25.039	9.100	8.939
Avaroa					

Fig. 5.2-3 Dimension of Superstructure for Group III Bridges

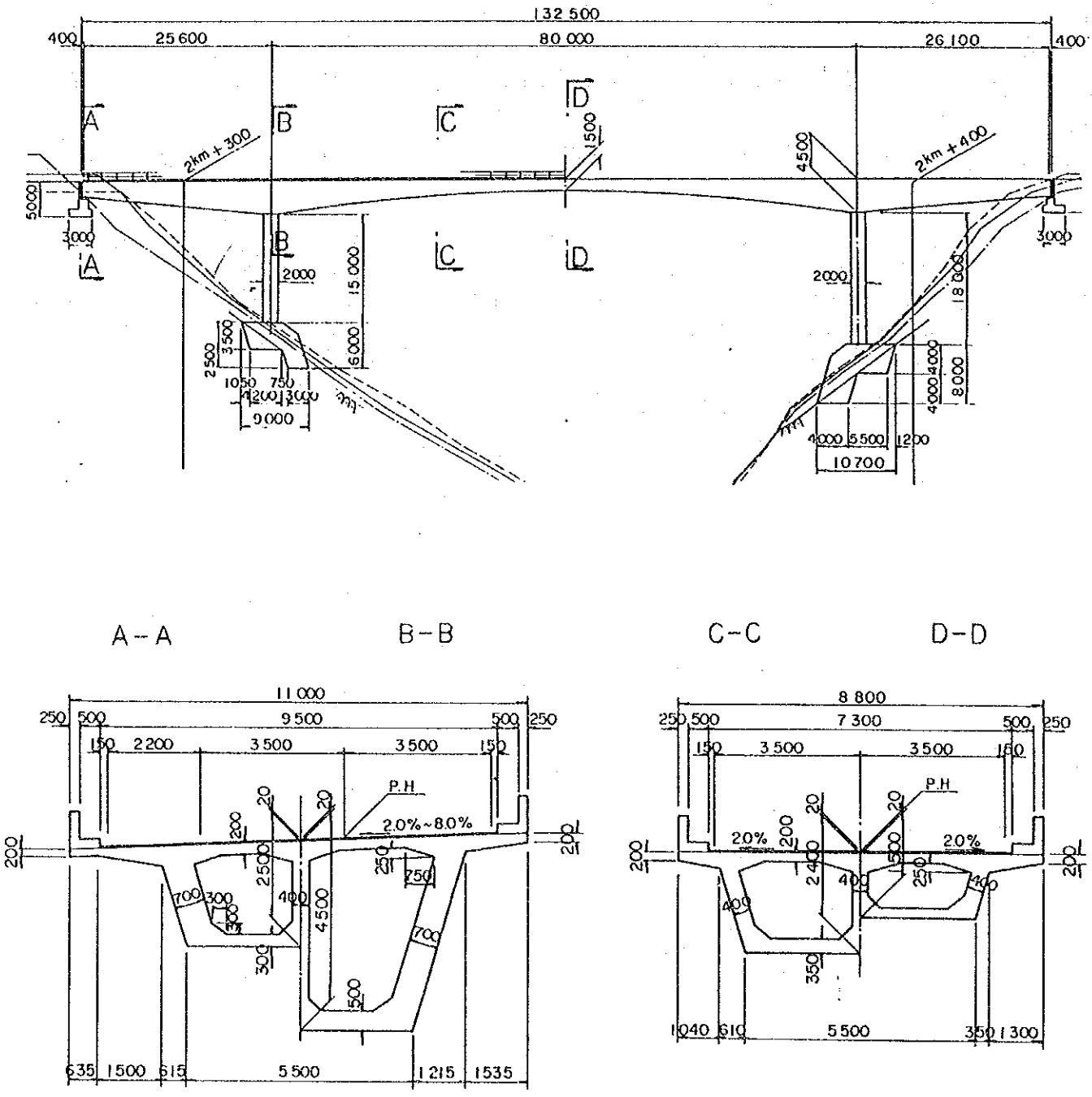


Fig. 5.2-4 Dimension of Bridge at Point A

this, rock anchors will be placed around the abutments to support the girders in the side spans.

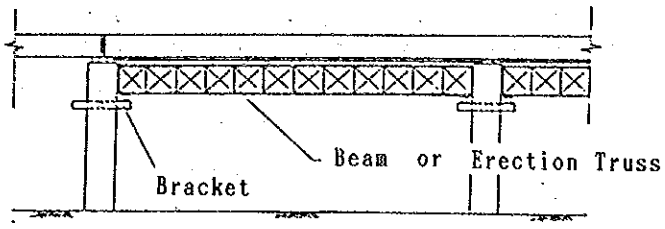


Fig. 5.2-5 Beam Staging Method

5.2.6 Material

The specified design strength (C_{28}) of the concrete used in the bridges will be 350Kg/cm^2 for prestressed concrete and 210Kg/cm^2 for reinforced concrete. Aggregate could be taken from the Coroico River and the Yara River: However, according to SNC's material laboratory, the maximum compressive strength of concrete that can be made using this aggregate is as low as 180Kg/cm^2 which is not adequate for these bridges. Consequently, it was decided that concrete aggregate for the bridges will be made using gravel from Alto Beni and sand from Suwapi which were previously used for Yara Bridge. Alto Beni and Suwapi are located about 10Km toward Quiquibay and about 25Km toward Pto Linares from Bella Vista.

5.3 Structural Design

5.3.1 Design of Tunnels

The No.35+500 - No.36+400 subsection of the existing road is narrow and has a complicated horizontal alignment. In addition, the topography on both sides of the road is extremely steep, making road improvement very difficult. In section 4.3.3, the tunnel method and widening method were compared. As a result, the tunnel method has been adopted for economical and technical reasons. This section outlines the preliminary design of the two tunnels.

(1) Topography and Geology

1) Topographical Conditions

Along the planned tunnel section on the existing road runs the Coroico River. Two ridges meet almost perpendicularly to form this river. The Cajones River flows down between these ridges, and there is a waterfall here, too. The ends of the ridges form a precipitous cliff, which is about 150 m in height and about 500 m in length (see Photos 8 and 9). The tunnels are to be planned through these ridges.

2) Geological Conditions

Since the planning of tunnels in this area has been considered since the early stages of the project, boring surveys and rock specimen tests was conducted. The geological structure of the two ridges showed it was composed of Paleozoic layers of slate, which was gray, well-compacted and fresh. There were few cracks, and the unconfined compressive strength averaged 530 kg/cm2.

(2) Alignment Design

1) Design of the Horizontal Alignment

The construction cost of tunnel is much higher than that for earth works road. To minimize the construction cost, it is desirable to make the tunnels as short as possible. In this project, straight tunnels was planned for the following reasons:

a) Straight tunnels minimize the length, thus reducing the construction cost.
b) Since this area is not supplied with electric power, the tunnels should be straight to ensure visual safety.

The horizontal alignment around the tunnel entrances should have as large a radius of curvature as possible to prevent traffic accidents. Due to the limitations in local topography conditions, however, the maximum radius of curvature possible was R=90. Besides the above considerations, the horizontal alignment was designed so that it was almost

perpendicular to the slope of the mountains, in order to minimize the influence of any uneven earth pressure.

2) Design of the Vertical Alignment

In view of the tunnel construction methods and the influence of automobile exhaust fumes, the gradient of the tunnels should be kept to the minimum. A gradient of 4%, however, was used considering the need to make the tunnels as short as possible and for a suitable connection to the Cajones Bridge.
By taking into account the above factors, the length of the two tunnels was set at 370 m and 365 m, with which, the influence of exhaust fumes is considered to be minimal. The horizontal and vertical alignments are illustrated in Figs. 5.3.1 and 5.3 2.

