Chapter 5
Design Standard

Chapter 5 DESIGN STANDARD

The geometric design basically employs the "Standard Specification for Geometric Design of Road"(December 1990) of Bina Marga together with AASHTO and Japanese design standards. The Bina Marga standards are in the same group with AASHTO, and Japanese design standards.

5.1 Design Policy

The applicable criteria and elements of design of the Tawaeli-Toboli were decided based on the Bina Marga, AASHTO and Japanese design standards.

(1) Functional Classification of Highway

In Decree No. 13/1980 ON ROADS, the Primary Road Network System is defined as follows :

> "The primary road network is the responsibility of the national government and is the system of roads to assist the development of all regions by connecting centers for community services which are or which will become cities."

Thus, the law classifies roads into three categories, according to their functions, as follows :

- -Arterial Road: National roads connecting the provincial capital cities, serving primary transportation requiring long distance routes with high average speed.
- -Collector Road 1: National roads connecting with arterial roads, serving collection and distribution transportation requiring medium-distance route with medium average speed.
- -Collector Road 2: Provincial roads connecting kabupatens;, serving collection and distribution transportation requiring medium-distance route with medium average speed.
- -Collector Road 3: Provincial roads to be connected to arterial roads or other collector roads, serving collection and distribution transportation requiring relatively short distance trip, at medium average speed.

(2) Road Classification

Tawaeli-Toboli road is considered as a primary arterial road by the central government according to the road characteristics. Considering the present road conditions and alignment, however, the service level of the Tawaeili-Toboli road is very low compared with the other primary arterial roads. Design criteria should be reconsidered to be adjusted to future traffic demand, and service level should be upgraded to that of primary arterial road.

Table 5-5-1 shows the classification of road and application of the standard classes. Among the nine standard classes in the table, the classes with an asterisk such as CLASS 2*-- are the classes specially prepared for mountainous terrain. A design speed one grade lower than the classes of flat or rolling terrain is applied to the classes with an asterisk, but their cross sections are same as the classes with an asterisk.

Table 5-1-1 Classification of Road and Application of Standard

DTV (pcu/day) / Function	Terrain	>50000	50000>	>30000	30000> 10000	10000>	>10000	10000> 1000	1000>
ARTERIAL	F/R	CLASS 1	CLASS 2						
	M	CLASS 1*	CLASS 2*						
COLLECTOR	F/R			CLASS 3	CLASS 3	CLASS 4			
	M			CLASS 3*	CLASS 3*	CLASS 4*			
LOCAL	F/R						CLASS 3	CLASS 4	CLASS 5
	M						CLASS 3*	CLASS 4*	CLASS 5*

Source: Bina Marga

note : DTV = Design Traffic Volume M = mountainous terrain
F = flat terrain Minimum Design Speed for Collector 40 km/h
R = rolling terrain Minimum Design Speed for Local 20 km/h

(3) Design Speed

Design speed for the road is defined in Table 5-1-2. according to the Bina Marga design standards.

Table 5-1-2 Design Speed

	CLASS 1	CLASS 2 & CLASS 1*	CLASS 3	CLASS 4 & CLASS 3*	CLASS 5 & CLASS 4*	CLASS 5*
Design Speed (km/h)	80	60	50	40	30	20

Source: Bina Marga

A minimum design speed of 30 km/h is generally required despite low design traffic volume. Thus the design speeds of 80 km/h through 30 km/h are allocated to CLASS 1 to CLASS 5. To CLASS 5*, which is for steep terrain and quite low traffic volume. Where topographical or other elements hinder adoption of proper design speed, a design speed with one grade lower can be applied.

For improvement of the Tawaeli-Toboli road, the design speed was established based on the Bina Marga and Japanese design standard.

(km/h)

Service Level	Flat & Rolling	Mountain
Preferable	60	40
Acceptable	60	30
Minimal	60	20

Source: Study Team

(4) Road Traffic

According to traffic count survey carried out by the study team, traffic volume (ADT) of the Tawaeli-Toboli road is 886 vehicles/day. The study team also predicted the future traffic demand of the road. Table 7-5-3 summarizes the present and future traffic volume of the Tawaeli-Toboli road.

Table 5-1-3 Present and Future Traffic Volumes of Taweili-Toboli Road

Vehicle Type	Year 1997 (vehicle/day)	Year 2018 (vehicle/day)
Motorcycles	716	1,778
Passenger Cars	177	907
Buses	319	1,102
Trucks	390	1,841
Total	886	3,850

(Note: The total does not include the number of motorcycles)

Source: Study Team

(5) Road Traffic Capacity and Number of Lanes

In general a one-lane road (4.5 m) with shoulders 1.0 m x 2 has a capacity of 1,000 pcu/day to 2,000 pcu/day. A two-lane road (6.0 m) with shoulders 1.0 m x 2 has a capacity of 20,000 pcu/day. The traffic volume (pcu/day) of the Tawaeli-Toboli road is calculated as shown in Table 5-1-4 based on the existing and future traffic volume (vehicle/day), applying the conversion factor of 0.3 for motorcycles, 1.0 for passenger cars, 1.5 for buses and 3.0 for trucks.

Table 5-1-4 Traffic Volume (pcu/day) of Tawaeli-Toboli road

	Year 2003	Year 2018
Pcu/day	3,123	8,616

Source: Study Team

According to Table 5-1-4, the capacity of the existing one-lane road between Tawaeli-Toboli will be saturated at year 2003 and a two lane road could accommodate the future traffic volume of 8,616 pcu/day in the year 2018.

5.2 Geometric Design Standard for Tawaeli-Toboli Road

Based on the above study, geometric design standards was established for an alternative study as shown in Table 5-2-1.

Table 5-2-1 Geometric Design Standard

Terrain		Flat, Rolling	Mountain	Mountain	Mountain
Design Speed	Km/h	60	40	30	20
Lane Width	M	3.00 x 2	3.0 x 2	3.0 x 2	3.00 x 2
Shoulder Width	M	1.0	1.0	1.0	1.0
Crossfall of Pavement	%	2	2	2	2
Crossfall of Shoulder	%	4	4	4	4
Max. Superelevation	%	10	10	8	8
Min. Radius Curve	M	115	50	30	15
Min. Curve Length	M	100	70	50	40
Max. Gradient	%	5	7	8	9
Abs. Max. Gradient	%	9	11	12	13

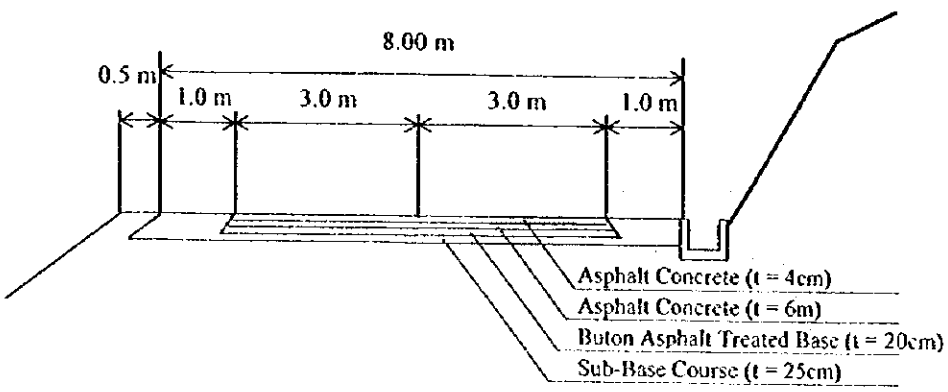
Source: Bina Marga

Other aspects are studied in Chapter 7 of this report.

5.3 Typical Cross Section

Typical cross section of a six(6) meter carriageway consisting of two lanes with one (1) meter shoulder on both sides applicable to the Tawaeli-Toboli road is shown below;

2-lanes



Source: Study Team

Typical cross section was employed for the whole section of Tawaeli-Toboli road.

5.4 Tunnel Design Standard

(1) General

Use of tunnel is one of the most effective means of the preventing slope failure and land slides, protecting existing environmental conditions, and securing proper road alignment within the mountain range. However, there are some disadvantages to be considered such as those indicated below:

- Construction cost is higher than that of roads and bridges
- Maintenance cost for ventilation system is needed
- Psychological problem due to closed space
- Potential for a secondary disaster following traffic accidents

At present, there is no design standard for road tunnels in Indonesia. For this study, there was a need to establish tunnel design standards for a two-lane traffic tunnel. "Spesifikasi Standar Untuk Perencan Geometric Jalan Luar Kota" and "Bridge Design Code" published by Bina Marga, and "Design Standard for Road Tunnel" and "Standard Specification for Tunnel" published by the Japan Road Association were employed for the establishment of tunnel standard for the study.

The tunnel engineering generally requires sufficient experience but there is no experience of the road tunnel construction in Indonesia at present. The following aspects, therefore, should be well considered at stages of detailed design and construction:

- Introduction of the tunnel engineering from foreign countries in which know how of the tunnel engineering for NATM has sufficiently been experienced.
- Procurement of the special equipment for tunnel construction such as drill jumbo, shotcrete machine.
- Detailed rock investigation: Structural elements of tunnel, especially the entrance and thickness of lining concrete, are extremely affected by rock conditions
- Management and operation to prevent the secondary accident since a traffic accident in the tunnel is apt to lead to the secondary accident.

(2) Interior Section and Construction Limit of Tunnel

The shape and dimensions of a tunnel should be determined based on the facilities required for the tunnel's interior and its stability.
Construction limit for tunnel is not mentioned in the "Specifikasi Standar Untuk Perencan Geometric Jalan Luar Kota", but, concerning roads, it is mentioned that the roadway should be at least 5.0 m in width. Also, concerning vertical clearance at Bridge Design Code mentions that parts of the superstructure or substructure of bridge crossing over a road or a railway should be at least 100 mm greater than the operation vertical clearance to allow for settlement and road resurfacing. Considering the above two items, a construction limit of 5.0 m for tunnels is applicable.
Shoulder width is selected to be 0.75 m, considering the Classes 3 to 4 of Bina Marga standards.

Shoulder height of tunnel is decided based on "A Policy of Geometric Design of Highway and Streets 1994". In this material, a trailer height of 4.1 m plus 0.1 m freeway is the minimum construction limit for shoulder space. Therefore 4.2 m is applicable for shoulder height.

Also, an inspection way with a width of 0.75 m should be provided on both sides of the traffic lanes.

Lighting and ventilation facility can be provided between construction clearance and interior area of tunnel.
Construction clearance of tunnel is shown in Chapter 7.

(3) Horizontal Alignment

Alignment standards should follow road alignment standards as the tunnel is a part of the road.
Since traffic accidents are prone to occur at the entrance of the tunnel, an application of higher standards for the tunnel entrances is desirable.
Curves with small radii are not applicable for this tunnel, as wider section needs to be designed in order to accommodate minimum sight distance, thereby increasing construction cost. Necessary considerations for design of tunnel are as mentioned below:

- Tunnel should be planned to be straight since drivers in Indonesia are unaccustomed to tunnels.
- Relation between design speed and minimum radius are shown below:

Design Speed	40 km/h	R=800 m
Design Speed	30 km/h	R=500 m
Design Speed	20 km/h	R=300 m

(4) Vertical Alignment

Tunnel gradient should be minimized for the following reasons:

- In consideration of the use of rail hauling in excavation, a tunnel gradient of less than 2 % is preferable.
- As exhaust density will rise in proportion to tunnel gradient, it should be less than 3 %.
- A steep grade will cause excess driving speed as well as gangerous passing maneuvers.

In consideration of the above, the following standards are established;

- 4 % gradient is applicable (the maximum grade that a truck can drive at half of design speed)
- Minimum gradient of tunnel is 0.3 % for drainage purpose.
- For the bypass route, maximum tunnel length of 3000 m and maximum gradient 3.5 % are applicable for upgrading of service level.

(5) Tunnel Cross Section

Tunnel cross-section should be decided based on soil conditions, excavation method, width of a traffic lane and type of tunnel support. Tunnel support are a vital part of tunnel structure, protecting the overall tunnel structure from failure of rock mass and earth pressure which constantly bears upon it. These tunnel support functions to stabilize the excavated section.

A tunnel cross section is discussed in Chapter 7.

5.5 Bridge Design Standard

(1) General

The design work of the proposed bridge structures was basically carried out in accordance with the "Bridge Design Code (Directorate General of Highways, Indonesia)" (hereinafter referred to as "Indonesian Bridge Design Code") as the prime design standards. Although the principal design concept is in accordance with the Indonesian Design Code, a bridge specification established by the American Association of State Highway and Transportation Officials (hereinafter referred to as "AASHTO") and a specification issued by the Japan Road Association as listed in (2) below will be applied as the need arises.

The structural calculation method for the bridge design basically follows the "Allowable Stress Design (working stress design) Method" in accordance with the Indonesian Bridge Design Code. However, prestressed concrete structures were designed to ensure their safety in the ultimate loading conditions prescribed in the code.

(2) Bridge Design Standard

1) Authorized Design Standards

The following standards will be applied for this study.

【The Republic of Indonesia】

- Bridge Design Code
  - Volume I (December, 1992)
  - Volume II (December, 1992)
- Standard Design of Bridge Superstructure (1993)
  (Reinforced concrete girder, prestressed concrete girder, composite girder)
- Standard Design of Box Culvert (1993)

【U.S.A】

[American Association of State Highway and Transportation Officials]

- Standard Specifications for Highway Bridges (Fifteenth Edition, 1992)

[American Concrete Institute](hereinafter referred to as "ACI")

- Building Code Requirements for Reinforced Concrete (ACI 318-83)

【Japan】
[Japan Road Association]
- Specifications for Highway Bridges (February, 1996)
  - Part I , Part II , Part III, Part IV, Part V

[Japan Highway Public Corporation]
- Design Standard for Highway and Bridges (February, 1994)
  - Part I , II, III, IV, V

2) Design Manuals

【The Republic of Indonesia】
[Directorate General of Highways]
- Bridge Design Manual

【Japan】
[Japan Road Association]
- Design Guideline for Concrete Highway Bridges (February, 1994)
- Construction Guideline for Concrete Highway Bridges (February, 1994)

(3) Loading Specifications
1) Bridge Loading Classification

Bridge design loadings to be applied are listed in Table 5-5-1 in accordance with the Indonesian Bridge Design Code. Design loadings in the code are grouped according to their origin into three groups and also classified by duration into two categories. In addition, an overstress is permitted in the basic working stress for some load combinations since these combinations have a low probability of occurrence and a short duration. These load combinations for working stress design are listed in Table 5-5-2 and the permitted overstresses is also given in Table 5-5-2 as a percentage of the allowable working stress.

Detailed application is referred to the Indonesian Bridge Design Code.

2) Application of Traffic Loads

Present traffic loads for design of road bridges consist of the "D" lane loading and the "T" truck loading. The "D" lane loading is applied across the full width of the bridge roadway and produces effects in the bridge equivalent to a queue of real of vehicles. The total amount of "D" lane loading applied depends upon the width of the bridge roadway.

The "T" truck loading is a single heavy vehicle with three axles which is applied in any position in a design truck lane. Each axlis comprised of two patch loadings which are intended to simulate the effects of the wheels of heavy vehicles. Only one "T" truck may be applied per design traffic lane.

(a) Design Traffic Lanes

Design traffic lanes are to be 2.75m wide. The maximum number of design traffic lanes to be used for various bridge widths is shown in Table 5-5-3.

Table 5-5-1 Summary of Design Actions

Design load		Duration	Group
Name	Symbol		
Self Weight	P_{MS}	Permanent	Permanent action
Superimposed dead load	P_{MA}	Permanent	Permanent action
Shrinkage & creep	P_{SR}	Permanent	Permanent action
Prestress	P_{PR}	Permanent	Permanent action
Earth pressure	P_{TA}	Permanent	Permanent action
Permanent construction	P_{PL}	Permanent	Permanent action
'D' lane load	T_{TD}	Transient	Traffic load
'T' truck load	T_{TT}	Transient	Traffic load
Breaking force	T_{TB}	Transient	Traffic load
Centrifugal force	T_{TR}	Transient	Traffic load
Pedestrian load	T_{TP}	Transient	Traffic load
Collision load	T_{TC}	Transient	Traffic load
Settlement	P_{ES}	Permanent	Environmental action
Temperature	T_{ET}	Transient	Environmental action
Stream/Debris	T_{EF}	Transient	Environmental action
Hydro/Buoyancy	T_{EU}	Transient	Environmental action
Wind	T_{EW}	Transient	Environmental action
Earthquake	T_{EQ}	Transient	Environmental action
Bearing friction	T_{BF}	Transient	Other action
Vibration	T_{VI}	Transient	Other action
Construction	T_{CL}	Transient	Other action

Source: Indonesia Bridge Design Code

Table 5-5-2 Load Combinations for Working Stress Design

Load combination	Combination No.						
	1	2	3	4	5	6	7
Permanent actions	O	O	O	O	O	O	O
Traffic loads	O	O	O	O			
Temperature effects		O					
Stream/Debris/Hydro/Buoyancy	O	O	O	O	O		
Wind load			O	O			
Earthquake effects					O		
Collision loads							O
Construction loads						O	
Permitted overstress	0%	25%	25%	40%	50%	30%	50%

Source: Bridge Design Code

Table 5-5-3 Number of Design Traffic Lanes

Bridge type	Bridge roadway width (m)	No. of design traffic lanes
Single lane	4.00 – 5.00	1
Two-way, no median	5.50 – 8.25	2
	11.30 – 15.00	4
Multiple-roadway	8.25 – 11.25	3
	11.30 – 15.00	4
	15.10 – 18.75	5
	18.80 – 22.50	6

Source: Bridge Design Code

(b) "D" Lane Loading

The "D" lane loading consists of a uniformly distributed load (UDL) combined with a knife-edge load (KEL) as shown in Fig. 5-5-1.

- Uniformly distributed Load: the UDL has an intensity q kPa, where the value of q depends on the total length L as follows:

$$L \leq 30\ m: \qquad q = 8.0\ kPa$$

$$L > 30\ m: \qquad q = 8.0\left(0.5 + \frac{15}{L}\right) kPa$$

- Knife-edge load: a single KEL of p kN/m will be placed in any position along the bridge. The KEL shall be applied perpendicular to the direction of traffic on the bridge. The value of p shall be 44.0 kN/m.
- The "D" lane loading will be arranged laterally in such a way as to produce the maximum effect. The lateral arrangements of UDL and KEL components of the "D" lane loading are to be the same. The concept of lateral distribution of "D" lane loading is shown in Fig. 5-5-2.