(3) Design of the Cross Section

The inner section of the tunnels must accommodate the construction gauge described in section 4.1.4 (see Figs. 4.1-2 and 4.1-3). The geometry of the inner section of the tunnels which are designed considering safety and the volume of earth to be excavated is shown in Fig. 5.3-1.

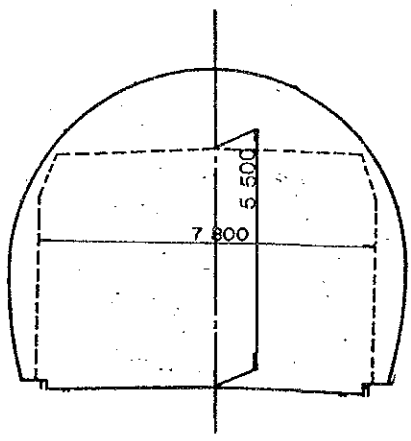


Fig. 5.3-1 Inner Section of Tunnels

(4) Structural Design

1) Internal Structure of the Tunnels

Since the soil conditions at the construction site of the tunnels composes of fresh, hard rock and have few cracks. The interior surface of the tunnels are reinforced with wire netting, rock bolts and sprayed concrete to prevent rock falls.

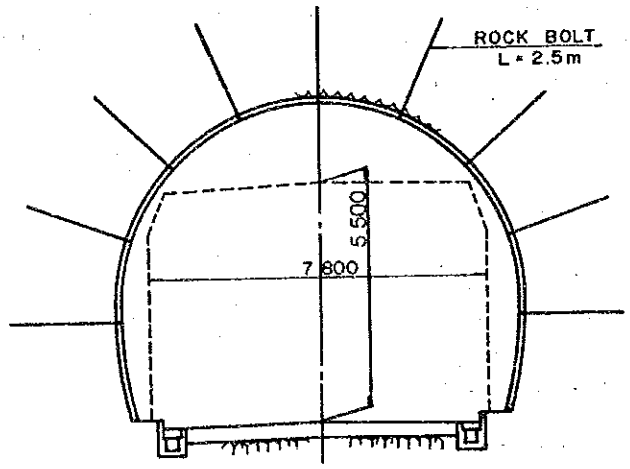


Fig.5.3-2 Internal Structure of Tunnels

2) Structure of Tunnel Entrance

The sub surface ground conditions at the construction site is fairly good as a whole, but some parts of the ground conditions around the tunnel entrance is likely to contain surface soil as well as cracks and weathered rock. To secure safety, the first 30 m section of the tunnels are reinforced with a concrete lining.

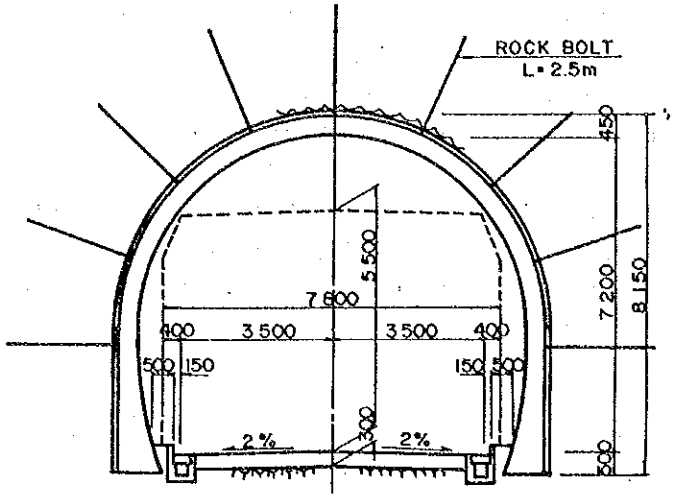


Fig. 5.3-3 Structure of Tunnel Entrance

5.3.2 Disaster Prevention Facilities

In this clause, the preliminary design for optimum disaster prevention countermeasures are discussed. The preliminary designs were attained after conducting studies on the selected disaster prevention works. Typical locations with regards to each structure, and practical experience from other similar road construction were considered.
Those design considerations are described for each disaster prevention measure, below;

(1) Type 1 : Sprayed Concrete

The structure is contained by a 15cm thick layer of sprayed concrete, wire mesh stretched prior to spraying and anchor pins pitched to fix the wire mesh on a slope. A basefooting should be installed at a toe of slope, to prevent the sprayed concrete sliding down. A structure is illustrated in Fig.5.3-5(1).

(2) Type 2 : Stone Masonry Retaining Wall

As illustrated in Fig.5.3-5(2), the structure is contained by cobble stones (35cm in thickness), backfilled concrete (20cm in thickness) and other material (40cm in thickness). The other backfilled material is indispensable in order to the drain water off from the reverse side of the slope.

(3) Type 3 : Concrete Crib with concrete spraying and Anchoring

This slope protection measure consists of a reinforced cast-in-place concrete frame and anchor bars, a coated with mortar grouting, which is placed at cross points on the frame in order to fix it on the slope. Spaces between the frames are filled with sprayed concrete to prevent weathering. The structure is illustrated in Fig.5.3-5(3).

(4) Type 5 : Concrete Pitching and Anchoring

A reinforced concrete structure with anchor bars should be pitched on the entire surface of slope. The structure is illustrated in Fig.5.3-5(4).

(5) Type 6 : Grid Type Concrete Retaining Wall

As illustrated in Fig.5.3-5(5), the structure is contained by erected concrete members. The retaining wall should be applied practically depending on the gradient and height as shown in Fig.5.3-4.

(6) Type 9 : Catch Netting

This measure for rock falls is illustrated in Fig.5.3-5(6) and is contained by wire mesh and wire rope stretched along the slope. Additionally anchor bars and/or anchor pins pitched to fixed wire mesh on the slope.

(7) Type 10 : Gabion Catch Wall/ Type 11 : Catch Fence in stalled on the road side

The design considerations for these catch facilities are as follows:

- Road side clearance of 3.0m width should be allowed for the deposit of fallen debris.
- Height of the wall or fence should be 2.0m. (It has been shown that the jumping height of boulders is generally up to 2.0m from previous experience.)

The structure of catch fence is illustrated in Fig.5.3-5 (7).

(8) Type 14 : Sub-surface Drainage for Landslide protection

A sub-surface drainage system should be applied taking the following considerations:

- collect ground water using gravel
- spread gravel under the ditches on top of slopes
- spread gravel along the top of each slope
- make a channel to drain the collected ground water to the vertical drainage system.

A structure is illustrated in Fig.5.3-5(8).

(9) Type 15 : Gabion Dam for Debris/Earth Flow

Design consideration for this gabion dam are described below:

Dam Height

A dam height should be determined by assuming that the entire deposit of silt may by caught in the dam.

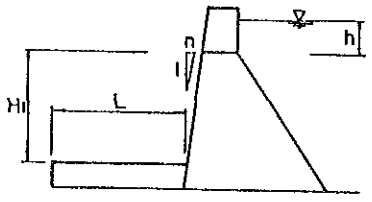
Crown Width

The crown width is determined by taking two times the maximum diameter of the deposited debris. Thus, since the assumed maximum debris diameter in the locations selected for the gabion dam is 1.5meters, a crown width of 3.0m was selected.

Apron

The apron lengths for each dam were determined using the following equation:

$$L = 1.5\ (h + H1) - nH1$$



where

L = Apron Length (m)
h = Overflow depth (m)
H1 = Effective head (m)
n = Slope gradient of front

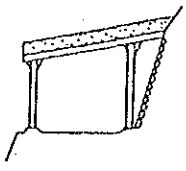
Furthermore, with regard to apron thickness, a value of 2.0 meters is normally adopted.