(c) "T" Truck Loading

The "T" truck loading consists of a tractor truck and semitrailer with axle weights and configuration as shown in Fig. 5-5-3. The weight from each axle is to be distributed equally between two uniformly loaded patches which represent the constant areas of the wheels. The spacing between the two heavy axles may vary from 4.0 m to 9.0 m in order to produce the maximum longitudinal effect.

(4) Seismic Design

Although a full dynamic analysis is required for large, complex and important bridges, equivalent static analysis is appropriate for proposed bridges in this study area.
Detailed description of seismic design can be referred to in the Indonesian Bridge Design Code.

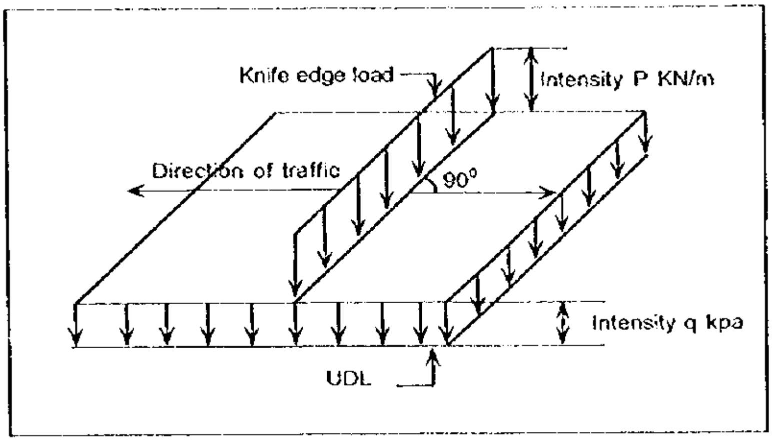


Figure 5-5-1 "D" Lane Loading

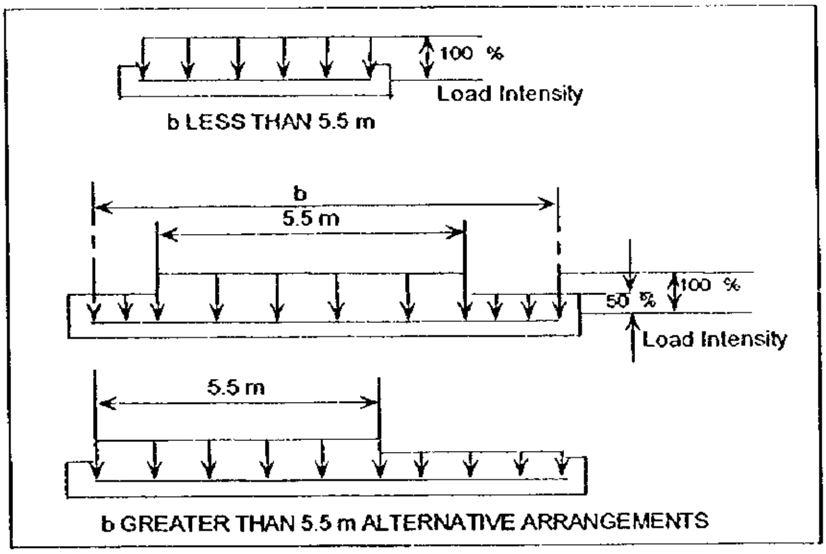


Figure 5-5-2 Lateral Distribution of "D" Lane Loading

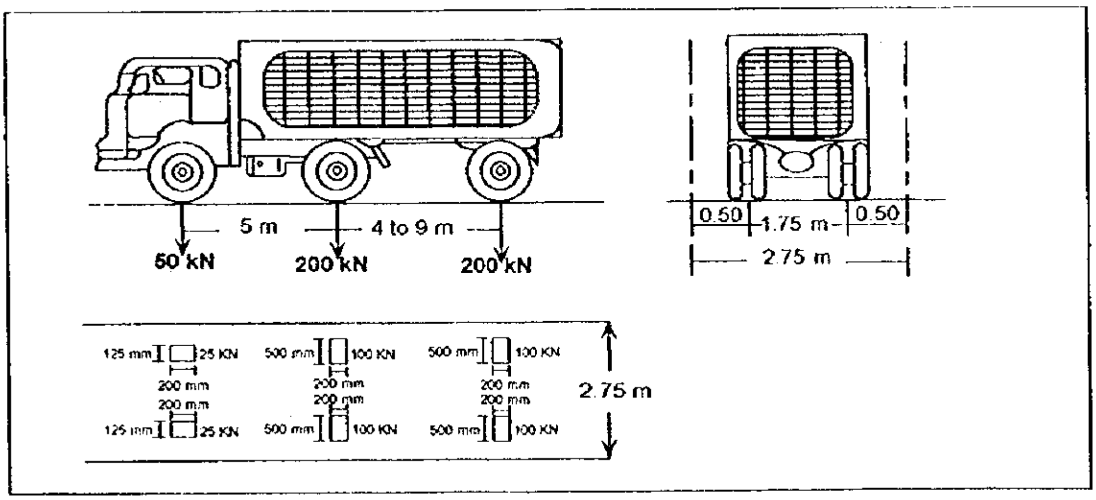


Figure 5-5-3 "T" Truck Loading

Source: Indonesian Bridge Design Code

5.6 Disaster Prevention Design standard

(1) General

Traffic is very frequently interrupted by road damage resulting from heavy rains and earthquakes in Central and Southeast Sulawesi(the Tawaeli-Toboli road has been closed for two weeks per year on the average and even in normal conditions, traffic is allowed to move a single direction [at intervals of four hours] at a time). In particular, traffic disturbances are largely caused by the failure of slopes. The stability of a slope is maintained mainly by a balance between the bearing strength of the ground and the sliding force resulting from slope gravity. However, the stability of a slope is greatly disturbed by following:

- Decreases in the strength of the ground due to groundwater seepage or rainfall;
- Changes in the balance of gravity due to artificial cuts;
- Increase of the pore pressure due to heavy rainfall or movement of groundwater; and
- Increases in gravity acceleration during earthquakes.

Prior surveys and measurements are of great importance as slope failures, etc., are determined by geological conditions (which occur within the study area) and are predictable by observing local topography. Many causes of failures exist; in some cases a combination of causes can result in a failure, making the prediction of the failure's location, scale, etc., more difficult in comparison with predicting landslides.

The following points should be kept in mind in choosing appropriate methods of slope protection:

- Sodding or vegetation is the preferred method in view of cost and aesthetic appearance.
- Slope protection works using structures are employed as an alternative where sodding or vegetation is difficult to perform due to meteorological, topographical, gradient conditions or presence of spring water.
- Areas where landslides are likely to occur should be avoided in route selection
- However, appropriate countermeasure work will become necessary if road construction in such areas is unavoidable.
- Permanent drainage facilities as well as temporary drainage facilities for the construction period should be very carefully planned.

(2) Surface Drainage

Surface drainage such as crest, berm, vertical, and toe ditches are designed in accordance with the following:

Run-off rain is calculated according to the Rational Formula using a two-year design rainfall probability period.

$$Q = \frac{1}{3.6 \times 10^6} \times C \times I \times A$$

Where,

Q = Run-off (cubic meter/sec)
C = Run-off coefficient
i = Rainfall intensity within time of concentration (mm/h)
A = Catchment area (sq. meters)

The following values are used for the run-off coefficient in the above formula:

Paved road -- 0.80
Road shoulders, man-made slopes---------- 0.70
Hilly areas with steep gradients --------------- 0.50
Hilly areas with moderate gradients ---------- 0.30

Average run-off speed is obtained by the Manning Formula:

$$V = \frac{1}{n} \times R^{\frac{2}{3}} \times I^{\frac{1}{2}}$$

Where,

V = Average run-off speed (m/sec)
R = A/P: Hydraulic radius (m)
A : Cross-sectional area of water flow
P : Length of wetted perimeter
i = Hydraulic gradient
n = Roughness coefficient ($sec/m^{1/3}$)

A roughness coefficient of 0.02 is applied to ditches made of rough stone or wet stone masonry in the above formula.

Although crest and berm ditches are expected to be cleaned during maintenance work, their actual cross sections are 20 percent greater than the calculated cross sections in order to provide a margin of safety.

(3) Earth Work

1) Recutting

Unstable slopes with steep gradients should be recut to achieve a stable gradient. This work was applied in the Study for the two following cases:

- deeply eroded slopes.
- slopes adjacent to a slope damaged by a landslide.

Cut slopes are usually provided with surface drainage and berm. The goals of a berm are (1) to moderate the average gradient of a slope and (2) to reduce the velocity of surface water running down the slope, thus preventing erosion and scouring. In general, berms 1.0m to 2.0m in width are constructed at 5.0m to 10.0m intervals. When a slope consists of different layers, it is desirable to provide berms at the borders of the different layers.

In the Study, recutting was designed by the following standards.

Table 5-6-1 Re-cutting Standards

	Type of Rock		
	Earth	Soft Rock	Hard Rock
Gradient	1.0:1	0.5:1	0.25:1
Berm Interval	Every 5m	Every 5-7m	Every 7m
Berm Width	1.5m	1.0m	1.0m

Source: Study Team

2) Refilling and compaction

Some slopes can be damaged by a single huge gully, despite the rest of the surface being sound. In the Study, refilling was applied to repair such erosion.

A slope should be restored to its original condition by refilling with compaction. Refilling material should be either soil or soil cement. Generally, refilled slopes collapse due to insufficient compaction. In this case, the following points should be kept in mind:

- The thickness of one layer should be less than 50cm,
- Compaction should be carried out for each layer by a compaction, and
- A completed slope should be compacted by slope tamping.

After refilling, a slope surface should be protected from further erosion by surface drainage and vegetation.

(4) Hydro-seeding

Hydroseeding is a method that sprays a mud-like mixture composed of seeds, fertilizer, and soil onto a slope, using a pump gun or air compressor. Specifications for hydroseeding applied in the study are as follows:

- The quantity of soil to be used should be 0.01 m3/m2

- Since hydroseeding is not common in Indonesia, the type of grass that is suitable in Sulawesi is not well known. It will be necessary in the future to determine the type of grass most appropriate for local conditions by making trial sprays.
- Asphalt emulsions should be used for film curing and sprayed at a ratio of 1 liter/m^2

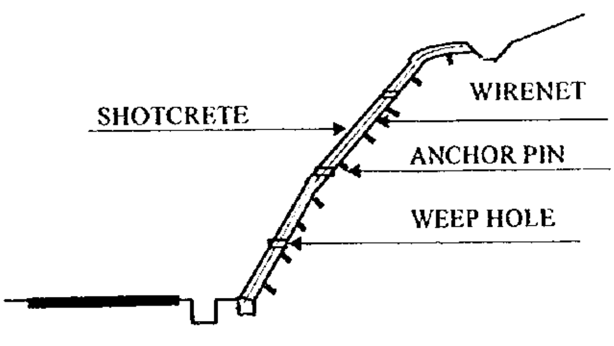
(5) Shotcrete

In shotcrete, mortar or concrete is sprayed onto slopes of highly weathered or easily weathered rock using spray guns, in order to prevent further weathering, erosion, and scouring of a slope surface due to surface water flows. In general, the method is not applied to earthen slopes or slopes having spring water, since adhesion with slope surface is poor and separation may occur.

The thickness of shotcrete is determined based on the slope gradient, degree of weathering, cracking conditions, etc., but the standard thickness is 5 to 10cm for mortar and 10 to 15cm for concrete. The specifications for shotcrete applied in the Study are as follows:

- The concrete mixture is 1:3:2 (cement, sand and gravel), and the ratio of water to cement is 45 percent.
- Shotcrete that is 15cm thick is applied with a reinforced steel net to slopes with loose rock of a fairly large size, and to slopes that are expected to have large volumes of surface water. For other slopes shotcrete 10cm thick with a wire net is applied.
- Weep holes are provided every 2 square meters, which is the maximum standard interval.

An illustration of shotcrete is shown in Figure 5-6-1



Source: Study Team

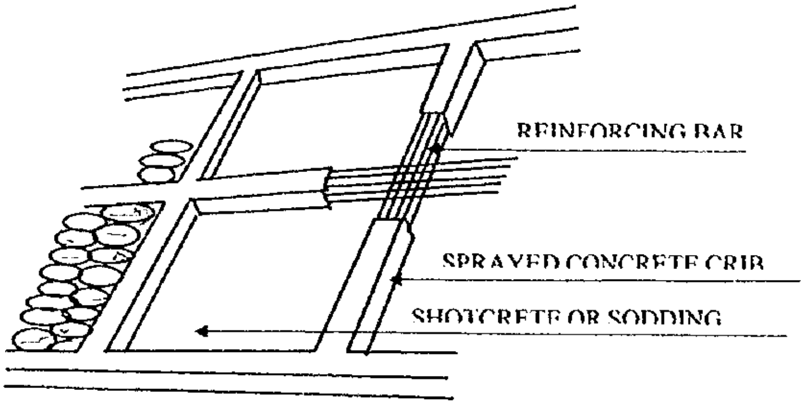
Figure 5-6-1 Shotcrete

(6) Cribwork

There are three basic crib types; cast-in-place concrete cribs, sprayed concrete cribs, and pew-cast concrete cribs. In the study, sprayed concrete cribs are applied because of the following advantages:

- No forms are needed to pour the concrete of the cribs,
- Since the configuration of crib is adaptable to a slopes surface, undulations on the slope surface are allowable, and
- On-site work is simpler than that of the other two methods.

An illustration of a crib is shown in Figure 5-6-2



Source: Study Team

Figure 5-6-2 Sprayed Concrete Crib

(7) Retaining Wall

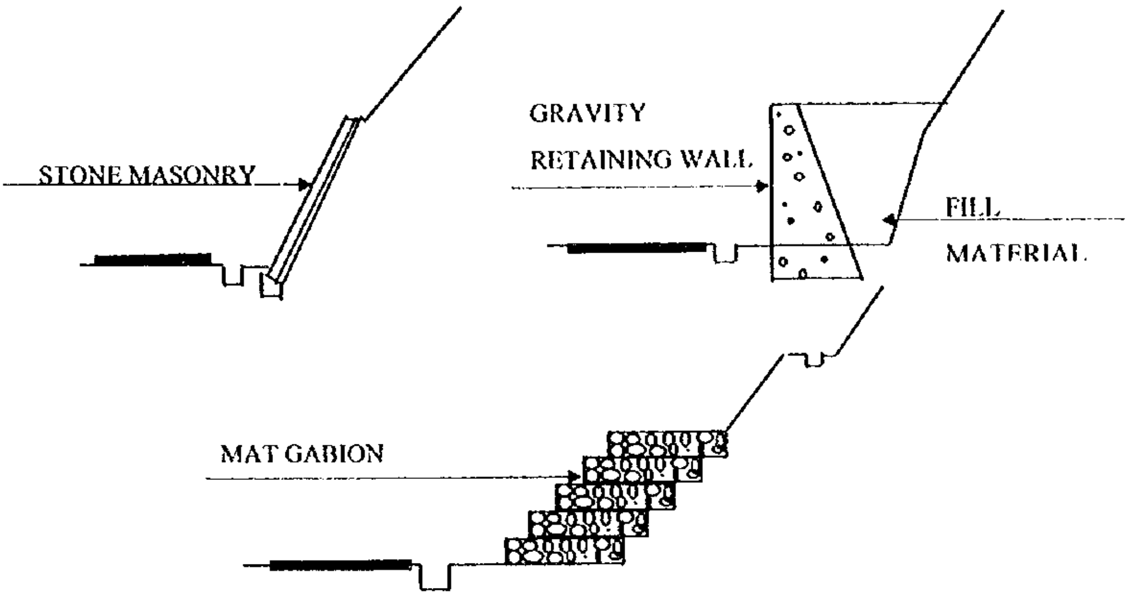
The types of retaining wall adopted in the study are stone masonry, gravity-type, and gabion retaining walls. They are illustrated in Figure 5-6-2 with their applicable heights. Design standards for the stabilization of retaining walls are shown in the Table 5-6-2.

Table 5-6-2 Design Conditions

Safety factor for sliding	>1.5
Stability for overturning	e<B/3
Safety factor for bearing capacity	>3.0

Note: e = eccentric distance
B = width of retaining wall

Source: Study Team



Source: Study Team

Figure 5-6-3 Retaining Wall

(8) Rockfall Prevention

In the study, barriers for debris catch basins and anchor wire nets are applied as road restoration measures.

1) Barrier for debris catch basin

This type of measure can be applied at road sections where there is room between the foot of the slope and the road shoulder. This space is for a barrier to serve as a debris catch basin.

In general, the barrier is a gravity-type retaining wall. It is designed based on the concept that the kinetic energy of falling rocks changes into transformation energy of the barriers and the bearing layer. Accordingly, the dimensions of the barrier depend on the weight and the bouncing height of the falling rock.

In order to enhance the capacity of the barrier, a catch fence at the top of the retaining wall can be jointly attached. An illustration of a rockfall barrier with a catch fence is shown in Figure 5-6-4

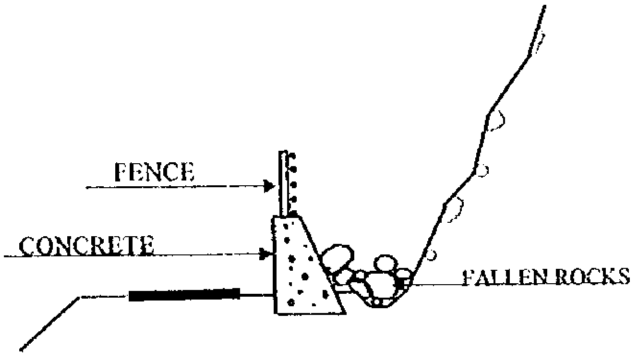


Figure 5-6-4 Rockfall Barrier

2) Anchor Wire Net

This net aims to prevent rocks from falling full force by covering a slope with a net. There are two kinds of net: the first is made of chemical fiber and the other is a steel net. The chemical fiber net is used for small falling rocks weighting less than 60kgf. The durability of this net is uncertain when exposed to sunlight. For this reason, the steel net was adopted for the purposes of the study.

An illustration of an anchor wire net is shown in Figure 5-6-5

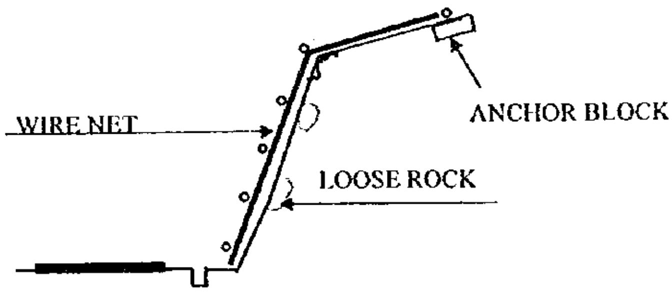


Figure 5-6-5 Anchor Wire Net

(9) Slope Stabilization after a Landslide

Three measures were taken to stabilize slopes at the site of landslides in the Study: (1) removal of unstable material, (2) use of counterweights, and (3) dewatering of groundwater.