Depth of Embedment

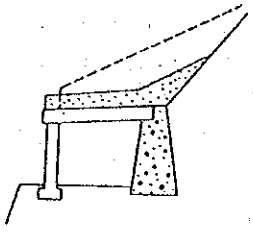
An embedment depth of 2.0 meters is normally used in the case of such bedrock, as found in the project area.
The structure is illustrated in Fig.5.3-5(9).

(10) Type 17 : Debris/Earth Flow Shed

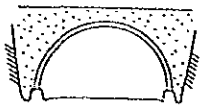
As specified in Table 4.5-3, "Comparison of countermeasure(s)", the location were they are applied the debris/Earth flow shed is limited to J + 0.6 (No.40 + 300). The following three types are generally considered for this kind of situation;



Gate Type

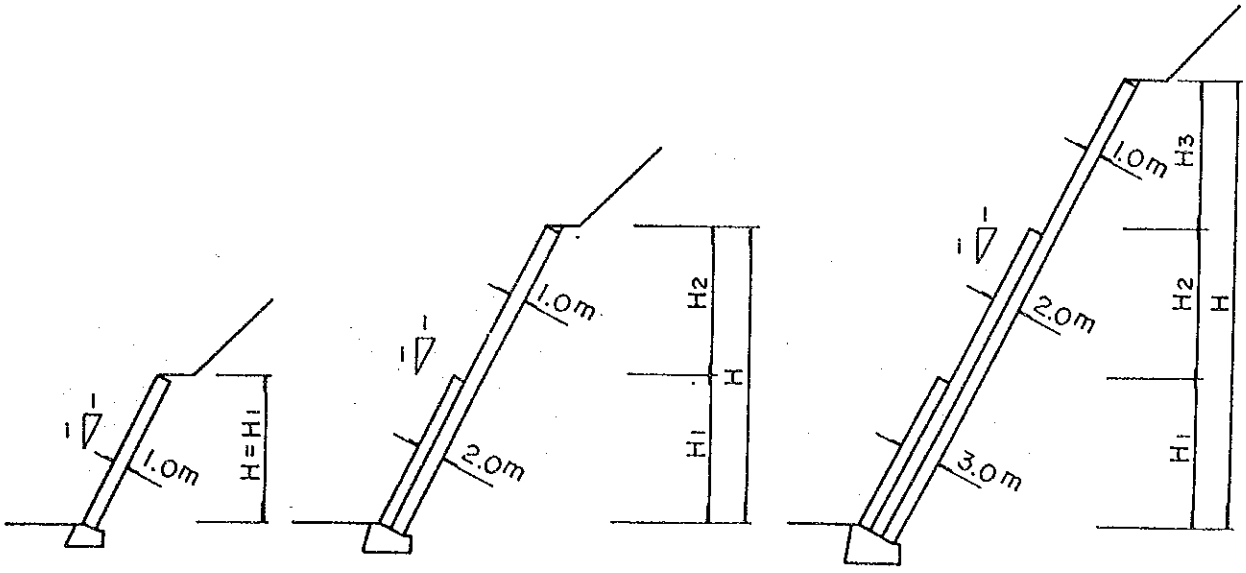


Retaining Wall Type



Arch Type

It is assumed that the slope at such a location is comparatively stable and mostly restrained by unsound rock.
Considering these conditions, it is not be necessary to protect the slope with a retaining wall. Comparing gate and arch type structures, the former is better than the latter from the view point of the easy construction. Thus, a gate type was adopted for this particular location.
Next, as to the type of material, reinforced concrete was adopted from an economical stand point.
The structure is illustrated in Fig.5.3-5(10).



Case 1 Case 2 Case 3

i = 2.0

Slope Height	Case	H1	H2	H3
H≦10	Case 1	H	-	-
10<H≦20	Case 2	10.0	H - 10.0	-
20<H≦30	Case 3	10.0	10.0	H - 20.0

i = 2.5

Slope Height	Case	H1	H2	H3
H≦7.5	Case 1	H	-	-
7.5<H≦15	Case 2	7.5	H - 7.5	-
15<H≦22.5	Case 3	7.5	7.5	H - 15.0

i = 3.3

Slope Height	Case	H1	H2	H3
H≦6	Case 1	H	-	-
6<H≦12	Case 2	6.0	H - 6.0	-
12<H≦18	Case 3	6.0	6.0	H - 12.0

Fig. 5.3 - 4 Application of Grid Type Retaining Wall

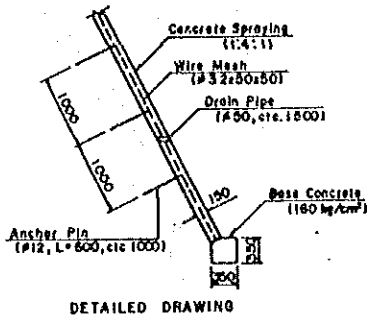


Fig. 5.3-5(1) Concrete Spraying (Type 1)

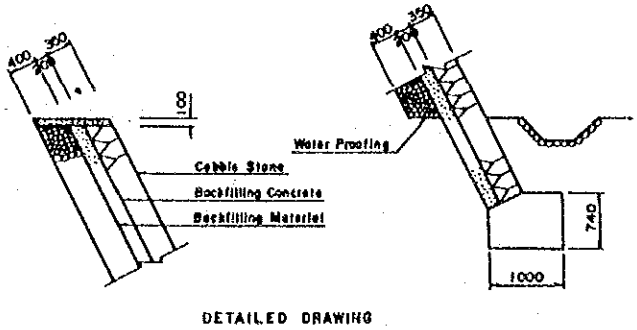


Fig. 5.3-5(2) Stone Masonry Retaining Wall(Type 2)

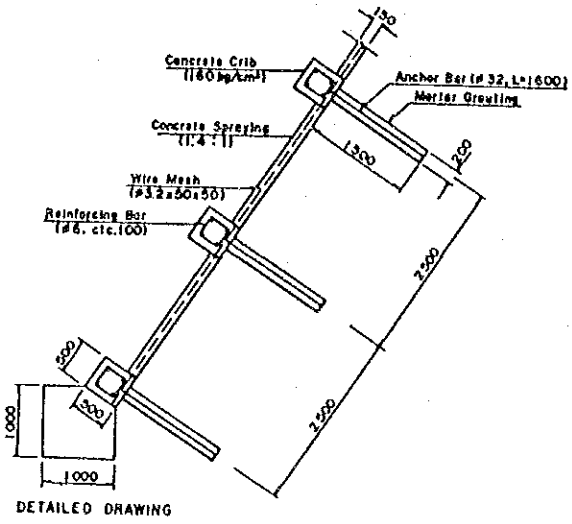


Fig. 5.3-5(3) Concrete Crib with Concrete Spraying and Anchoring (Type 3)

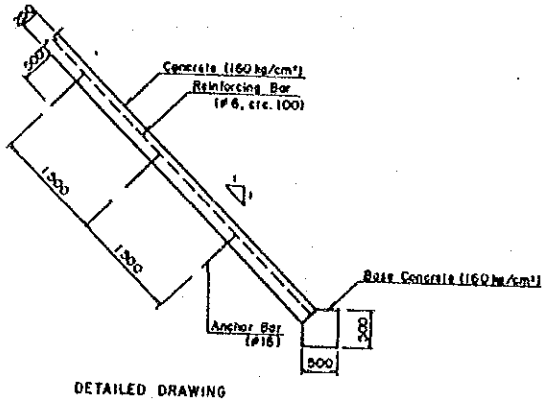


Fig. 5.3-5(4) Concrete Pitching and Anchoring (Type 5)

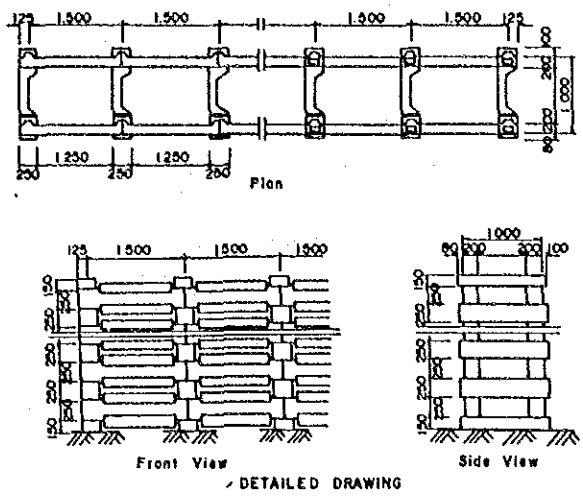


Fig. 5.3-5(5) Grid Type Concrete Retaining Wall (Type 6)

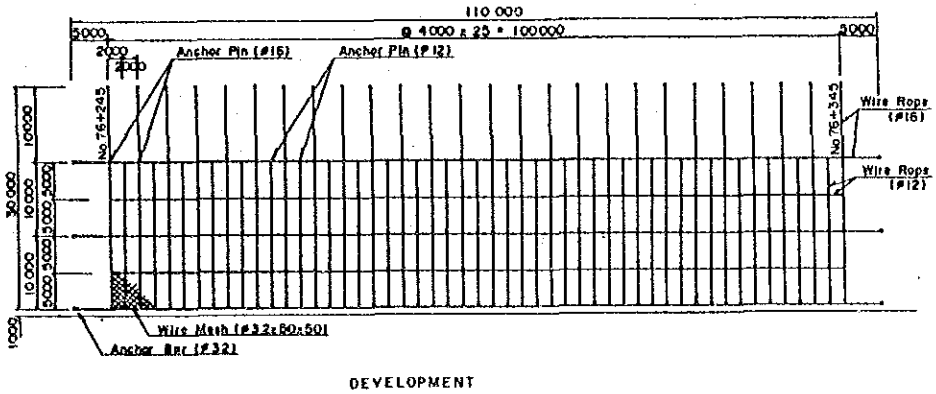


Fig. 5.3-5(6) Catch Net and Gabion Wall (Type 9/10)

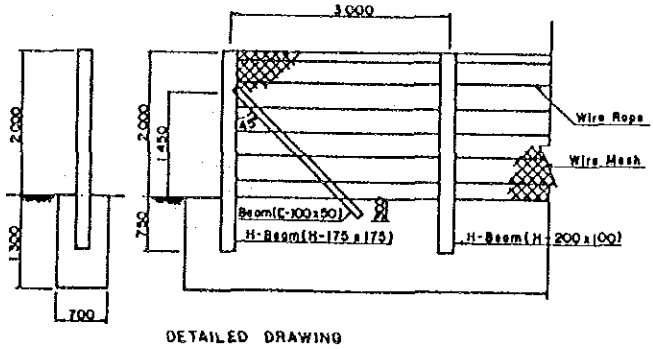


Fig. 5.3-5(7) Catch Net and Catch Fence (Type 9/11)

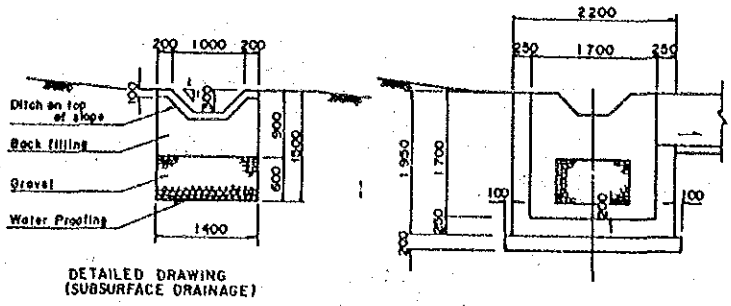


Fig. 5.3-5(8) Subsurface Drainage (Type 14)

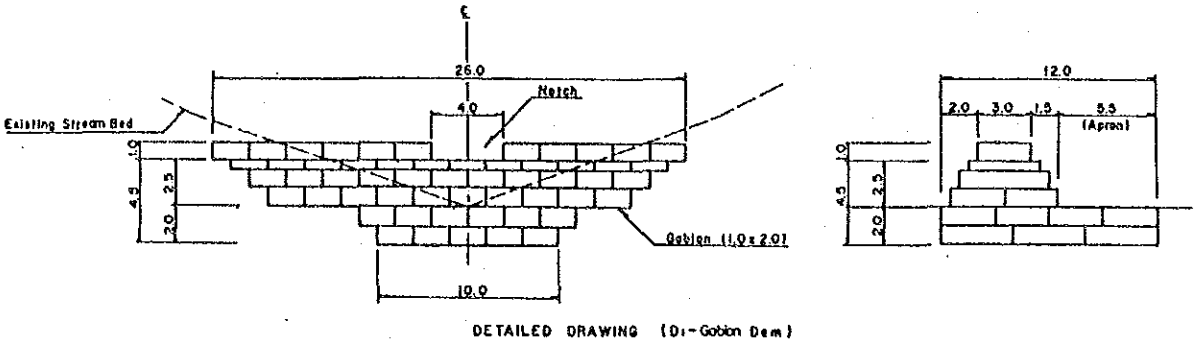


Fig. 5.3-5(9) Gabion Dam (Type 15)

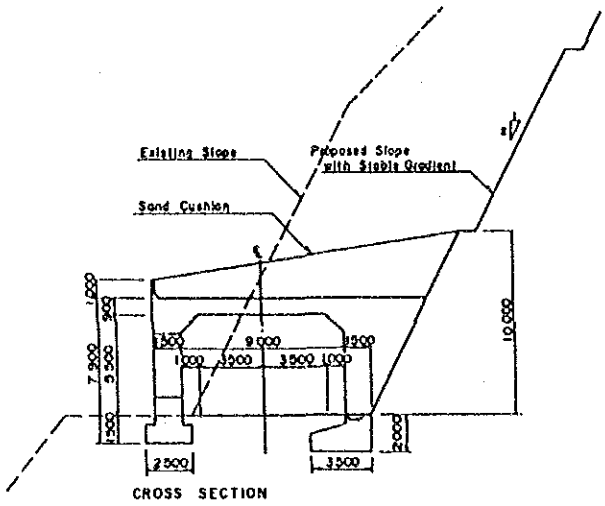


Fig. 5.3-5(10) Shed for Earth/debris Flow (Type 17)

5.3.3 Retaining Walls

As mentioned in section 5.1.2, (4), the number of retaining walls to be built was minimized. Retaining walls were used where they proved superior in safety and economy to earth work construction. The types of retaining walls employed were determined according to factors such as the intended use, allowable height and the economical factors for each individual site.

(1) Gravity Retaining Walls

Gravity retaining walls were constructed using plain concrete which was placed using cobblestone made available from the excavation as the aggregate. Since the front walls are perpendicular as seen in Fig. 5.3-6, the gravity retaining walls was employed for all backfilling work in areas consisting of steep ground.

(2) Masonry Retaining Walls

Masonry retaining walls are used widely in Bolivia and are relatively economical. This type of retaining wall can be constructed by placing locally produced rubble on the front, without the requirement for formwork. It can also achieve sufficient durability results, even with a poor concrete mix.