As for types of landslides, there are generally three : (1) the rotational landslide, (2) the two-dimensional landslide, and (3) the three-dimensional landslide.

1) Slope stabilization

In all the cases mentioned above, sliding should be examined to calculate the safety factor as follows:

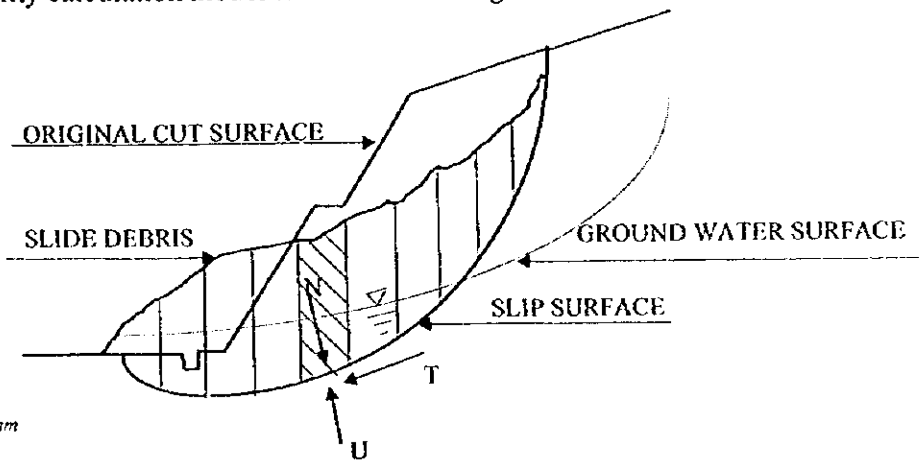
- Assume a sliding plane based on the results of boring and the configuration of slide debris.
- Assuming a present safety factor of 1.0, apply the linear equation below containing cohesion (c) and the angle of internal friction (ϕ).

$$Fs = \frac{\tan\phi \times \Sigma(N - U) + c \times \Sigma L}{\Sigma T}$$

Where,

Fs = Safety factor (assumed to be 1.0)
T = Shearing stress of slice on sliding surface (t/m2)
N = Vertical stress of slice on sliding surface (t/m^2)
U = Pore water pressure on sliding surface (t/m^2)
c = Cohesion (t/m^2)
ϕ = Angle of internal friction (°)
L = Sliced length of arc along sliding surface (m)

The stability calculation model is illustrated in Figure 5-6-6.



Source: Study Team

Figure 5-6-6 Stabilization Calculation Model

In case the counterweight method is taken up as a restoration measure, the safety for soliding should be calculated by the following equation.

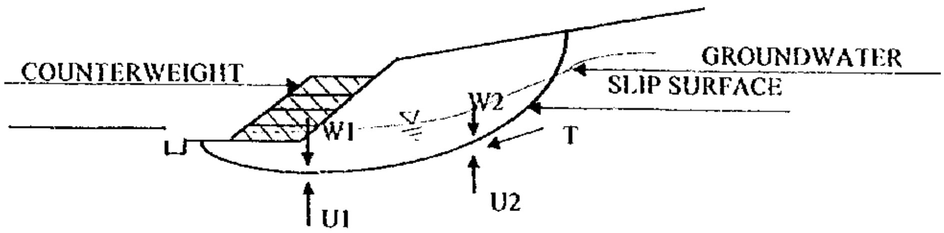
$$(Wc - Uc) \times \tan\phi \geq \frac{\tan\phi \times \Sigma(Ws - Us) + c \times \Sigma L}{\Sigma T}$$

Where,

Wc = Weight of counterweight (t)
Ws = Weight of slide debris (t)

Uc	=	Buoyancy of counterweight (t)
c	=	Cohesion (t)
ϕ	=	Angle of internal friction (°)
L	=	Length of sliding area surface (m)
T	=	Shearing stress of slice on sliding surface (t/m^2)

The stability calculation model is illustrated in Figure 5-6-7.



Source: Study Team

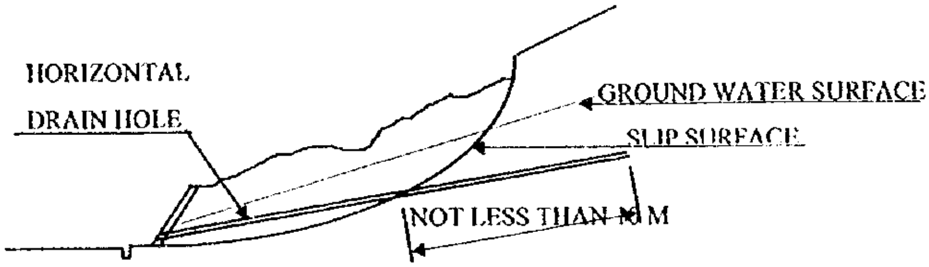
Figure 5-6-7 Stabilization Calculation Model for the Counterweight Method

2) Dewatering of groundwater

Dewatering of groundwater is one of the most effective measures against landslides. In the study, horizontal drain holes with the following specifications were selected as a countermeasure:

- A drain hole 66mm in diameter is made by boring,
- A hard polyvinyl chloride pipe is inserted into the bored hole to sustain the hole and to collect and drain the water,
- boring is conducted at an upward angle of 5 degrees, and installed about 10 meters into the water holding layer outside of the sliding plane,
- drain holes are made at intervals of 5 meters along the sliding plane.

An illustration of horizontal drain holes is shown in Figure 5-6-8.



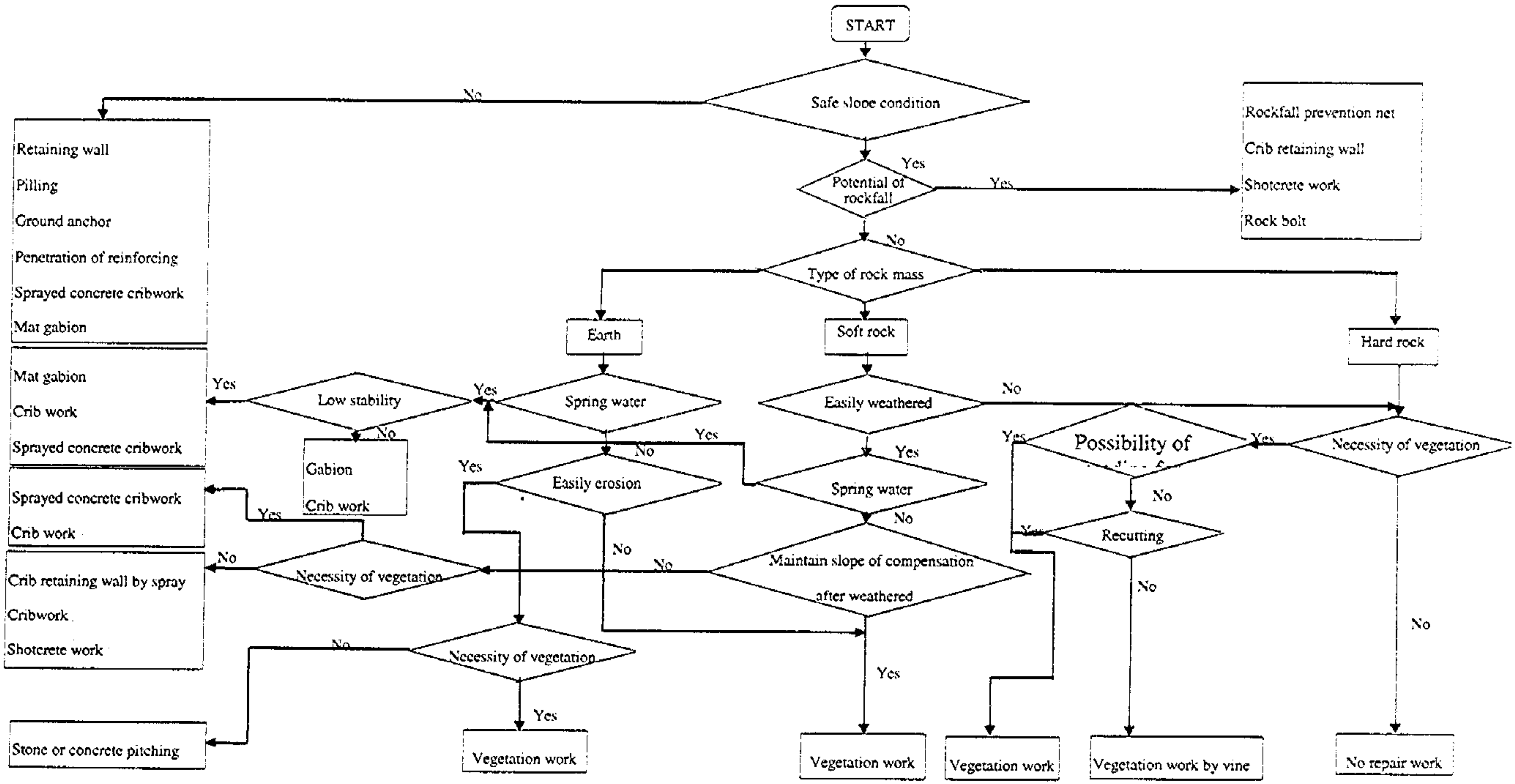
Source: Study Team

Figure 5-6-8 Horizontal Drain Holes

(9) Selection of Disaster Prevention for Slope

1) Cut Slope

Appropriate countermeasure work will be selected in following flow chart as shown on the following page.



Source: Study Team

Figure 5-6-9 Selection of Disaster Prevention Method for Slopes

5.7 Pavement Design Standard

Pavement design should be consider the following conditions:

- There are two types of design: that of flexible pavement and rigid (concrete) pavement;
- Two categories of road construction are involved in the project: widening or overlay of existing pavement, and new construction of bypass road;
- In the section of pavement type, investment efficiency sometimes should be considered in addition to the construction cost; and
- Construction aspects and local conditions sometimes govern the selection of pavement type when the reconstruction/adjustment of related roads is necessary.

(1) Method of Design

AASHTO Guide for Design of Pavement Structure by AASHTO was used for the design.

(2) Design CBR

Design CBR of 5% to 6 % was used based on the results of the CBR Test conducted by the study team.

(3) Design Life Period

The design life period is to be 20 years.

(4) Design Traffic Volume for the Pavement

Design traffic volume for the pavement design is to be for the period from 2002 to 2022, a period of 20 years.

Pavement structure is important in permitting smooth traffic and drainage. The present pavement situation of Tawaeli-Toboli is classified from poor to very poor, indicating that the existing pavement level is not adequate. Also, considering existing conditions, it can not be expected to secure proper strength of base layer can not be guaranteed in view of poor workmanship.

To cope with the above conditions, pavement structure which is explained in Chapter 7 was decided based on the above standards.

Chapter 6
Selection of Route

Chapter 6 SELECTION OF ROUTE

6.1 Field Reconnaissance Survey of Tawaeli-Toboli Road

(1) An improvement project of Tawaeli-Toboli road is being carried out by the local contractor under Dinas PU, funded by ADB. This project will be completed by June 1999.

(2) Excess excavated materials of this project are disposed directly into the valley side with no consideration to the environment. Also, cutting surface is apparently in critical condition due to soil characteristics and steep slope.

(3) Two existing bridges are being replaced. A steel truss bridge will be constructed in place of an old steel bridge. This construction component is a bridge project, not included in the project mentioned above.

(4) In most sections, the existing pavement is in unsatisfactory condition.

(5) There are limited safety facilities, such as guard rails, but no curve mirrors, safety posts. Road alignment is also poor.

(6) Existing road width of some sections is too narrow for buses and trucks.

(7) There is a fault crossing the road at 16km+500 m. Counter disaster facilities are urgently needed.

(8) There is no side drainage system for the road.

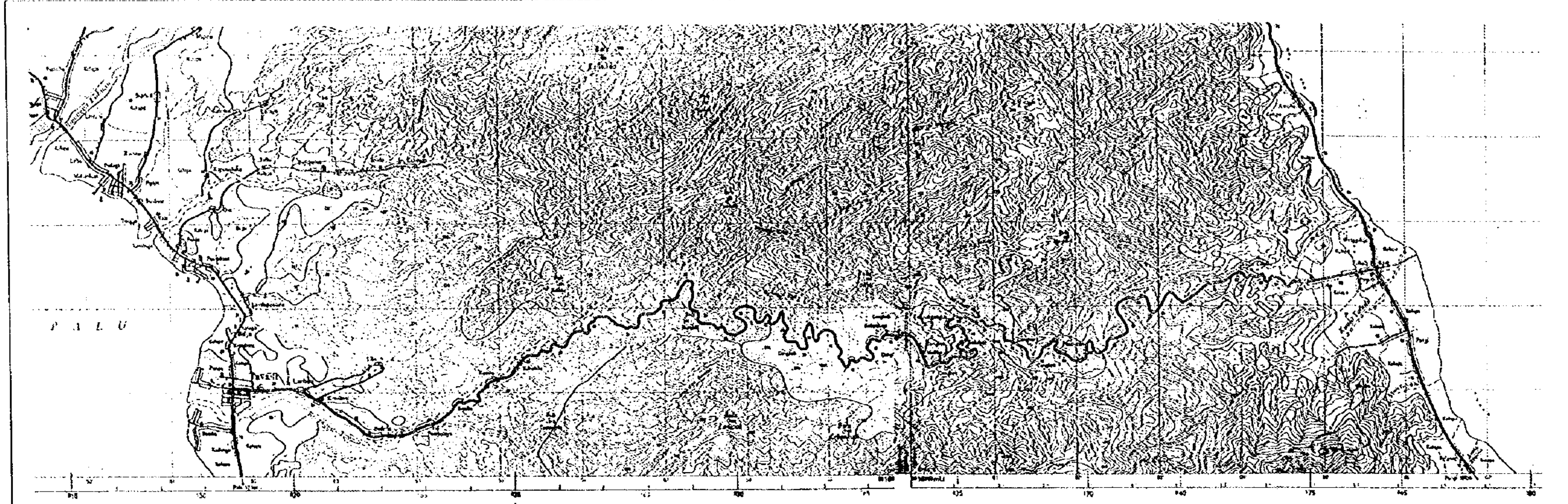
(9) Most of the cut slopes need permanent facilities for slope protection.

(10) Tawaeli-Toboli road had been controlled by traffic regulations as shown in table below (1997-1998).

Hour	10-12	12-14	14-16	16-18	18-20	20-22	22-24	00-02	02-04	04-06	06-08	08-10
From Toboli	O	X	X	X	O	X	X	X	O	X	X	X
From Tawaeli	X	X	O	X	X	X	O	X	X	X	O	X

Source: Bina Marga

Existing road conditions of the Tawaeli-Toboli road are as shown in Figure 6-1-1.



Item		Segments (from 0.000 km to 48.000 km)	Remarks
Road Length (km)		0.000km; 5.000km; 10.000km; 15.000km; 20.000km; 25.000km; 30.000km; 35.000km; 40.000km; 45.000km; 48.000km	Remarks
Terrain		Flat; Rolling; Mountainous; Flat	
Existing Road Condition	Surface	Asphalt Concrete Pavement	
	Condition	Fair Condition; V/P; Poor; Fair; V/P; Fair; Poor; V/P; Poor; V/P; Fair; Poor; V/P; Fair; V/P; Poor; Very Poor; Poor; V/P; Poor; Fair; Fair to Good condition	V/P =Very Poor
	Width	4.5~5.0 m; 4.0~4.5 m; 3.6~4.0 m; 4.0~4.5 m; 4.0~4.7 m; 3.0~4.0 m; 4.0~5.3 m	
	L Shoulder	1.5~2.0 m; 1.0~2.0 m; 0.5 ~ 1.5 m; 1.0~3.0 m; 2.0~5.0 m; 0.5 ~ 3.0m; 1.0~2.0 m	
	R Shoulder	1.5~1.8 m; 1.0~6.0m; 0.5~1.5m; 0.5~1.8 m; 0.5~3.0 m	
Slope Condition	Cut	None; Unstable; Stable; Unstable; S; U; Stable; Unstable; Stable; None	S =Stable U =Unstable
	Embankment	None	
Bridge	Type (span)	Steel (57 m); Concrete (5 m); Steel (48 m); Concrete (5 m); (5 m); (5 m); (6 m); (5 m); (5 m); (6 m); (14 m); (7 m); (6 m); (6 m); (6 m); (6 m); (6 m); (6 m); (8 m); (6 m); (6 m); (6 m); Concrete (6 m)	
	Station No	4+460; 4+765; 5+655; 7+288; 7+855; 8+180; 13+232; 13+500; 18+383; 20+445; 23+095; 26+138; 28+055; 28+670; 31+825; 33+815; 33+950; 34+465; 39+550; 41+210; 41+830; 41+950; 42+840	

Figure 6-1-1 Existing Conditions of Tawaeli-Toboli Road

6.2 Salient Features of Alternative Routes

As mentioned above, the existing road is obviously in dangerous condition. To cope with this situation, alternative routes were studied based on the following policy.

- To avoid large cutting slope prone to landslides and slope failures;
- To provide counter disaster facilities where there are cut-and-fill slopes;
- To sustain living environment of local inhabitants;
- To secure dual-lane traffic throughout the year:
- To sustain environmental conditions; and
- To up grade service level.

Based on the above policy, four alternative routes were selected as described below considering the design standards of Chapter 5:

Alternative A: This is an improvement of the existing road. Most of the existing alignments are maintained but sharp curves are improved to the design speed 20 km/h of which design criteria is the same as Bina Marga's design in 1995, using a topographic map of 1:1000 scale. This alternative includes all necessary disaster prevention work for cut-and-fill slopes, pavement widening to 6.0 m, drainage system, bridges and traffic safety facilities.