The cross section of this type of retaining wall can be simplified, by using highly permeable, high-quality material for the backfill behind the wall and adjusting the wall thickness, so that the earth pressure is reduced. Values of the parameters to be used for cut and fill operations filling are shown in Fig. 5.3-6.

(3) Grid Type Retaining Walls

Typically, lattice retaining walls are used when the surface area of fill on steep slopes or steep cut slopes in mountainous regions, which run parallel with natural slopes, are to be kept within certain limits. Also they are used when there is considerable seepage from inside of the slopes. Lattice retaining walls are highly permeable. Since they are made of precast concrete units, which are

filled with locally produced material and are assembled in a lattice pattern, they are particularly suited for construction in mountainous regions. When high walls are required, a double- or triple-frame structure can be built. Foundations for the frames need to be placed upon solid ground in order to prevent subsidence. Dimensions and the configuration for cut and fill operations for this are shown in Fig. 5.3-6.

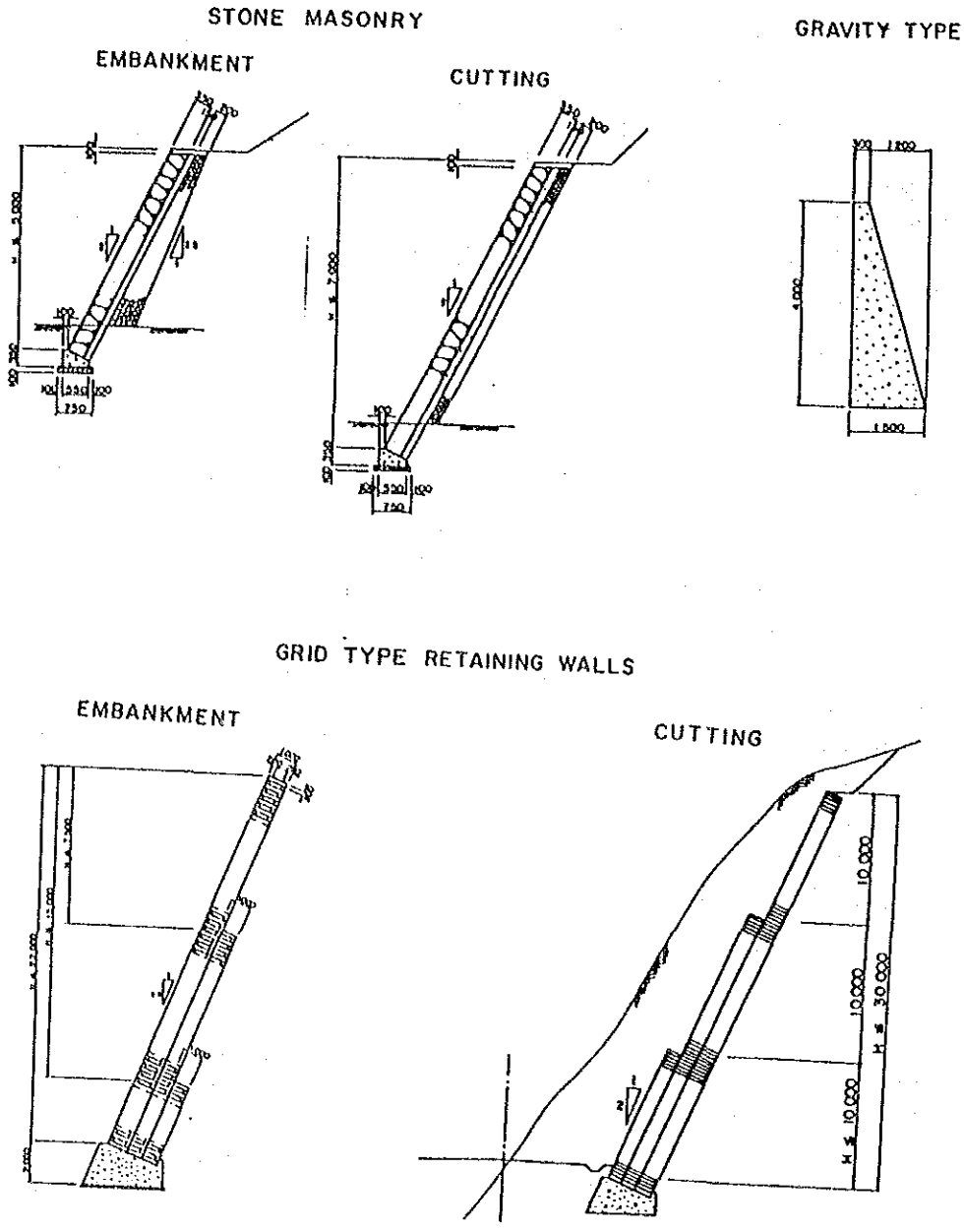


Fig. 5.3-6 Retaining Wall

5.3.4 Drainage Facilities

The major types of planned drainage facilities mentioned in 5.1.4 are as follows:

(1) Culvert Pipes

Circular culverts are used for transverse drainage of roads where water flow is relatively low. Culvert pipes that are widely used in Bolivia are concrete and corrugated metal types. In this project, concrete pipes are used for economical reasons. The economical cross section of concrete pipes is somewhere between 0.60 m and 1.80 m, and more than two pipes should not be used in one place.

(2) Box Culverts

Box culvert are used when the transverse drainage requirements exceed the capacity of culvert pipes. The dimensions of the box culverts determined by flow calculations for this project are 3.0 m x 3.0 m and 4.0 m x 4.0 m. A reinforced concrete structure was adopted. Since the flow velocity of the streams where box culverts will be provided for this project is high, water inlet and outlet structures were planned so that scouring of land fill is prevented.

(3) Roadside Gutters on Cut Slopes

Stone gutters as shown in Fig. 5.3-7, which are the most widely used in Bolivia, were adopted. Stone gutters are an economical solution since they can be constructed using rubble produced during the project.

The geometry of the gutters was determined taking into consideration the following points:

A slightly gentle gradient of 45 degrees was adopted for the side walls allowing for wheel load on the road side (and the earth pressure) and the slope stability for the toes of cut section. Slightly larger than required cross sections were adopted for the slopes, so as to allow for the shrinkage of the cross-sectional area of flow, due to sedimentation caused by erosion of the slopes.

(4) Ditches at the Top of Slopes

As in the case of gutters on cut slopes, stone gutters were also adopted for the purpose of drainage at the toe of slopes. Since the loading and earth pressure acting on these gutters is smaller than that on the gutters of cut slopes, a slightly sharper gradient was adopted for the side walls, as shown in Fig. 5.3-8.

(5) Berm Ditches

Precast gutters with a high degree of workability were adopted here, since the berm ditches are provided along narrow berms (W=1.5 m). A slightly larger cross section (30 cm x 30 cm) relative to the volume of flow was adopted for the gutters since they were not ideally located for easy maintenance. The dimensions of the gutters are shown in Fig. 5.3-9.