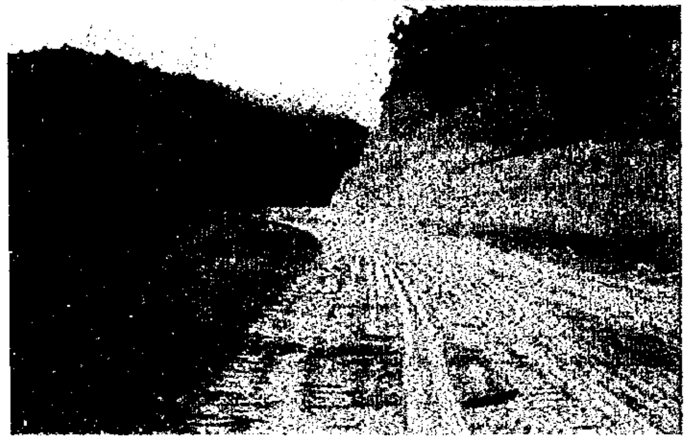
Alternative B: This alternative improves the existing sharp curves into design speed of 30 km/h as minimum requirement and provides a tunnel of 650 m in length shortening the route by 3.4 km while the other existing road alignments are maintained.

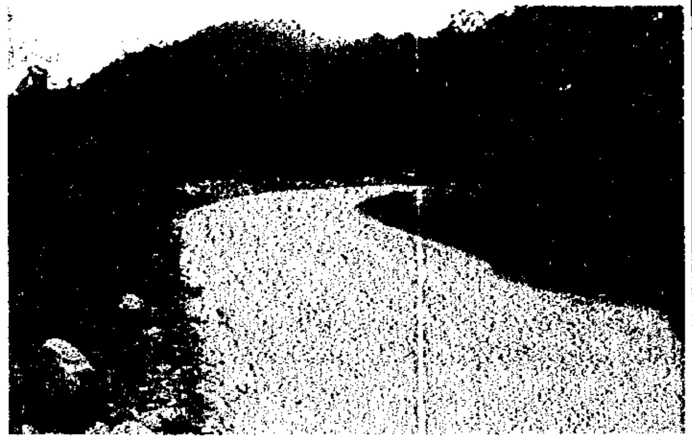
Alternative C: This alternative employs a new alignment of 12.4 km in length on the southern side instead of the existing alignment from 8 km+500 m to 22 km + 0 m, while it maintains the same alignment with Alternative B after the location of 22 km, including a tunnel of 650 m in length.

Alternative D: This alternative fully employs a new alignment on the northern side beyond the gorge on the mountain side of the existing alignment, excepting the existing sections in the lowland areas. This has a preferable design speed of 40 km/h with 5 tunnels with a total of 5350 m in length and with the shortest route length of 35.3 km.

The route of each alternative is shown in Figure 6-2-1, using a topographic map on a scale of 1: 50,000.

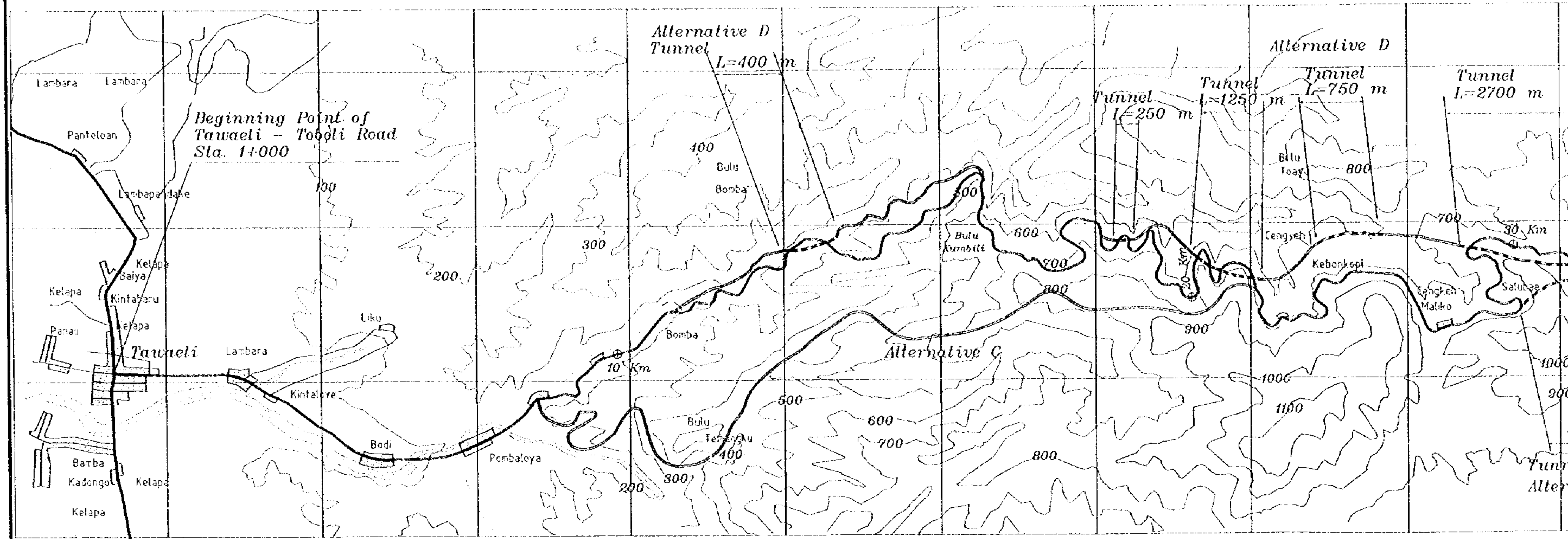
A geometric design criterion for each alternative is described in Table 7-3-1.


7 km


21 km


29 km


40 km

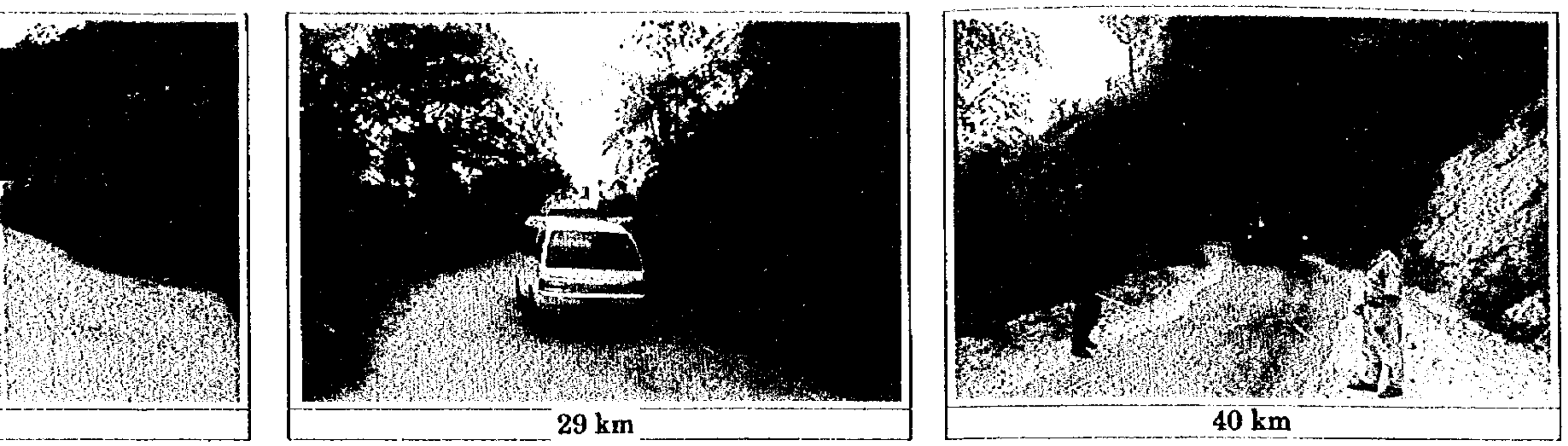



9 km


27.5 km

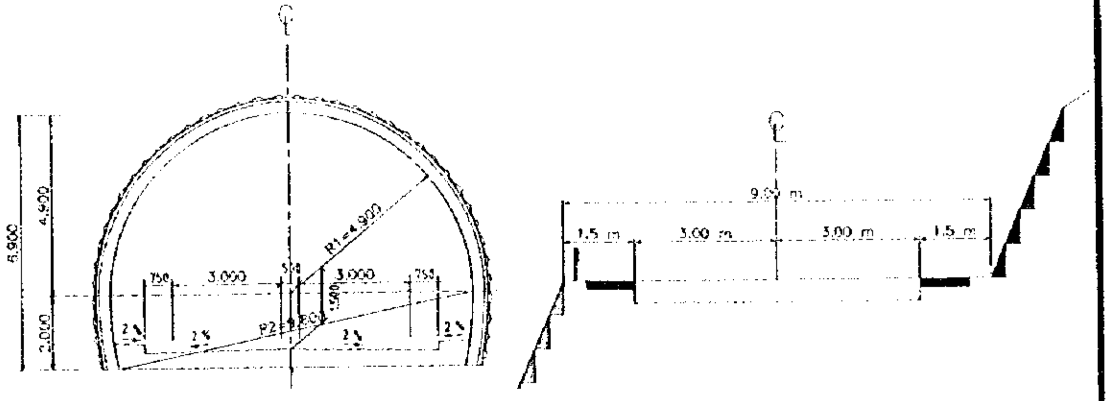

36 km


Construction Exam

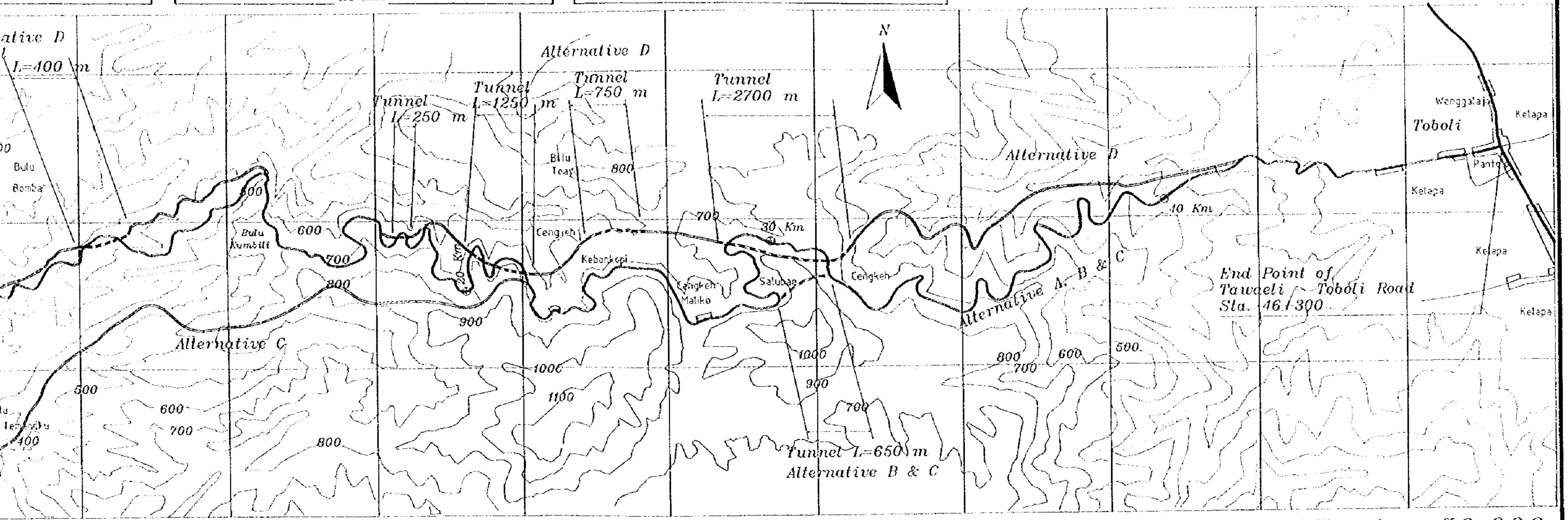


29 km

40 km



Typical Tunnel Section and Cross Section



SCALE 1 : 50.000

0 km 1 km 2 km 4 km

36 km

Construction Example

Alternative	Length Km
A (Red)	45.30
B (Red)	42.35
C (Blue)	41.35
D (Black)	33.42

Figure 6-2-1 Location of Alternative Route

6.3 Comparison of Alternate Routes and Recommendations

(1) Comparison of Alternative Routes

Figure 6-3-2 shows the comparison of alternative routes.
The characteristics of each route are as follows:

Alternative A: This is an improvement of the existing road. There is difficulty in improving vertical alignment due to the steep terrain; therefore, sight distance should be maintained. Minimum radius is 15 m with a design speed of 20 km/h.

Alternative B: Minimum curve radius of 30 m was applied with design speed of 30 km/h. Tunnel is provided to avoid a landslide area. Bridges are added in conjunction with improvement of road alignment. Other improvements are the same as alternative A.

Alternative C: A bypass is planned to provide lower cost of disaster prevention facilities and less traffic impact during the construction of the bypass. Other improvements are the same as alternative B except for the bypass.

Alternative D: Provides a preferable service level to design speed of 40 km/h to satisfy the road criteria with 5 tunnels, bridges and 2 bypasses.

Project cost of each alternative is shown in Figure 6-3-1.

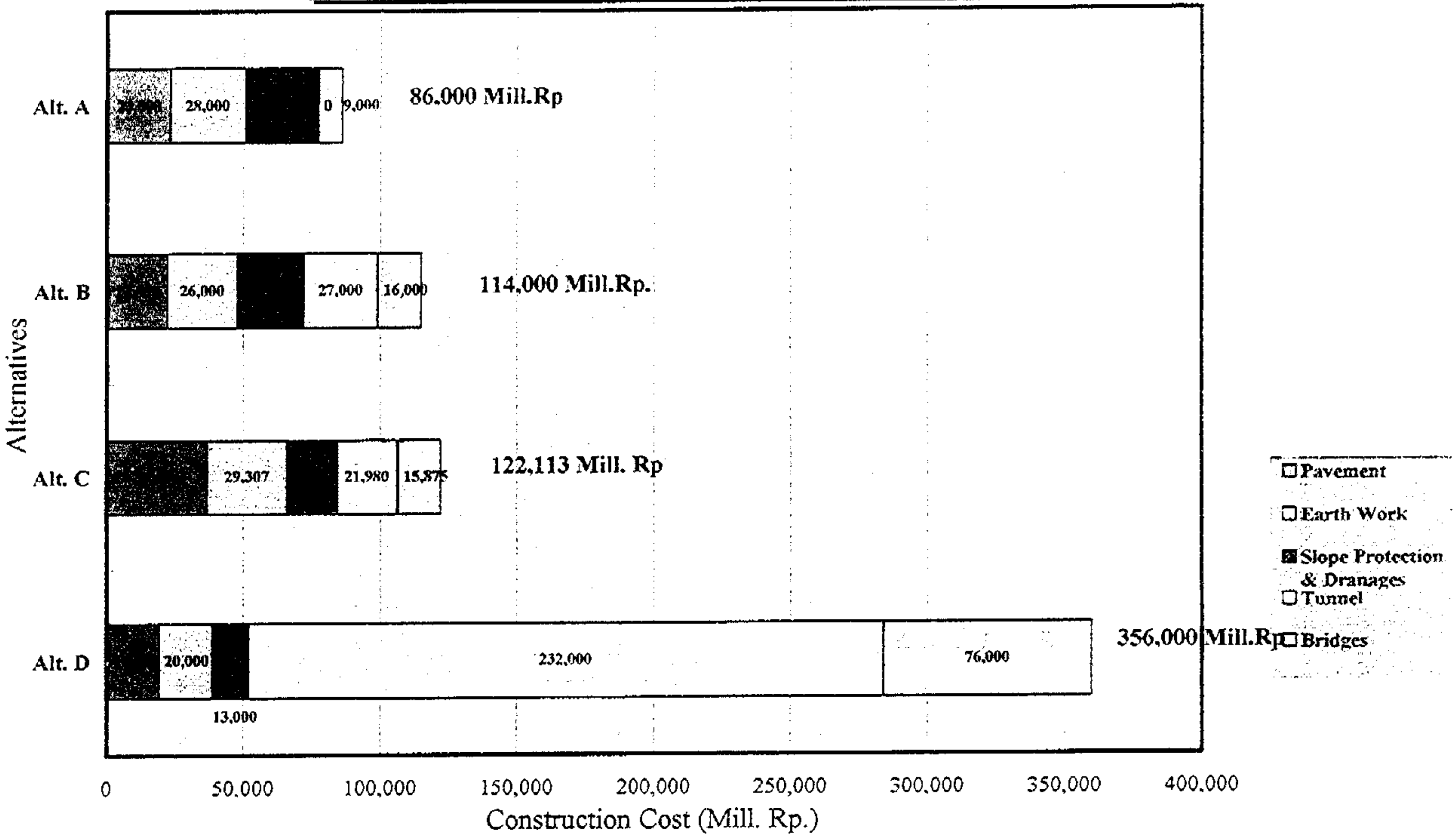
(2) Recommendations

Alternative A has the lowest construction cost but the design speed of 20 km/h is not acceptable and the route length of 45.3 km is longer than the other alternatives. Alternative D has a preferable design speed of 40 km/h and the shortest route length, but the cost of Rp 356 billion is too great to attain economic viability.
Alternative C has the following advantage and disadvantages points compared with Alternative B:

- The route is shorter than Alternative B by 1.5 km, avoiding critical disaster-prone areas.
- This is less impact on traffic in Alternative C during construction as the existing section for improvement of Alternative C is 28.82 km, which is shorter than that of Alternative B by 13.5 km.
- The construction cost of alternative C is higher than that of Alternative B by only 7%.