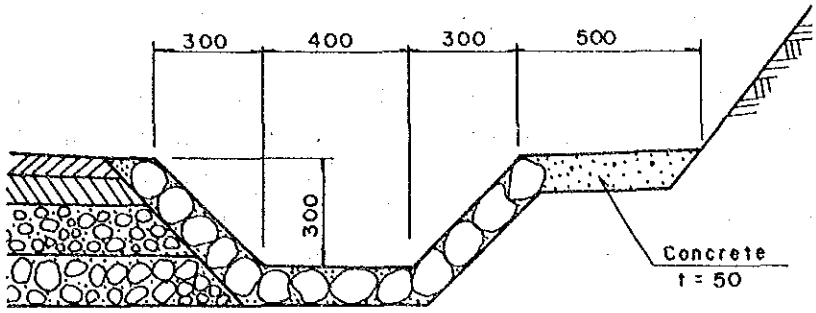


Fig.5.3-7 Details of Side Ditch

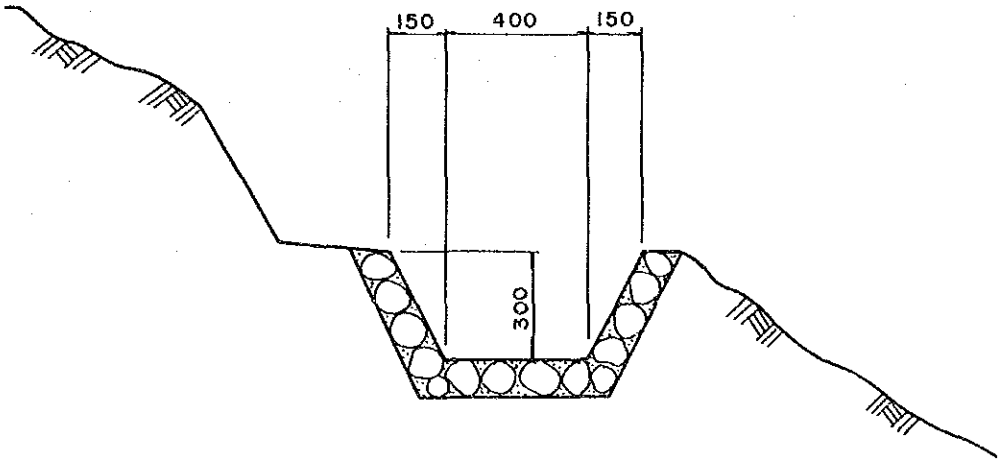


Fig.5.3-8 Drainage at Toe of Slope

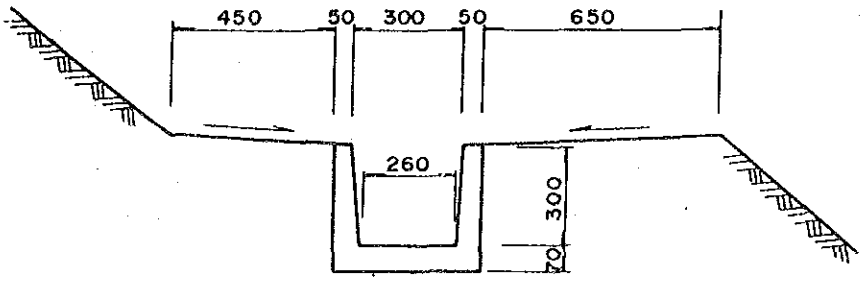


Fig. 5.3-9 Drain Ditch on Berm

5.4 Earthmoving Planning

5.4.1 The Volume of Excavated Soil

The total volume of excavated soil produced in the Santa Barbara - Bella Vista section amounts to about 10,220,000 m^3. Of this about 1,513,000 m^3 is utilized for embankment material and the remaining 8,707,000 m^3 is discarded. The volumes of excavated soil are shown in Fig. 5.4-1.

(1) Excavation

More than half (approx. 5,514,000 m^3, or 118,000 m^3/km) of the amount of soil produced from total excavation,occurs in the subsection between the starting point and point (K) (No.46+750). The amount of soil produced from excavation within the point (K) - point (Q) subsection (No.79+550) is as little as 2,063,000 m^3 (63,000 m^3/km), while that within the point (Q) - point (V) subsection (No.101+30) increases to 2,303,000 m^3 (106,000 m^3/km). The amount of excavated soil in the subsection between point (V) and the end point (No.108+630) again decreases to 330,000 m^3 (45,000 m^3/km).

The major factor contributing to the greater amounts of excavated soil in the above two subsections is the steepness of the topography. The faces cut for the existing road are mostly perpendicular, instead of being set at angles specified by the relevant codes in accordance with the geological conditions. There are even some places where cut faces overhang (Photo 5) without any protective or reinforcing construction. Since the improvement plan for this project was drawn up in accordance with the specified gradients, the maximum cutting height at some places has reached 50 m.

In addition, a minimum radius of curvature of 50 m or more was secured, and the gradient was kept within 7% (specific gradient i=8%, L 400 m) in order to correct the undesirable alignment of the existing road. This resulted in considerable changes in alignment, causing the production of a total of more than 10,000,000 m^3 of excavated soil for the overall sections.

(2) Embankment

The amount of soil required for land embankment in the section between the starting point and point (K) is as small as 264,000 m^3 (6,000 m^3/km), while that in the section between point (K) and the end point is 1,249,000 m3 (20,000 m^3/km). On the contrary, the amount of soil used for embankment in the point (Q) - point (V) subsection is as great as 643,000 m3 (30,000 m^3/km), the greatest for this project.

The amount of soil required for embankment is as small as approximately 1,500,000 m^3, compared with the amount of soil produced from cutting which is approximately 10,000,000 m^3. One of the reasons for this is that since the topography between the starting point and point (K) is steep and embankment cannot be utilized due to the Coroico River which runs parallel, the alignment had to be shifted upward towards the mountains.

(3) Unusable Soil

Since the difference between the amount of excavated soil and that soil required for embankment is great, the volume of unused soil totals as much as 8,710,000 m^3, even if all soil utilized for embankment is taken from the excavated soil.

Although a considerable amount of surplus soil is expected from the cutting operation between points (Q) and (V), there is not a great amount of waste soil here due to the comparable embankment requirement. By contrast, since the embankment requirement is very small between the starting point and point (K), the surplus soil produced here (5,250,000 m^3, or 113,000 m^3/km) accounts for about 60% of all the excess soil.