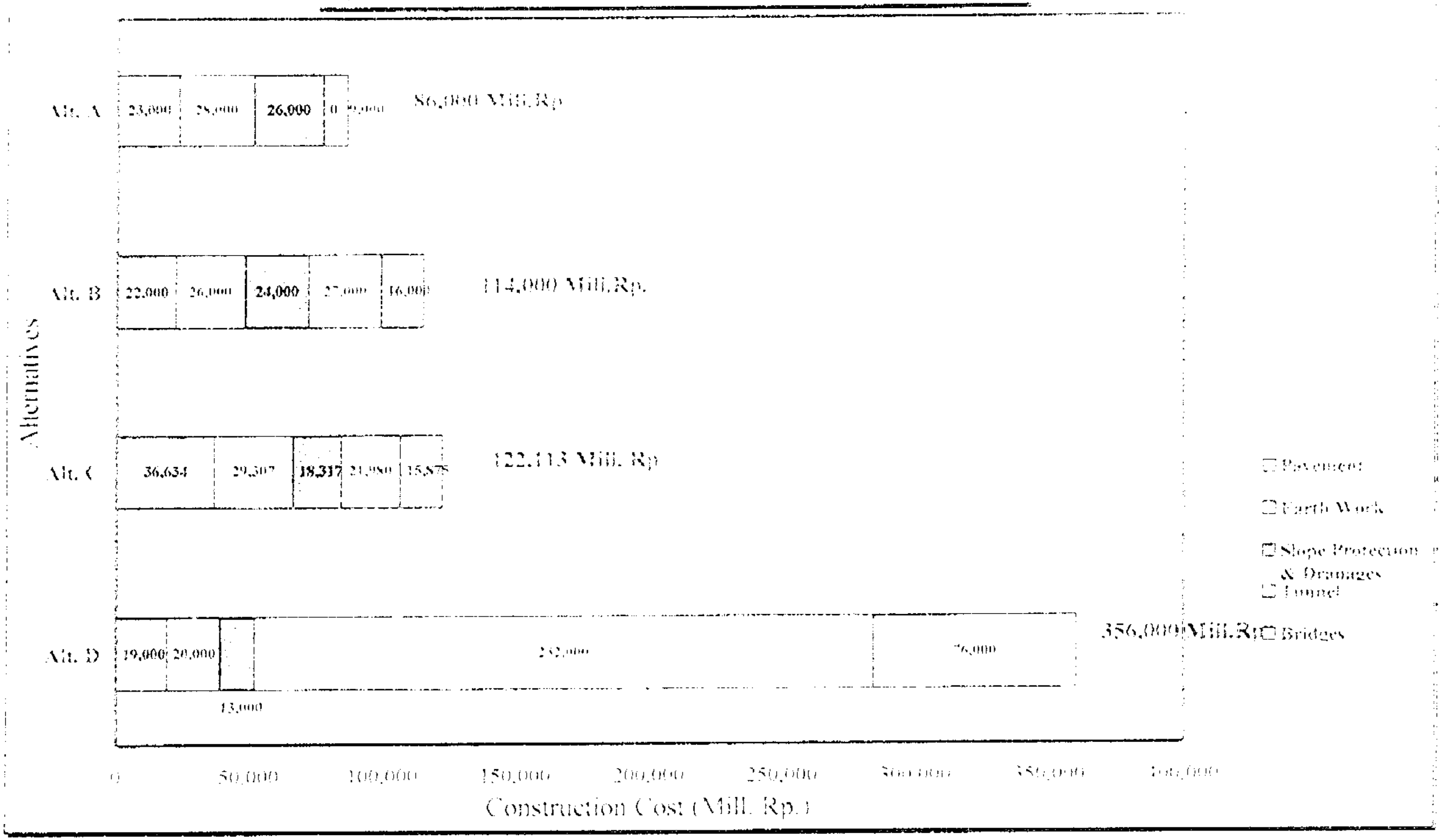
Alternative C was recommended based on the above comparisons although it has some disadvantagement.

Figure 6-3-1Total Construction Cost For Tawaeli - Toboli Road

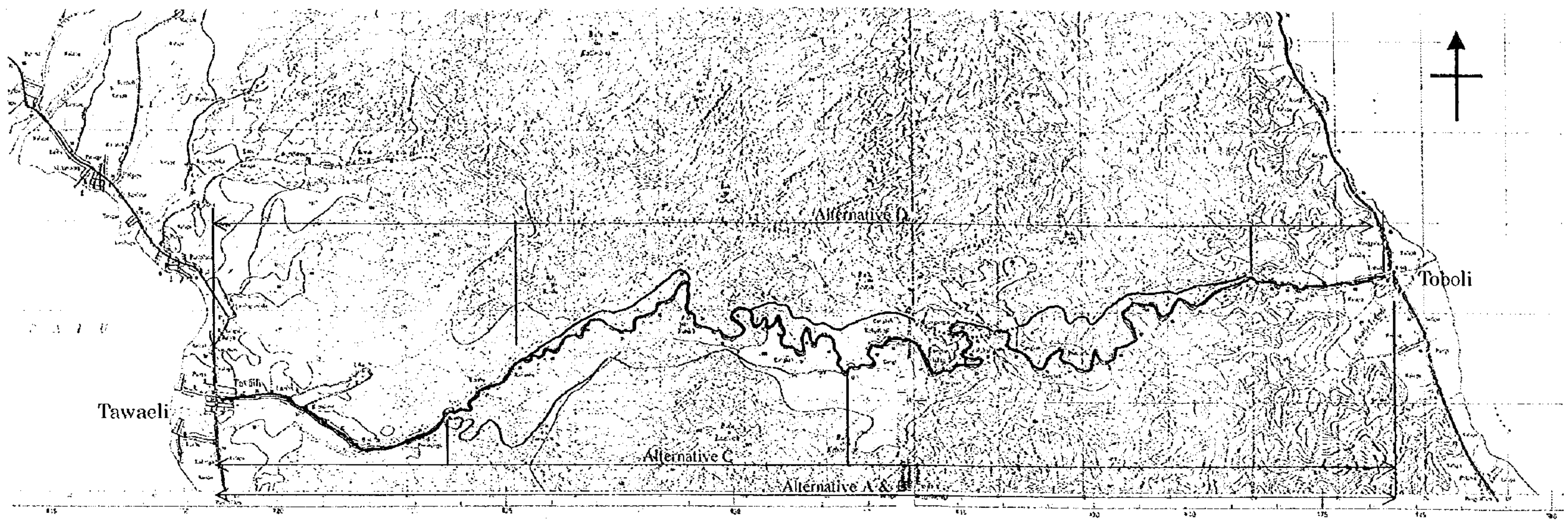


(The prices based on July 1997)

Figure 6-3-1 Total Construction Cost For Tawaeli - Toboli Road



The prices based on July 199[illegible]



	Alternative A (Improvement of Existing Road)	Alternative B (Improvement of Existing Road with Tunnel)	Alternative C (Construction of New Bypass (South))	Alternative D (Construction of New Bypass (North))
Aims / Purpose of the Route	This is an improvement of existing road. Most of the existing alignments are maintained but sharp curves are improved to design speed of 20 km/h of which design criteria is the same as Bina Marga's design of 1995, using topographic maps of scale 1:1000. This alternative includes all necessary disaster prevention works for cut and fill slopes, pavement widening of 6.0 m, drainage system, bridges and traffic safety facilities.	This alternative improves the existing sharp curves into design speed of 30 km/h as minimum requirement and provides 650 m-long tunnel as a shortcut of the existing road by 3.4 km long while the other existing road alignments are maintained.	This alternative employs a new alignment of 12.5 km long on the southern side in stead of the existing alignment from 8 km+500 m to 22 km+00 m, while it maintains the same alignment with Alternative B after the location of 22 km +00, including the tunnel of 650 m long.	This alternative employs fully a new alignment on the northern side of the existing alignment excepting existing sections in the mountainous area (4.6 km) and in the lowland areas (9.8 km). This has a preferable design speed of 40 km/h with 5 tunnels of 5,350 m long and with the shortest route length of 33.42 km.
Total Length of Road	Total Length = 45.30 km Construction of New Road = 0.56 km	Total Length = 42.35 km Construction of New Road = 1.57 km	Total Length = 41.35 km Construction of New Road = 12.72 km	Total Length = 33.42 km Construction of New Road = 20.37 km
Necessary Facilities	Tunnels = 0 m Bridges = 560 m Slope Protection = 300,000 m2	Tunnels = 650 m Bridges = 920 m Slope Protection = 260,000 m2	Tunnels = 650 m Bridges = 840 m Slope Protection = 247,470 m2	Tunnels = 5,350 m Bridges = 2,260 m Slope Protection = 126,000 m2
Environmental Effect	Minimum effect to existing environment.	There is a possibility of water pollution problem during construction of tunnel.	25 ha of forest area will be affected by new construction.	44 ha of forest area will be affected by new construction.
Disaster Prevention Aspect	Slope failure and landslide area will be protected by disaster prevention facilities.	Disaster prone area is protected and avoided by tunnel and bridge.	Disaster prone area of Tawaeli side will be avoided by the bypass.	Disaster prone area will be avoided by new route and tunnels.
Impact During Construction	Detour and traffic control will be need.	Detour and traffic control will be needed.	The time period for detour and traffic control will be minimized.	The time prevention period for detour and traffic control will be the shortest.
Construction Cost	Rp 86,000 million	Rp 114,000 million	Rp 122,113 million	Rp 356,000 million
Recommendations	Not recommendable due to low design speed.	Secondarily recommendable due to the lower cost and less affected areas of forest.	First priority is given to this route, but the affected area of forest is great.	Construction cost is too high.

Figure 6-3-2 Comparison of Alternative Routes

Chapter 7

Preliminary Engineering Design

Chapter 7 PRELIMINARY ENGINEERING DESIGN

7.1 General

This chapter describes the results of preliminary engineering design for Tawaeli-Toboli road covering the followings:

- Preliminary geometric design
- Preliminary design for disaster prevention works
- Preliminary bridge design
- Preliminary tunnel design
- Preliminary pavement design.

7.2 Geometric Design Policies

The following is a list of certain control points and design criteria for the Tawaeli-Toboli road:

- Terrain of Tawaeli-Toboli road is classified as undulating (rolling) to mountainous over most of the area.
- There are many areas suffering from hazards which need protective countermeasures.
- Existing towns and villages are to be avoided as much as possible to mitigate adverse environmental impact associated with resettlement.
- Securing of two-way traffic and safety throughout the year.

7.3 Preliminary Geometric Design

There are four alternative routes for Tawaeli-Toboli road as mentioned in Chapter 6, each route having its own characteristics.
Selected geometric design components for each alternative are as shown in Table 7-3-1.

Table 7-3-1 Selected Geometric Design Components for Alternative Routes

Item	Unit	Alternative A	Alternative B	Alternative C	Alternative D
Road Class		Class 4	Class 3	Class 3	Class 3
Terrain		Flat, Rolling & Mountain	Flat, Rolling & Mountain	Flat, Rolling & Mountain	Flat, Rolling & Mountain
Design Speed	Km/hr	20	30	30	40
Lane Width	M	3.0 x 2	3.0 x 2	3.0 x 2	3.00 x 2
Shoulder Width	M	1.0	1.0	1.0	1.0
Crossfall of Pavement	%	2	2	2	2
Crossfall of Shoulder	%	4	4	4	4
Max. Superelevation	%	8	8	8	10
Min. Radius Curve	M	15	30	30	50
Max. Gradient	%	9	8	8	7
Abs. Max. Gradient	%	13	12	12	11
Min. Clearance	M	-	5.1	5.1	5.1
Number of Lane		2	2	2	2

Source: Bina Marga

7.4 Preliminary Design for Disaster Prevention Works

7.4.1 Selection of Method of Disaster Prevention Works for Slope

(1) Cut Slope

Appropriate methods of disaster prevention for slopes are selected according to the following flow chart shown on the next page (Figure 7-4-1). Selected types are shown in Table 7-4-1.

Table 7-4-1 Slope Protection Type

Formation	Geology	Slope Protection Type
Celebes Formation	Sandstone Conglomerate Mudstone	Sprayed Concrete Cribwork Stone Masonry(slope height is up to 5m)
Metamorphic Rock	Schist	Sprayed Concrete Cribwork
	Gneiss	Shotcrete

Source: Study Team

(2) Fill Slope

These types of protection are applied to fill slope and slope failure on valley side, as follows:

Steep gradient : Caisson Type Pile
Geo-textile
Gentle gradient: Retaining Wall
Mat Gabion

An alternative of the protections for fill slope and slope failure is shown in Table 7-4-2.

7.4.2 Location and Height Each Slope Protection Types

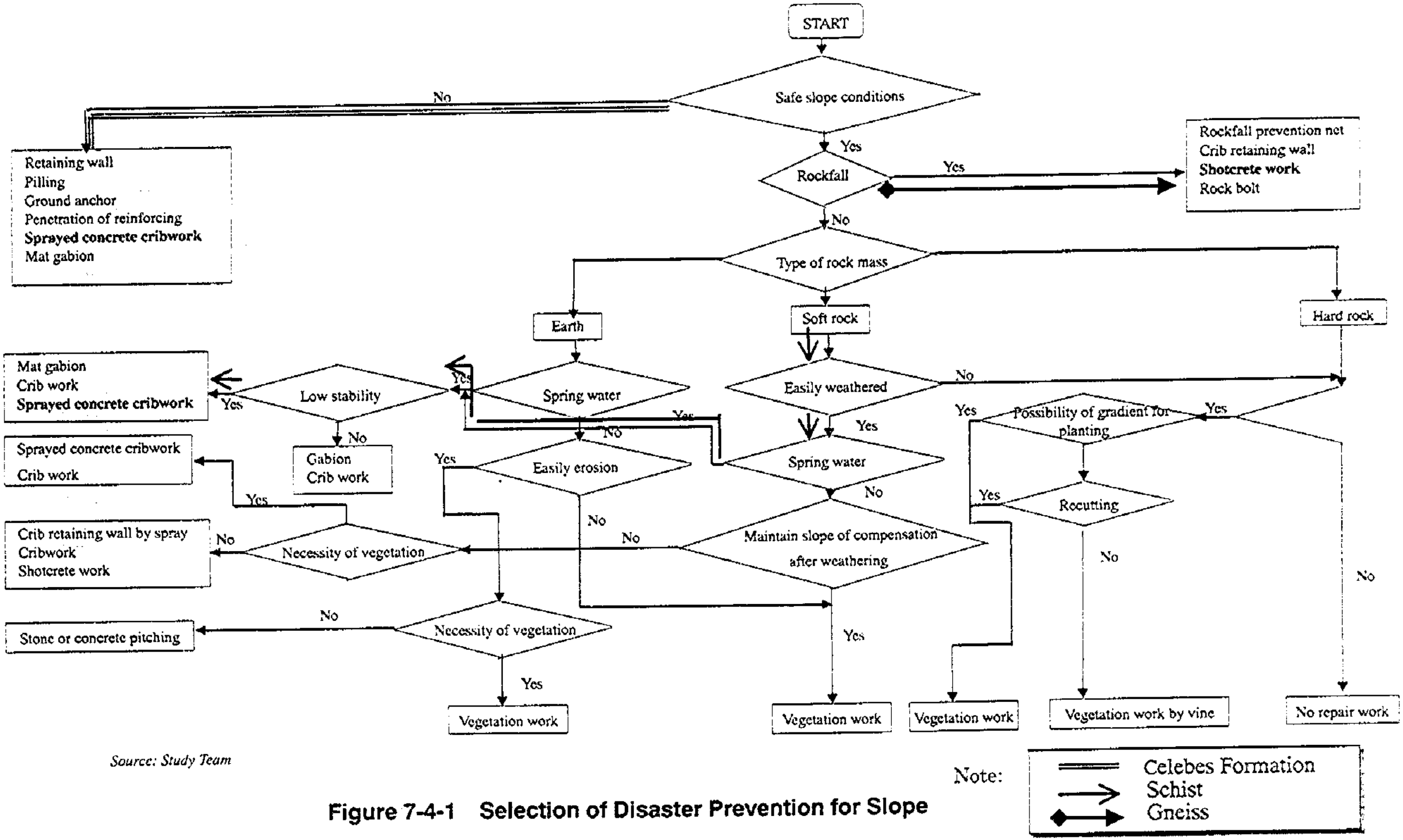
The location and height of each slope protection is shown in Figure 7-4-2.

7.4.3 Quantities of Slope Protection

Table 7-4-3 List of Quantities

	Cut			Fill
	Sprayed Concrete Cribwork(m^2)	Shotcrete (m^2)	Stone Masonry(m^2)	Mat Gabion (m^3)
Quantity	83,921	22,654	6,530	10,518
Average Height	12.5	13.1	1.7	5.0

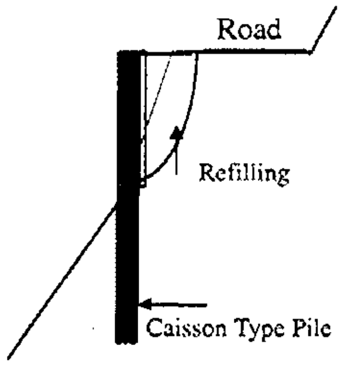
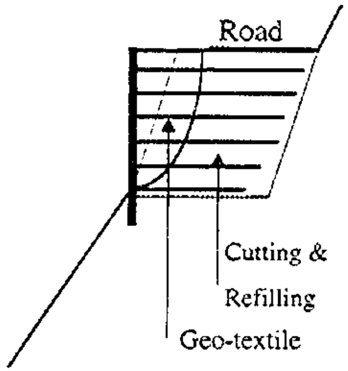
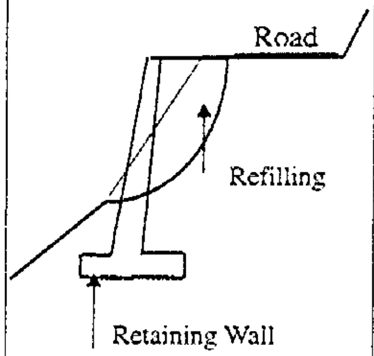
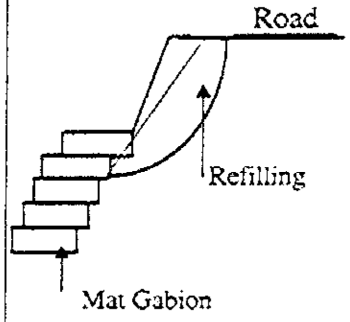
Source: Study Team



Source: Study Team

Figure 7-4-1 Selection of Disaster Prevention for Slope

Table 7-4-2 Countermeasure for Fill Slope and Slope Failure at Valley Side

		Type A Caisson Type Pile	Type B Geo-textile	Type C Retaining Wall	Type D Mat Gabion
Cross Section					
Slope Condition	Steep Gradient	Not Good	Good	Not Good	Not Good
	Gentle Gradient	Good	Good	Good	Good
Construction Aspect		Difficult	Normal	Normal	Easy
Traffic Impact		No impact	Road close for construction	Impact	No impact
Construction Cost Aspect		High	High	Low	Low
Conclusion		Not apply	Apply to steep gradient	Not apply	Apply to gentle gradient