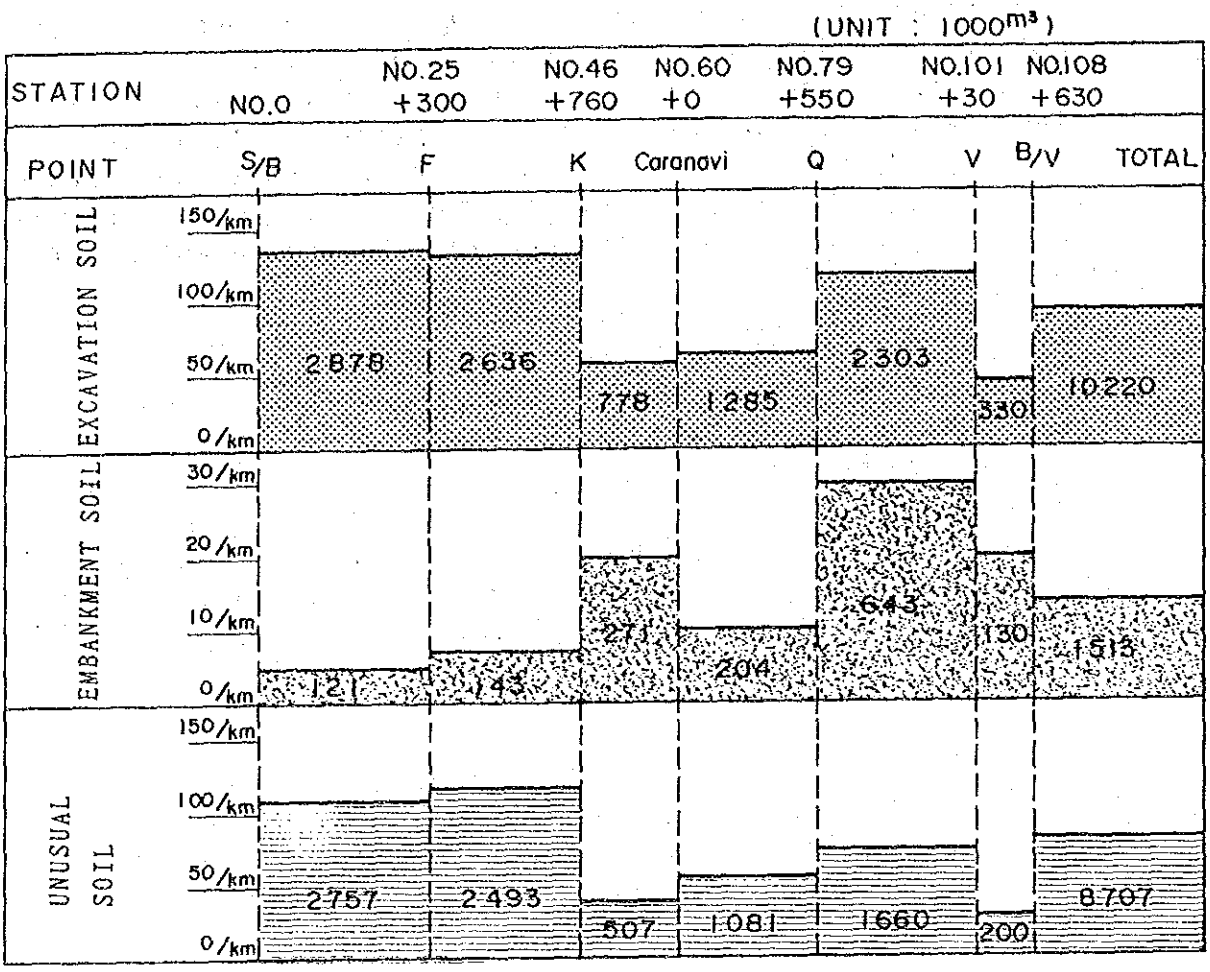


Fig.5.4-1 Volume of Excavated Soil

5.4.2 Spoil-Bank Plan

Finding spoil banks for a total of 8,710,000 m^3 of waste soil was the most difficult part of this survey. Based on the requirements for shortest possible transportation distance and minimized construction cost, wastelands and woodlands in the wide dry riverbeds of the Coroico River, isolated from croplands such as farms and orchards, were selected.

In the cases where the use of such croplands as spoil banks is inevitable, these areas will be either purchased through negotiation with the land owners; or rearranged after the

dumping of waste soil and covered with a one meter thick or so surface soil layer so that they may be used again for crops. Cultivating new cropland as substitute land may be another solution.
Spoil banks located on the dry riverbed of the Coroico River may be used as cropland, too. This can be done by preparing slopes and other land surfaces, and covering them with a surface soil as mentioned above, while keeping the required width of the river. This will increase the area of arable land available. As for the protection of the slopes, it will be necessary to cover them with large rocks or provide wire gabions so that any failure of the slope will not dam the river.

In about a 7 km section between the Choro River (near No.26) and the Quitacalzon River (near No.33), the difference in elevation between the existing road and the Coroico River is small, and the dry riverbed is narrow. In this section, there is about 1,100,000 m3 of surplus soil, and it is difficult to find spoil banks for this waste. The only large spoil bank in this section is a 35,000 m3 riverbed in San Pedro (near No.30), where about 500,000 m3 of surplus soil will be dumped.

Beyond No.80, the road runs away from the river. Spoil banks, therefore, were selected along the gentle hillsides and valleys.

There are 30 spoil banks within the starting point - No.40 subsection, 15 within the No.40 - No.80 subsection, and 20 within the No.80 - end point subsection, making a total of 65 spoil banks overall. There are five large spoil banks with a capacity of 500,000 m3 or more between the starting point and Caranavi. At most spoil banks on dry riverbeds, the surplus soil is dumped directly from the existing road.

5.4.3 Soil Transportation Plan

Since there are 65 spoil banks within the total excavation length of 108 km, the length of a single excavated section per spoil bank averages about 1.7 km. Given the excavation length of 1.7 km per spoil bank, the average transportation distance to a spoil bank is a little less than 500 m. In

some places, however, the surplus soil cannot be dumped directly from the existing road and needs to be transported via an access road. Therefore, to account for this, the average transportation distance for the project was set at 1 km. The spoil bank and surplus soil transportation plans are illustrated in Fig. 5.4-2.

Overall, an economical soil transportation plan could be formulated since the transportation distance averaged a figure as short as 1 km. It is imperative, however, that proper care be taken so as not to cause environmental destruction. It is also necessary to take into consideration the high water level during the wet season, as well as the stabilization of spoil banks and the proper preventive measures against failure.

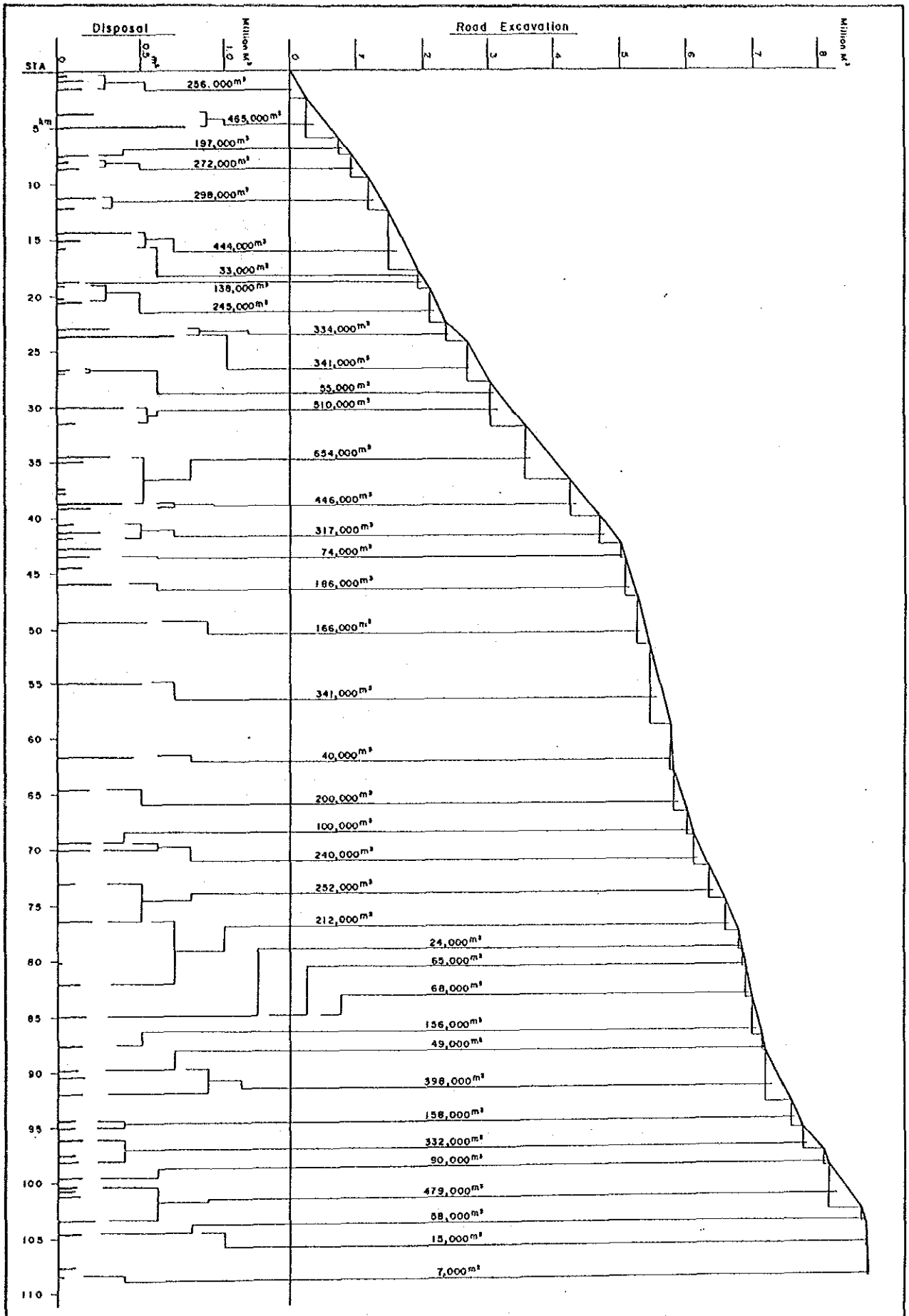


Fig. 5.4-2 Spoil-Bank and Transportation Plan.