Source:StudyTeam

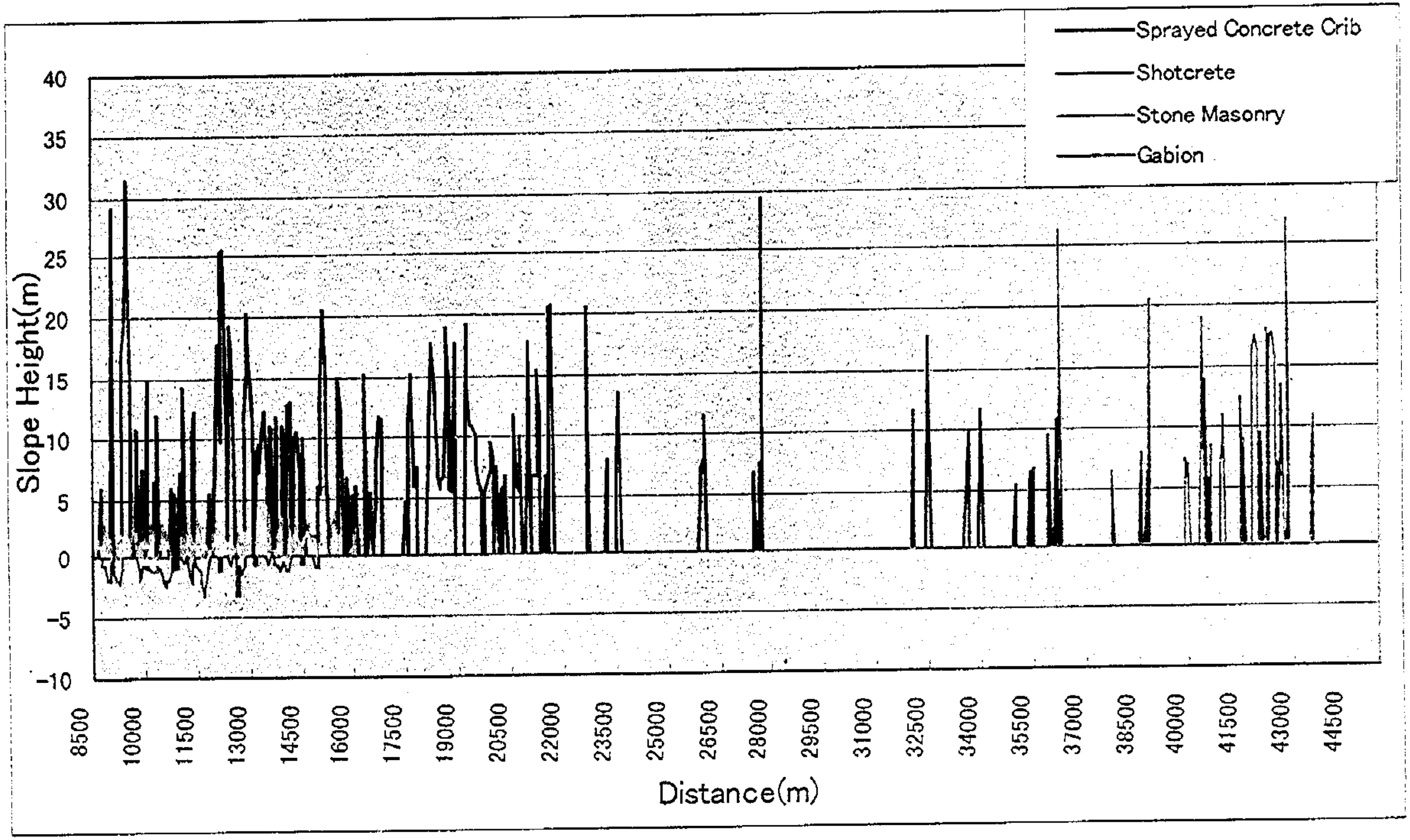


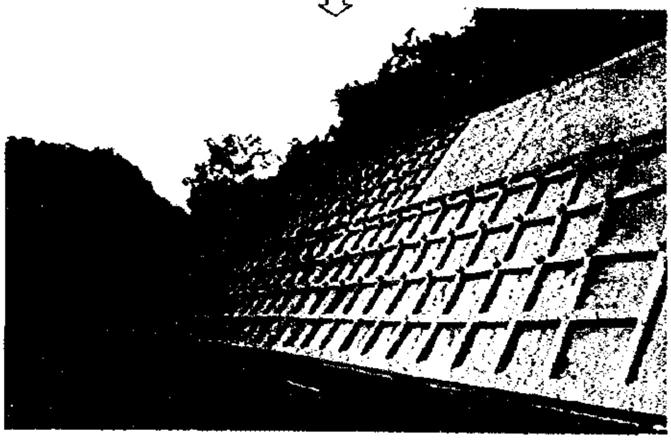
Figure 7-4-2 Location and Height of Each Protection Types

7.4.4 Image of Slope Protection

(1) Sprayed Concrete Cribwork Type

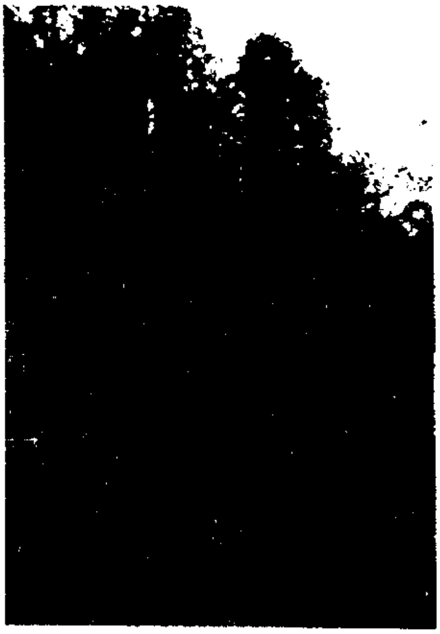
Existing Slope

Proposed Slope Protection



(2) Shotcrete Type

Existing Slope



Proposed Slope Protection

Figure 7-4-3 Image of Slope Protection

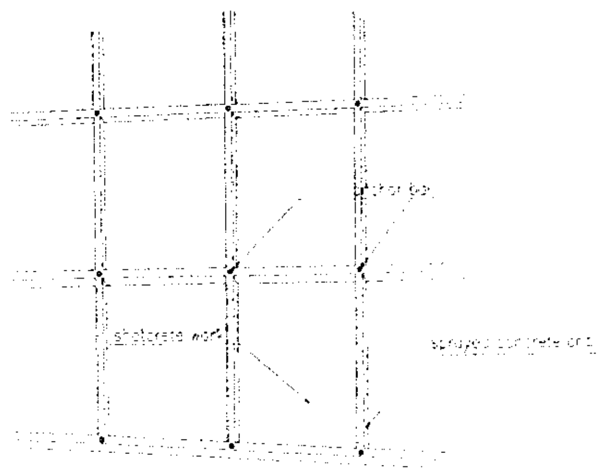
Preliminary Design for Sprayed Concrete Cribwork

(1) General

In order to stabilize road slope, sprayed concrete cribwork was recommended by the study team as a countermeasure in hazardous between Tawaeli and Toboli. Slope stabilization calculations are analyzed according to the height and features of sprayed concrete cribwork. Sprayed concrete crib of 15cm×15cm in cross section and frame dimensions of 115cm×115cm are adapted on the basis of similar constructed works.
The average protection height between Tawaeli and Toboli is 12.5 meters, and the gradient of cutting works is 1:0.5 (for soft rock).
It was suggested that the thickness of unstability superficial stratum be 1.5 meters and the internal friction angle between sprayed concrete cribwork and superficial stratum ϕ be 35° ,also that the diameter of the applied anchor bar be 22 millimeters and the pitch of the applied anchor bar be 1.15 meters. After strength calculation and stability analysis for anchor bar and sprayed concrete cribwork, the length of anchor bar is to be 2.49 meters, and here adopted to 2.5 meters for normalization.

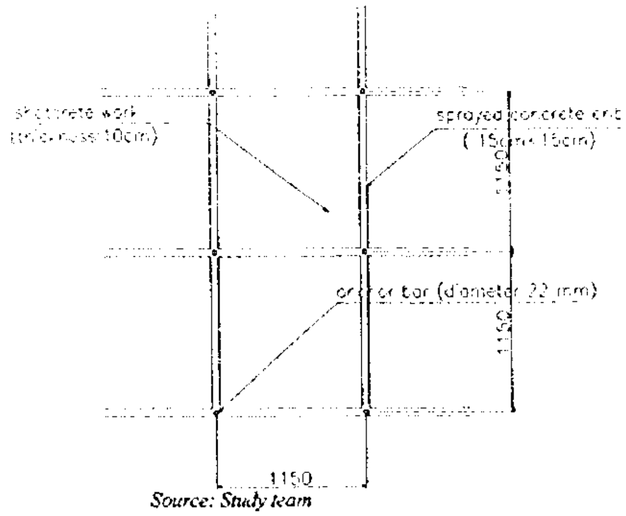
(2) Calculation of Slope Stabilization on Sprayed Concrete Cribwork

To improve present road network and make it satisfy the design standard of future plan for road network between Tawaeli and Toboli, a great amount of earth works for the construction will be conducted in the future, including cut work and fill work. Cutwork will be accompanied by many countermeasures for slope hazard, and sprayed concrete cribwork being recommended by the study team as one of the measures. The structural sketch drawing of sprayed concrete cribwork is shown in Figure 7-4-4 and Figure 7-4-5.



Source: Study Team

Figure 7-4-4 Sprayed Concrete Cribwork



Source: Study team

Figure 7-4-5 Dimension of Sprayed Concrete Cribwork

Design conditions of sprayed concrete cribwork are as follows:

- cut gradient 1:0.5 (for soft rock)
- cut slope of 12.5 meters in height.

Slope sliding was examined to calculate the safety factor as follows:

- Assume a sliding plane based on the results of unstability superficial stratum of a thickness of about 1.5 meters.
- Assume a present safety factor of 1.0, apply the linear equation below containing cohesion (c) and the angle of internal friction (ϕ =35°) in the slope material.

$$F_0 = \frac{\Sigma N \times \tan(\phi) + c \times \Sigma l}{\Sigma T}$$

Where,

F_0	=	safety factor (assume to be 1.0)
T	=	shearing stress of slice on sliding surface (t/m^2)
N	=	vertical stress of slice on sliding surface (t/m^2)
c	=	cohesion (t/m^2)
ϕ	=	angle of internal friction in the slope material (ϕ =35°)
l	=	sliced length of sliding surface (m)

After using anchor bar in the slope, the safety for sliding was calculated by the following equation.

$$F_s = \frac{\Sigma N \times \tan(\phi) + c \times \Sigma l + P_t}{\Sigma T}$$

Where,

F_s	=	safety factor after using anchor bar (assumed to be 1.2)
T	=	shearing stress of slice on sliding surface (t/m^2)
N	=	vertical stress of slice on sliding surface (t/m^2)

P_r = resistance force of anchor bar to sliding body (t/m^2)
c = cohesion (t/m^2)
ϕ = angle of internal friction in the slope material (ϕ =35°)
l = sliced length of sliding surface (m)

The resistance of anchor bar to sliding body P_r is expressed as follows:

$$P_r = F_s \times \Sigma T - (\Sigma N \times \tan(\phi) + c \times \Sigma l)$$

In regards to anchor bar design, the sharing force acting on the anchor bar is calculated by following equation:

$$S = P_r \times D_h / n$$

where

D_h = the intersperse of anchor bar along horizontal direction (suggest to be 1.15 meters)
S = shearing force acting on very anchor bar by the sliding body (t/ anchor bar)
n = quantities of anchor bar along slope surface (anchor bar)
P_r = resistance force of anchor bar to sliding body (t/m^2)

The pulling force acting on the same anchor bar is calculated by following equation.

$$T = S/\tan(\phi)$$

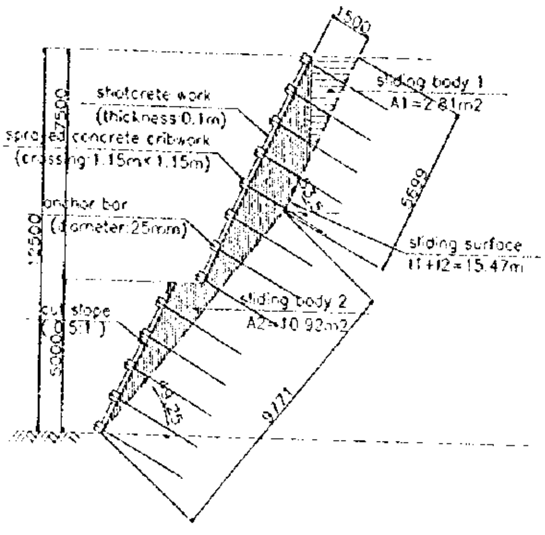
Needed length of anchor bar is calculated by the following expression

$$L = La + Lt + Lb$$

where

L = needed length of anchor bar (m)
La = additional length (usually suggested to be 0.4 meters)
Lt = the thickness of unstability superficial stratum (here suggested to be 1.5 meters)
Lb = the anchored length of anchor bar (m)

On the basis of theory, calculation had been done to identify necessary quantities of anchor bar for slope protection .
The stability calculation model is illustrated in Figure 7-4-6.



Source: Study team

Figure 7-4-6 Stabilization Calculation Model

First, calculate cohesion c using F_0 equation:

$$F_0 = \frac{\Sigma N \times \tan(\phi) + c \times \Sigma l}{\Sigma T}$$

slope angle : $\theta = 63.5^\circ$ (1:0.5) angle of breaking plane:
sliding body 1 α 1=($\theta + \phi$)/2 = (63.5° +35°)/2 = 49.25°
sliding body 2 α 2= θ = 63.5° length of breaking plane:
sliding body 1 and sliding body 2 l1+l2= 12.67m
the specific gravitation of soft rock: 2.4 (t/m^3)
the specific gravitation of sprayed concrete crib: 2.3 (t/m^3)
the specific gravitation of shotcrete : 2.3 (t/m^3)

Forced calculation of sliding body (per unit length along horizontal direction)

Sliding body number	Sectional Area ; A (m^2)	Weight W=r*V(tf)	Breaking angle	T=W*sin α (tf)	N=W*cos α (tf)
①	2.81	6.74	63.5	6.035206	3.009158
②	10.92	26.21	49.25	19.85426	17.10753
total				25.88947	20.11669

Source: Study Team

from above

$\Sigma N \times \tan(\phi)$=20.11669×tan35=14.08585482(tf)

ΣT=25.88947 (tf)

$\Sigma L = l1 + l2 = 15.47m$

$$1.0 = \frac{14.08585482 + c \times 15.47}{25.88947}$$

$c = 0.763 (t/m^2)$

After using anchor bar in the slope, with a safety factor of 1.5.

$$F_s = \frac{\Sigma N \times \tan(\phi) + c \times \Sigma l + P_r}{\Sigma T}$$

Forced calculation of sprayed concrete crib and shotcrete in sliding body
(per unit length along horizontal direction)

Type	Sectional area A (m^2)	Weight W=r*V(tf)	Breaking angle	T=W*sin α (tf)	N=W*cos α (tf)
spray concrete L	0.0225	0.786	49.25	0.595535	0.513146
sprayed concrete V	0.0225	0.366	49.25	0.276985	0.238665
shotcrete	0.01	1.380	49.25	1.045440	0.900808
total				1.917960	1.652620

Source: Study Team

from the above,

$\Sigma N \times \tan(\phi) = (20.11669 + 1.652620) \times \tan 35 = 15.24303159 (tf)$

$\Sigma T = 25.88947 + 1.91796 = 27.80743$ (tf)

$\Sigma L = l1 + l2 = 15.47m$

$$1.5 = \frac{15.24303159 + 0.763 \times 15.47 + P_r}{27.80743}$$

$P_r = 14.664501 52 (t/m^2)$

Sharing force acting on the same anchor bar:

$S = P_r \times D_h/n$

$= 14.664 \times 2.0/13 = 2.256$ (tf/anchor bar)

the pulling force acting on the same anchor bar:

$T = S/\tan(\phi)$

$T = 2.256/\tan(35) = 3.222$ (tf/anchor bar)

needed length of anchor bar:

$L = La + Lt + Lb$

$= 0.4 + 1.5 + Lb$

$$Lb = \frac{T \times F's}{\pi \times d \times \tau}$$

where

T = pulling force on the same anchor bar (m)

F's = safety factor of anchor bar for pulling force (here suggested to be 2.0)

d = diameter of pin hole (here 42 millimeters)

τ = friction of soft rock around anchor bar (0.1tf/m^2)

$$Lb = \frac{3.222012095 \times 2}{\pi \times 0.035 \times 100}$$

$$= 0.586056\text{m}$$

$$L = 0.4+1.5+0.586056=2.486056 \text{ m}$$

Take 2.5 meters as standard length; therefore the length of anchor bar for slope protection is 2.5 meters.

Through the above calculation , when safety factor of slope stabilization is 1.2, the length of anchor bar of slope protection can be taken to be 2.5 meters.

7.5 Preliminary Bridge Design

7.5.1 Bridge Improvement Policies on Taweli-Toboli Road

(1) Existing Bridge Conditions

Existing bridges including box culverts on Taweli-Toboli road are listed in Table 7-5-1.

Table 7-5-1 Existing Structures along Tawaeli-Toboli Road

Structure	No.	Length (m)	Width (m)	Construction Year
Stone arch	6	2.1～6.0	6.1～6.7	1973～1981
Reinforced concrete plate (slab)	13	4.7～6.3	5.7～7.0	1970～1981
Reinforced concrete girder	3	6.6～12.8	7.0	1970～1976
Steel girder	1	6.8	7.5	1973
Steel truss	3	48.2～75.3	3.5	1981
Reinforced concrete box culvert	2	2.0	5.7, 7.0	1980, 1981

Source: Study Team

- All structures are aged more than 15 years and the oldest one is aged 27 years.
- Most structures have enough width for the present traffic condition with exception of the steel truss bridges.

- New steel truss replacement bridges are now under construction at three (3) locations.
- A large number of abutments of reinforced concrete slab bridges, judged by visual examination, are not sufficiently stable.
- Settlement of pavement on the abutment is caused by heavy truckloads, insufficiency of compaction, and rainfall.
- Some slope collapses around the abutment due to steep slope location are noticed.

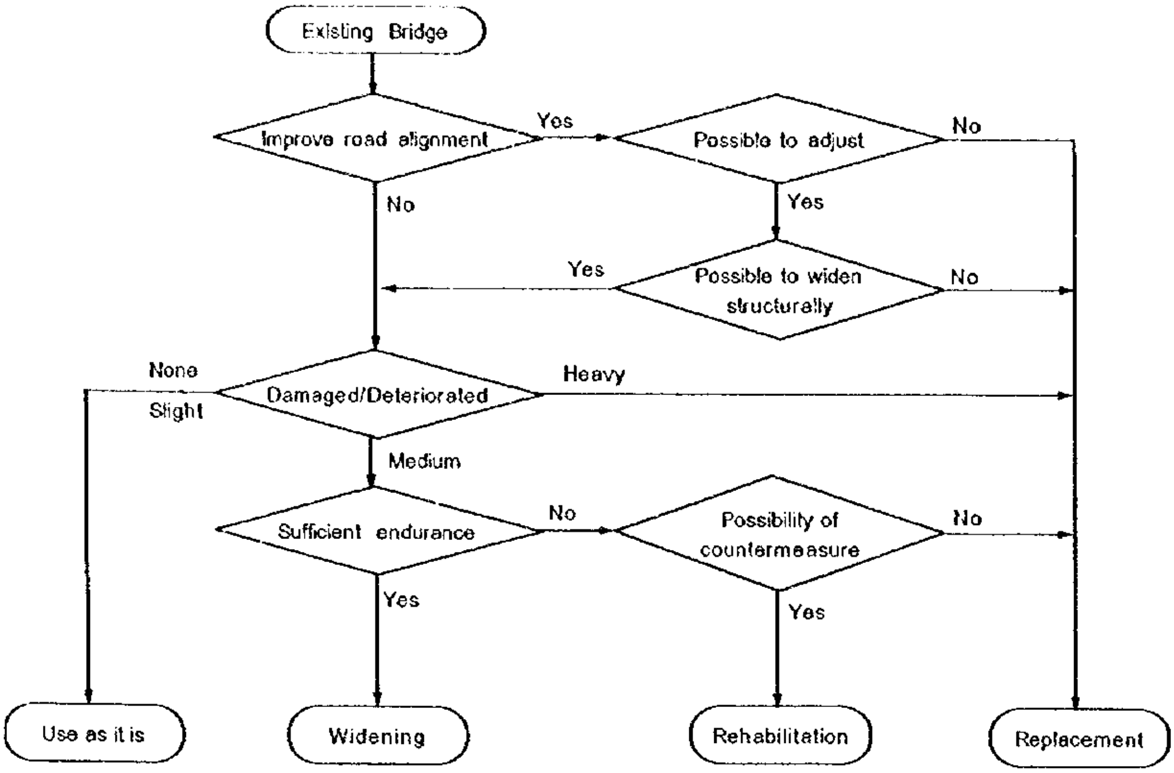
(2) Bridge Improvement Policies

Existing bridges are evaluated by their effective width, loading capacity, clearance against flood water level, settlement of substructure as well as degree of deterioration of structure.

Furthermore, the improvement plan for each bridge is determined from among the four categories in accordance with the overall road improvement plan.

- Replacement (bridges affected by the improvement of road alignment, seriously damaged or deteriorated bridges, bridges with narrow width and widening is judged impractical);
- Rehabilitation (replacement of deck slab, repair and reinforcing of structure, protection of substructure);
- Widening; and
- Use as is.

General procedure for the determination of bridge improvement plan is shown in Figure 7-5-1.



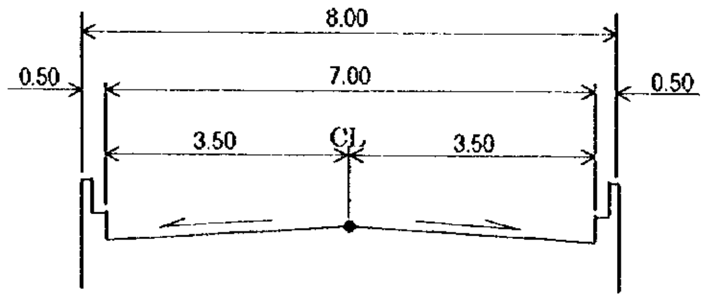
Source: Study Team

Figure 7-5-1 General Procedure for the Determination of Bridge Improvement Plan

(2) Bridge Improvement Plan

1) Bridge Width

Standard bridge width on Tawaeli-Toboli road is shown in Figure 7-5-2 in accordance with the standard section of road (refer to Clause 7.4).



Source: Study Team

Figure 7-5-2 Standard Bridge Section

2) Other Criteria

- Main Material of Bridge Structure

The bridge is to be replaced if the existing bridge is of timber or stone, as it is not likely that a timber or stone bridge can withstand future traffic demand. Therefore these bridges to be replaced.

- Endurance

Endurance can be estimated based on the bridge inventory by Bina Marga BMS. In that inventory, the degree of damage of bridges is recorded by use of numerals from 0 to 5 according to amount of damage.

In this study bridges ranked from 3 to 5 should be replaced based on the judgement that they lack endurance under the new road and bridge improvement plan.

3) Bridge Improvement Plan on Tawaeli-Toboli Road

Bridge improvement of existing bridges on Tawaeli-Toboli road should be planned as follows;

- Replacement (New construction)
  Bridges which are less than 7.0 m in effective width, constructed from stone or lack sufficient endurance should be replaced. Judgement on bridge endurance was based on the bridge inventory modified by actual site investigation.

- Rehabilitation
  Rehabilitation work of the bridges will be carried out through road maintenance work. Therefore rehabilitation work should not be included in the category of this bridge improvement plan.

- Widening
  In this improvement plan the widening of bridges is treated as new construction due to the complications of their structural features.

7.5.2 General Description of Bridge Type

(1) Superstructure

In general the type of bridge is determined based on the surrounding conditions such as road alignment, crossing of river, environmental requirements and economic priorities. In particular future road network and river improvement plan should be taken into consideration determining bridge span length. Bridge types are subject to their applicable span lengths.

Table 7-5-2 Bridge Types and Standard Span Length

Material	Type of Structure	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	200
Reinforced Concrete	Simple T-Girder	■															
	Hollow Slab	■	■														
	Rigid Frame	■															
Prestressed Concrete	Hollow Slab		■	■													
	Simple I-Girder			■	■												
	Simple T-Girder			■	■	■											
	Simple Box Girder				■	■	■										
	Continuous Box Girder				■	■	■	■	■	■	■						
	Box Girder with hinge											■	■	■	■	■	■
	Continuous Rigid Girder						■	■	■	■	■	■	■				
Steel Bridge	Simple Composite Girder		■	■	■												
	Simple Box Girder			■	■	■											
	Countinuous Box Girder							■	■	■	■	■	■	■	■	■	
	Truss Girder											■	■	■	■	■	■

Span length (m)

Source: Study Team

(2) Substructure

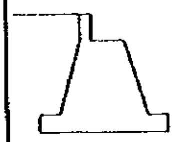
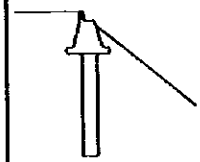
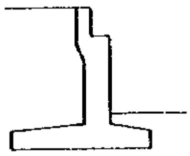
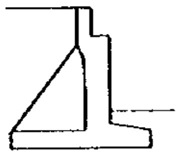
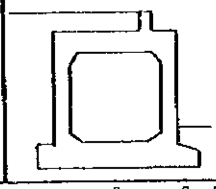
1) Abutments

An abutment is a structure located at the end of a bridge which provides the basic functions of:

- Supporting the end of the first or last span
- Retaining earth underneath and adjacent to approach roadway, and, if necessary
- If necessary supporting part of the approach roadway or approach slab

To provide this functions, a variety of abutment forms are used. The style of abutment chosen for a given bridge varies depending on the geometry of the site, size of the structure, and preferences of the client. A simplification would be to think of an abutment as a retaining wall equipped with a bridge seat.

Table 7-5-3 Abutment Types and Standard Height

Abutment Type	Height(m) 10 20 30 40	Remarks
Gravity Abutment		
Stub Abutment		
Cantilever Abutment		
Counterfort Abutment		
Rigid Frame Abutment		

Source: Study Team

2) Piers

A pier is a structure located at the end of a bridge span which provides the basic function of supporting spans at intermediate points between end supports (abutments). Piers are predominantly constructed using concrete, although steel and, to a lesser degree, timber are also used. Concrete is customarily reinforced. Prestressed concrete, however, is sometimes used as pier material for special structures.
The basic design functions of a highway bridge pier can be summarized by the following list. In general, a pier is designed to:

- Carry its own weight
- Sustain superstructure dead and live loads

- Transmit all loads to the foundation

In addition to providing the structural functions detailed above, a properly designed pier should also be aesthetically pleasing and economize the use of materials as much as possible. Also, piers should be located so that they provide minimal interference with traffic passing or water flow underneath the structure. General style of piers are as follows;

1. Hammerhead shaped pier
2. Column bent pier
3. Pile bent pier
4. Solid wall pier
5. Integral pier
6. Single Column pier

Structural type of piers should be determined to satisfy surrounding conditions as well as structural requirements. In rivers and canals, pile bent or oval shaped solid wall piers are generally used.

(3) Foundations

Bridge foundation is generally classified in two categories, that is, shallow foundation and deep foundation. A shallow foundation calls for spread foundation, while a deep foundation calls for a pile foundation. Type of foundation is principally selected in proportion to the depth of bearing layer.

Along the entire Tawaeli-Toboli road, a bearing layer of weathered rock with SPT value of more than 50 is distributed at a depth of about 5 m from the ground surface, according to the boring survey. (refer to Chapter 4).

Therefore, in this preliminary design, spread foundation is applicable as the bridge foundation.

7.5.3 Preliminary Bridge Design

(1) Design Criteria of Bridge

To simplify construction and design of bridges, a bridge type designed in compliance with the Indonesian Bridge Design Standard and listed in the Standard Bridge Design by Bina Marga, with an effective width of 7.0m was considered.

1) Superstructure

Type	:	Prestressed concrete simple I-girder
Width	:	0.5+7.0+0.5 (m)
Length	:	10 m ~ 30 m
Live Load	:	D loading, T loading
Concrete	:	Girder; Specified strength at 28 days = 400 kgf/cm^2

		Cross Beam; Specified strength at 28 days = 250 kgf/cm^2
Tendon	:	Uncoated 7 wire super strand (ASTM A-416, grade 270)
		Diameter; 12.7 mm
Re-bar	:	BJ-32
		fsy = 320 MPa

2) Substructure

Abutment	:	Reinforced concrete, Cantilever abutment
Pier	:	Reinforced concrete, Hammerhead shaped pier
Concrete	:	Specified strength at 28 days = 240 kgf/cm^2
Re-bar	:	BJ-32
		fsy = 320 MPa

3) Foundation : Spread foundation

(2) Summary of Bridge Improvement

Bridge improvement plan on Tawaeli-Toboli road is summarized in Table 7-5-4.

Table 7-5-4 Summary of Proposed Bridges and Box Culverts on Tawaeli-Toboli Road

NO	LOCATION (STA.)	STRUCTURE	NUMBER	SPECIFICATION Length L(m)	Span (Nos.)	Width W(m)	REMARKS
1	2 + 700	Bridge	BR-1	75.0	2		Under construction of New Bridge
2	4 + 860	Bridge	BR-2	57.0	2		Under construction of New Bridge
3	5 + 190	Bridge	BR-3	4.9	1	7.00	Keep existing
4	6 + 70	Bridge	BR-4	48.0	2		Under construction of New Bridge
5	7 + 550	Bridge	BR-5	4.8	1	7.20	Keep existing
6	8 + 200	Bridge	BR-6	4.7	1	7.10	Keep existing
7	8 + 550	Bridge	BR-8	4.7	1	7.20	Keep existing
8	8 + 600	Bridge	BR-9	20.0	1		
9	8 + 970	Bridge	BR-10	40.0	2		
10	9 + 95	Bridge	BR-11	20.0	1		
11	10 + 390	Box Culvert	BC-1	20.0	1Box		
12	10 + 570	Bridge	BR-12	30.0	1		
13	10 + 640	Bridge	BR-13	20.0	1		
14	10 + 750	Box Culvert	BC-2	20.0	1Box		
15	10 + 840	Bridge	BR-14	30.0	1		
16	11 + 230	Box Culvert	BC-3	20.0	1Box		
17	11 + 670	Bridge	BR-15	50.0	2		
18	11 + 760	Box Culvert	BC-4	20.0	1Box		
19	11 + 920	Box Culvert	BC-5	20.0	1Box		
20	11 + 975	Bridge	BR-16	20.0	1		
21	12 + 725	Bridge	BR-17	30.0	1		
22	13 + 90	Bridge	BR-18	20.0	1		
23	13 + 325	Box Culvert	BC-6	20.0	1Box		
24	13 + 470	Box Culvert	BC-7	20.0	1Box		
25	13 + 670	Box Culvert	BC-8	20.0	1Box		
26	14 + 40	Bridge	BR-19	20.0	1		
27	14 + 425	Bridge	BR-20	20.0	1		
28	14 + 750	Box Culvert	BC-9	20.0	1Box		
29	14 + 850	Bridge	BR-21	40.0	2		
30	15 + 675	Box Culvert	BC-10	20.0	2		
31	16 + 225	Bridge	BR-22	30.0	1		
32	16 + 470	Box Culvert	BC-11	20.0	1Box		
33	16 + 865	Box Culvert	BC-12	20.0	1Box		
34	18 + 525	Box Culvert	BC-13	20.0	1Box		
35	18 + 740	Box Culvert	BC-14	20.0	1Box		
36	19 + 640	Bridge	BR-23	30.0	1		
37	20 + 240	Bridge	BR-24	40.0	2		
38	20 + 430	Box Culvert	BC-15	20.0	1Box		
39	20 + 640	Bridge	BR-25	20.0	1		
40	24 + 10	Box Culvert	BC-16	20.0	1Box		

No.	LOCATION (STA.)	STRUCTURE	NUMBER	SPECIFICATION Length L(m)	Span (Nos.)	Width W(m)	Remarks
41	24 + 270	Bridge	BR-26	10.0	1	6.70	Replace due to insufficient width
42	24 + 400	Box Culvert	BC-17	20.0	1Box		
43	25 + 520	Bridge	BR-27	30.0	1		
44	25 + 770	Bridge	BR-28	20.0	1	6.10	Replace due to insufficient width
45	27 + 125	Bridge	BR-29	5.8	1	7.20	Keep existing
46	27 + 280	Box Culvert	BC-18	20.0	1Box		
47	27 + 415	Bridge	BR-30	30.0	1		
48	32 + 315	Box Culvert	BC-19	20.0	1Box		
49	32 + 975	Bridge	BR-31	10.0	1	6.00	Replace due to insufficient width
50	33 + 150	Bridge	BR-32	20.0	1		
51	33 + 930	Box Culvert	BC-20	20.0	1Box		
52	34 + 365	Box Culvert	BC-21	20.0	1Box		
53	34 + 780	Bridge	BR-33	10.0	1	6.00	Replace due to insufficient width
54	35 + 220	Bridge	BR-34	10.0	1	6.00	Replace due to insufficient width
55	35 + 370	Bridge	BR-35	10.0	1	6.00	Replace due to insufficient width
56	37 + 120	Box Culvert	BC-22	20.0	1Box		
57	37 + 400	Box Culvert	BC-23	20.0	1Box		
58	38 + 25	Bridge	BR-36	20.0	1		
59	38 + 400	Box Culvert	BC-24	20.0	1Box		
60	39 + 335	Bridge	BR-37	20.0	1		
61	39 + 650	Box Culvert	BC-25	20.0	1Box		
62	40 + 50	Bridge	BR-38	40.0	2		
63	40 + 530	Bridge	BR-39	20.0	1		
64	40 + 690	Bridge	BR-40	20.0	1		
65	41 + 140	Bridge	BR-41	20.0	1		
66	41 + 255	Bridge	BR-42	10.0	1		
67	41 + 980	Bridge	BR-43	20.0	1		
68	42 + 300	Bridge	BR-44	10.0	1		
69	42 + 675	Bridge	BR-45	10.0	1		
70	43 + 250	Bridge	BR-46	10.0	1		

Source: Study Team

7.6 Preliminary Tunnel Design

7.6.1 Categorization of Rock Mass

(1) Overview

It is of utmost importance to discern the various ground characteristics, that is to say behavior of ground-rock of the vicinity and the approximate ground pressure affecting the tunnel during excavation, in the event of designing and constructing underground structures such as tunnels.

The characteristic feature of tunnel structure is that it is linear, therefore general comprehension of geological variations in the horizontal alignment is necessary.

In general, characteristics of rock mass differ due to influence of period of formation, geological structure, erosion and deterioration factors, as well as presence of aquifers; therefore an engineering-based evaluation can be very difficult to make. Furthermore, at the design stage, the scope of survey is very limited, making it extremely difficult to obtain a proper grasp of the geological qualities of the ground in question.

In order to design the tunnel effectively and practically, each case should have categorized and patternized indicators made according to geological qualities.

Thus, classification is carried out according to characteristics of different types of rock obtained through a comparison of geological survey results and previous examples of completed projects.

In addition, as a criterion for pre-design, a standard combination for tunnel support according to rock mass classification is as shown in Table 7-6-3.

The indices used in evaluating rock mass are generally obtained from surveying and tests. As these indices can be considered objective and consistent, it is most appropriate that indices be obtained from the entire length of the project in question. Therefore, indices obtainable from analyses such as surface geological survey, seismic prospecting, boring survey, soil quality and rock tests, etc., will be used.

(2) Rock Mass Classification of Project Tunnel

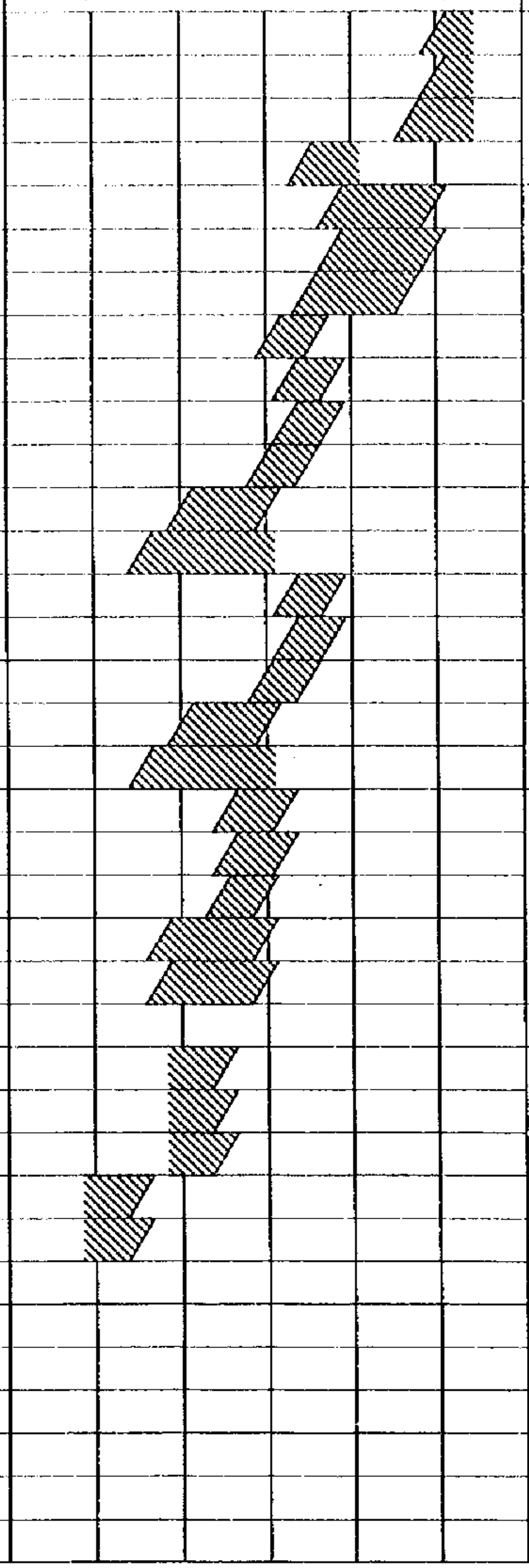
Whereas the behavior of the interior of the tunnel is dominated by intraterranean conditions of rock quality and geological structure, the behavior of the tunnel entrance area is dominated by these conditions as well as by external conditions of topography, meteorology, etc.