5.5 Maintenance

Proper maintenance and repairs must be carried out to ensure the quality of the road as built and keep it in good condition. Maintenance of the road includes both preventive maintenance to keep the road in a serviceable condition and repairs to restore the serviceability of the road if damaged.

5.5.1 Maintenance System

Like other national roads, this road section will be directly maintained by SNC. The existing five maintenance offices will be responsible for the maintenance of the road, as nominated (see Fig. 5.5-1).

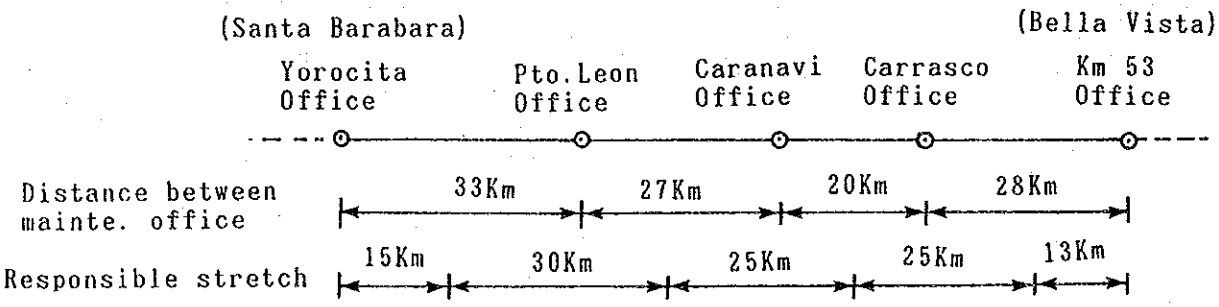


Fig. 5.5-1 Proposed Maintenance System

5.5.2 Scope of Maintenance and Repair Work

The maintenance and repairs of the road include regular inspection, road-related work, slope-related work, cleaning of the drainage facilities, and clearing of trees in the road area.

(1) Inspection

a) To watch for any abnormality and damage on and around the road which may affect the road structure
b) To watch for any obstacles and hazards which may hinder traffic on the road
c) To monitor traffic on the road

A patrol team consists of one maintenance worker, one driver, and one patrol car (pickup).

(2) Road-Related Work

The surfaces of the road can be classified either as gravel or asphalt pavement.

a) Gravel Pavement

Since pot holes, corrugation, bents, and thinned road surfaces accelerate any existing damage, it is necessary to efficiently supply materials for repair. Equipment used includes dump trucks, motor graders and rollers. A maintenance team consists of a manager, an operator, and a driver.

b) Asphalt Pavement

Damage to the surface of the asphalt pavement include pot holes, faulting, local cracking, and bents. Patching is widely used for repair. Equipment used includes trucks and small vibrating rollers. Materials used include crushed stone, sand, prime coat, tack coat, and asphalt. A maintenance team consists of a manager, an operator, a driver, and a laborer.

(3) Slope-Related Work

In addition to regular inspection, it is necessary to perform patrols after unusual severe weather such as storms to monitor the changing road conditions, while preparing for damages and defects at the early stages.

(4) Cleaning of Drainage Facilities

Since damage to roads is often caused by water, it is important to keep drainage facilities in a good condition. What should be noted in the project area is that if driftwood or other matters clogs the inlets of the lateral drainage facilities, overflow of water could cause damage to the road. Appropriate inspection, monitoring and cleaning need to be carried out so that the drainage facilities can function properly. Work here includes cleaning to be done either manually or by using tractor shovels (wheel type).

(5) Clearing of Trees along the Road

In the project area, there are many native trees which grow rapidly. Consequently, poor sunshine and draft delay the drying of the road surface, and a wet road surface tends to cause premature wear due to friction between road surface and car wheels. These trees may also increase clearance limits and reduce sight distances. Periodic clearing of these trees is needed to prevent such problems. These works are done either manually or by using tractor shovels (wheel type).

6. COST ESTIMATES

6. COST ESTIMATES

6.1 Cost Breakdown

6.1.1 Cost Breakdown

The project cost was broken down as shown in Fig. 6.1-1.

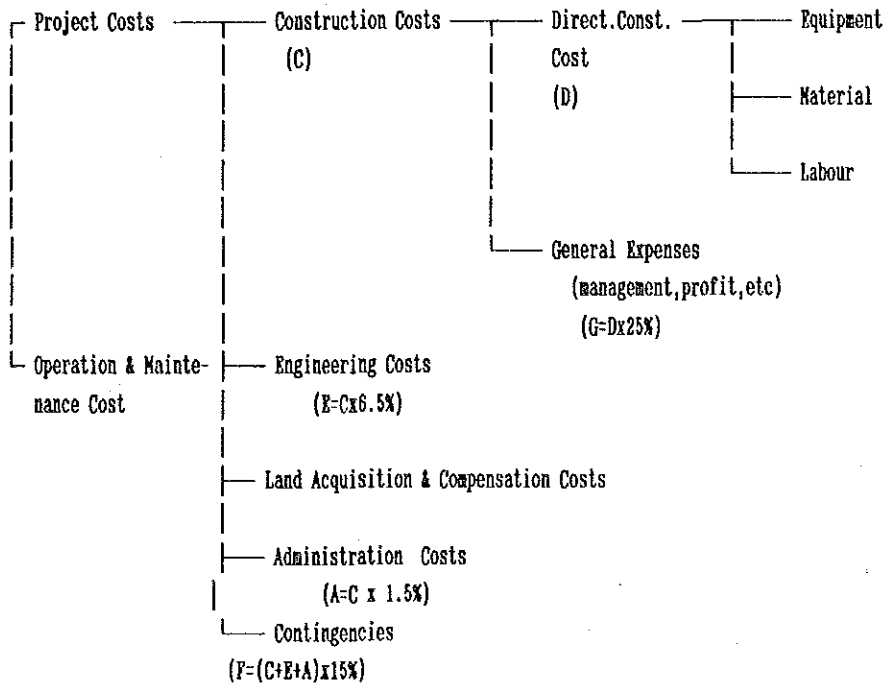


Fig. 6.1-1 Cost Breakdown

Percentages indicated in Fig.6.1.1 were determined considering the size of this project and the costs of past projects implemented by SNC in Bolivia.

6.1.2 Separation into Foreign and Local Currency Portions

The local currency portion of the construction cost includes:

a) cost of materials purchased in Bolivia, e.g., gasoline, lubricants, propane gas, cement and bricks,
b) labour costs,

c) costs of land acquisition and compensation, and
d) taxes and duties for imported materials and equipment.

The remainder of construction costs will be financed by foreign currency. Engineering and administration costs will be financed in foreign and local currency, respectively.