Therefore, in determining rock mass classification, tunnel entrance and tunnel interior are treated separately.
As for what the tunnel entrance area entails, it is often considered as the area from the point of the tunnel portal to the point inside the entrance where the formation of a grand arch 1 to 2 x D (diameter of tunnel) is possible.
The geological survey shows that the rock of the interior of the tunnel consists mainly of gneiss, a metamorphic type of rock. Seismic speed from three strata beginning with the upper strata is vp=0.15~0.50km/sec, vp=0.60~1.80km/sec, and vp=2.50~3.30km/sec, respectively. It is also prognosticated that a lower speed belt (vp=1.40km/sec) of about 20m in width is located at the center of the tunnel at approximately the tunnel entrance on the commencing side.
Furthermore, the one-axis compressed strength value derived from rock test of the geological survey was comparatively low at approximately qu = 50kg f/cm2. This is due to the fact that the rock itself is aged, and that the test value was obtained from a core of a relatively shallow depth. Judging from the actual period of the rock formation, rock type, and overburden of tunnel, it is supposed that the area around the tunnel center consists of a solid rock mass.
The rock involved in this tunnel project has gneiss as its parent rock, and the seismic speed value is shown in relation to the rock category in the table below, which is drawn according Japanese standards. These categories classify rock with favorable conditions in class "A" (extremely hard rock with few cracks) and categorized downward to "D II" in order of degradation of rock condition.
Seismic speeds of the tunnel in question are shown in the right-hand column. Based on these results, the rock category is postulated as shown in Table 7-6-1.
The lower speed belt is classified under "D II". Furthermore, although the seismic speed at the center of the tunnel is analyzed at 2.5~3.3 km / sec, the area of the planned tunnel is assumed to be of better bedrock, so is categorized at "C II". And also a comon rock massclassification is shown table 7-6-2.

Table 7-6-1 Rock Categories at Tunnel Interior

Standard Japanese Classification		Project tunnel Classification	
Category	Seismic speed (km/sec)	Category	Seismic speed (km/sec)
A	More than 5.0		
B	3.80~4.9		
CI, CII	3.2~3.7		
DI	2.5~3.1	DI~CII	2.5~3.3
DII	Less than 2.4	DI	0.15~1.80

Standards for Rock Mass Classification



Rock Mass Grade		Petrological Classification	(1) Standard by Seismic Waves Velocity (Vp, km/sec.)	(2) Gɴ	(3) Standard by Boring Core Sample: Condition of core sample	(3) RQD (%)	(4) Standard by geological condition (the result of ground investigation or the condition of excavated rock mass)	(5) Standard by Observation: Hitting with Hammer	(5) Spacing of Cracks	(6) After excavated condition: Standard by stability of face	(6) Convergence
A		a b c d_1		—	Core recovery rate is more than about 90 % with complete column shape, having length of more than about 20 cm, without containing small pieces.	more than 90	• The condition of rock is very hard and fresh, and consists of massive blocks without cracks, which continuous and stable over the large area. • Rock mass is not inferior by water.	Hammer is bounded. The rock is cracked with fresh surface only when hitted strongly.	50 ~ more than 100 cm	• The condition of face stability is very good and not loose for a long time. • Height of looseness, less than 1.6 m	very small
B		a b c d_1		—	Core recovery rate is more than about 70 % and the core shows large block or short column or bar shape. Core having length may be about 10 to 20 cm, but in rare case, 5 cm or so.	70 ~ 90	• The condition of rock is hard and fresh, and contains relatively less cracks. • The condition of rock is relatively hard but shows somewhat altered property due to weathering. • The condition of rock is hard but assumes a layer from having bedding or schistosity and tends to be cracked along the surface. • Rock mass is not inferior by water.	The rock develops cracking or cut relatively largely along the joint or crack when hit strongly.	30 ~ 70 cm	• Cutting face keeps stability and excavation without support has sectionally fall of rocks but generally stable. • The sectionally loose zone must support. • Height of looseness, 1.5 to 3.0 m	very small
C	I	a b c d_1 d_2		— (a, b, c) more than 4 (d_1, d_2)	Core recovery rate is roughly 40 to 70 % containing many cracks, and the core also cracks easily to a mass less than 5 cm. Restore of original shape is difficult or impossible.	10 ~ 70	• Altered property due to weathering, and the condition of rock is somewhat soft. • The condition of rock is relatively hard but contains many small cracks thereby showing the appearance of small masses. Joint may contain clay despot. • Bedding and schistosity are remarkable. Easy cleavage with thin layer. • Narrow, small fault is contained. • Rock mass is not inferior by water.	Crushed easily with hammer. Crushed into small pieces along the cracked face. Faces without containing cracks are hardly crushed.	about less than 50 cm	• Cutting face keeps stability. • Excavation without support needs concrete shotcrete for crown area at once after blasting. • Height of looseness, 2.0 to 4.0 m	less than 50 mm
C	II	a b c d_1 d_2		— (a, b, c) more than 4 (d_1, d_2)						• Cutting face keeps stability. • Excavation without support has fall of rock near crown area and which needs presupport. • Height of looseness, 2.0 to 4.0 m	
D	I	a b c d_1 d_2		2 ~ 4	Core recovery rate reduces down to less than 40 %, the core may be composed of small pieces or may be a rubble sand state or clay state.	about less than 10	• Considerably weathered rock mass, partly showing soiled part containing some hard parts, is soft and fragile. • Rock mass has many cracks and rock can be crushed easily from any part other than crack. • Rock mass has crushed rock zonein which tendency toward clay, not much progressing, and contained some hard components in which clayer soil and fine pieces are being mixed. • Soils, taluses, etc. • The case which the cutting face having fragile by water is D II grade.	Crushed easily if hitted by hammer. Crashed easily by fingers.	—	• Cutting face and excavation without support have substantially fall of rocks, and the side wall has some squeezing. The ground needs presupport and landside protection at a early stage. • Plastic zone or height of looseness, 3.0 to 6.0 m	less than 60 mm
D	I	e		more than 2							
D	II	a b c d_1 d_2 e		1 ~ 2				Crushed easily by a small hammer damage. The point of hammer sticks into rocks.	—	• Cutting face has substantially fall of rocks. At excavation without support, the side wall has squeezing. • Plastic zone or height of looseness, 3.0 to 6.0 m	less than 200 mm
E		a b c d_1 d_2 e		less than 1	—	—	• Fault and crushed rock zone or taluses with a considerable width, on which considerable eccentric earth pressure is acting. • Inferior by ground water is relatively, and to be weakness.	—	—	• Cutting face has squeezing and in a striking case, cutting face collapse. • Excavation without support has squeezing with circumference pressure. • Plastic zone, more than 7.0 m	less than 400 mm

1. Petrological classification
   a: Metamorphite: (phyllite, graphite, schist, silicic graphite, schist, quartzschist, greenschist, gneiss, serpentine, hornfels, etc.) Plutonite: (grabboro, peridotite, etc.)
   b: Paleozoic strata and Mesozoic formation: (Slate, sandstone and conglomerate, graywacke, limestone, quartzite, schalstein, etc.)
   c: Vocanic rock : (liparite, andesite, basalt, etc.)
   Dike rock : (quartz porphyry, granite, diabase, etc.)
   Plutonite : (granite, diorite, etc.)
   d: Tertiary formation and lower diluvium: (mudstone, shale, siliceous shale, sandstone and psephite, limestone, tuff, breccia, agglomerate, etc.)
   But, the classfication d_1, and d_2 is due to the compressive strength of fresh rock mass which point of out 200 kgf/cm².
   d_1: $qu \geq 200$ kgf/cm²
   d_2: $qu < 200$ kgf/cm²
   e: Upper diluium (loam and cray, volcanic crushed forenation, etc.)
   Alluvium (talus, surface soil, etc.)
2. The condition of boring core sample. RQD, and spacing of cracks applies petrological classification for a, b, c, d_1.
3. $G_N = qu / \gamma h$ qu: uniaxial compressive strength of ground, γ: unit weignt of ground, h: depth of overburden

Table 7.6.2 Common Rock Mass Classification

7.6.2 Design of Cross - Section

(1) Tunnel Interior Cross - Section

The tunnel interior cross section must include consideration of the following conditions:

- I. Width and construction limit in accordance with standards indicated in road structure statutes
- II. Construction limit in case of existence of inspection way and/or pedestrian way
- III. Variation of crossfall
- IV. Dimensions of pavement and drainage
- V. Margin for ventilation facilities
- VI. Implementation error

Furthermore, design and form must be pragmatic in terms of implementation and economics, carried out in full consideration of topographical and geological conditions. The width and construction limit of the project tunnel is as shown in the next figure.

- Full width : W=0.75+(0.75+3.00)x2;0.50+0.75=9.50 m
- Crossfall : i= 2.00% (upward grade from both portals)
- Inspection way : Installed on both sides (w=0.75 m, h=2.00 m)

According to the above items, the interior cross section of the project tunnel results in the following way:

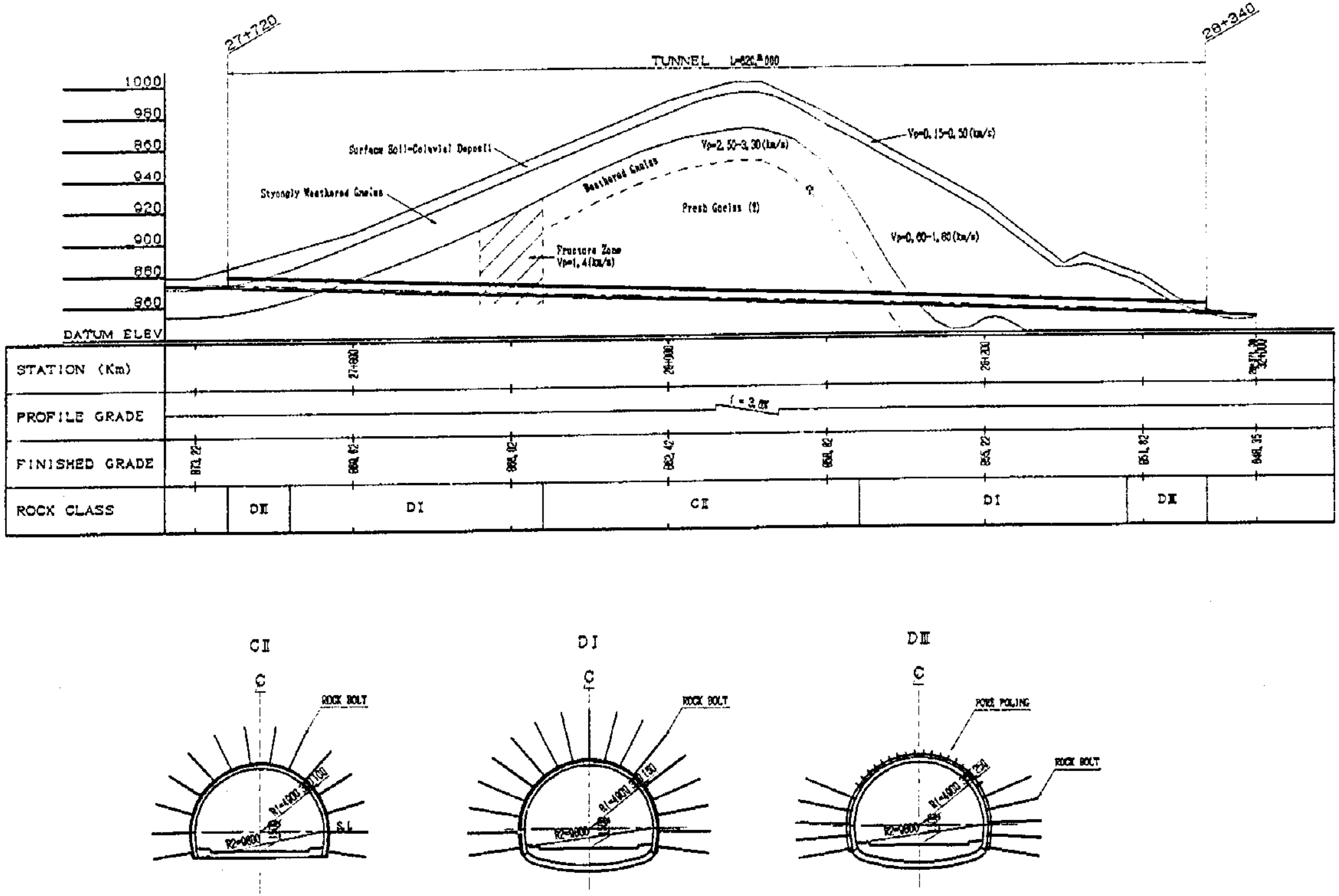


Figure 7-6-1 Plan and Profile of Tunnel

(2) Examination of Support Structure

The purpose of support structure is to preserve stability and provide safer conditions in the basically unstable status which prevails in the rock mass throughout excavation, construction, as well as following completion.
The stability of the surrounding rock during tunnel excavation depends on the rock mass conditions. Therefore, it is necessary to select a construction method and support structure which meets the conditions presented locally.
The support structure of the project tunnel will be considered according to the differing construction methods of the tunnel interior and tunnel entrance.

1) Consideration of support structure for tunnel interior

As the scale of the project tunnel is two-lane with an excavation width of 10m, planning of tunnel support structure will be conducted according to the " Road & Tunnel Engineering Standards (structure edition)".
Standard support structure combinations taken from " Road & Tunnel Engineering Standards (structure edition)" are shown in Table 7-6-3.

Table 7-6-3 Standard Support Structure Combinations

Rock category	Rock bolt			Steel arch support			Spray thickness (cm)	Cover thickness (cm)	Invert thickness (cm)
	Length (m)	Interval		Upper half	Lower half	Interval (m)			
		Circuit direction (m)	Length direction (m)						
A	Differs greatly according to rock condition. Considered separately.								
B	3.0	1.5 upper	2.0	None	None		5	30	
CI	3.0	1.5	1.5	None	None		10	30	(40)
CII	3.0	1.5	1.2	H-125 U-21	None	1.2	10	30	(40)
DI	4.0	1.2	1.0	H-125 U-21	H-125 U-21	1.0	15	30	45
DII	4.0	1.2	Under 1.0	H-125 U-21	H-125 U-21	Under 1.0	20	30	50
E	Differs greatly according to rock condition. Considered separately.								

(For interior width of 10 m, upper half cross-section construction technique)

As a result, support structure for the project tunnel will be as indicated in Table 7-6-4.

Table 7-6-4 Tunnel Support Patterns (tunnel interior patterns)

Category of rock quality	Spray thickness (cm)	Cover thickness (cm)	Invert thickness (cm)	R x B Length	F x P Length	Support	Remarks
C II	10	30	None	3.0	None	H-125 Two piece	Progressive length
D I	15	30	45	4.0	None	H-125 Four piece	Progressive length

Note: The implementing pitch of the circuit direction of the rock bolt is 1.5 m for C-class rock and 1.2 m for D-class rock respectively.

2) Consideration of support structure of tunnel entrance

In regards to the support structure of the tunnel entrance, it is a general rule that it is constructed with more durable strength than the interior. The table 7-6-5 indicates the criteria for the standard combinations of support structures of an interior width of about 10 m. In general, upper half cross-section construction techniques are used, but in cases where the bearing capacity of the supporting strata is small, a side-drift progression construction technique may also be used.

Table 7-6-5 Standard Combinations for Tunnel Entrance Support Structures

Excavation method	Steel arch support			Spray thickness (cm)	Cover thickness (cm)		
	Upper half	Lower half	Interval		Arch	Side-wall	Invert
Upper half cross-section technique (leaving core)	H-200	H-200	1.0	25	35	35	50
Side-drift progression technique (leaving core)	H-200	-	Over 1.0	25	25	*	under 50

Source : Road & Tunnel Engineering Standards (structure edition)

The supporting structure of the tunnel entrance in the project tunnel is indicated in Table 7-6-6.

Table 7-6-6 Tunnel Support Patterns (tunnel entrance patterns)

Category of rock quality	Spray thickness (cm)	Cover thickness (cm)	Invert thickness (cm)	R x B Length	F x P Length	Support	Remarks
D III	25	35	50	4.0	3.0	H-200	Progressive length 1.0 m

Note: (1) The implementing pitch of the circuit direction of the rock bolt is 1.2 m.

(2) The implementing pitch of the circuit direction of fore-piling is 0.6 m.

7.6.3 Examination of Construction Methods

(1) Excavation methods

Regarding excavation methods for the project tunnel, the spraying/rock bolt method is adopted in this case, as it is a basic construction method in Japan, and has been utilized in underground electrical generation plants and water tunnels in Indonesia.

Generally, the spraying/rock bolt method combines the functions of spray concrete, rock bolts, and steel arch support; and is commonly used in situations for retaining rock mass. Specifically, it entails the provision of immediate support (by spraying concrete) following excavation which helps prevent the loosening of surrounding rock during construction, followed by the implementation of rock bolts which retain the rock-supporting capacity. At present, this is one of the most representative methods of tunnel excavation.

(2) Types of excavation

Among types of excavation for tunnels are use of explosives, use of excavating machinery, and use of manpower. In terms of labor efficiency and constructability, the former two have become the most implemented. In choosing the most suitable type of excavation, sufficient consideration must be made for ease in construction, economics, safety, etc., as well as for rock conditions, scale of tunnel, and environmental circumstances to prevent loosening of ground rock.

In selecting the type of excavation for the project tunnel, it should be remembered that metamorphic gneiss is widely distributed throughout the area and it its predicted that there should be relatively favorable rock conditions in the area of depth of the tunnel.

Therefore, judging from the points mentioned above, excavation will be carried out by blasting methods.

- Excavation construction method : spray/rock bolt method
- Type of excavating : Excavation by blasting

See order of construction indicated on the following page.

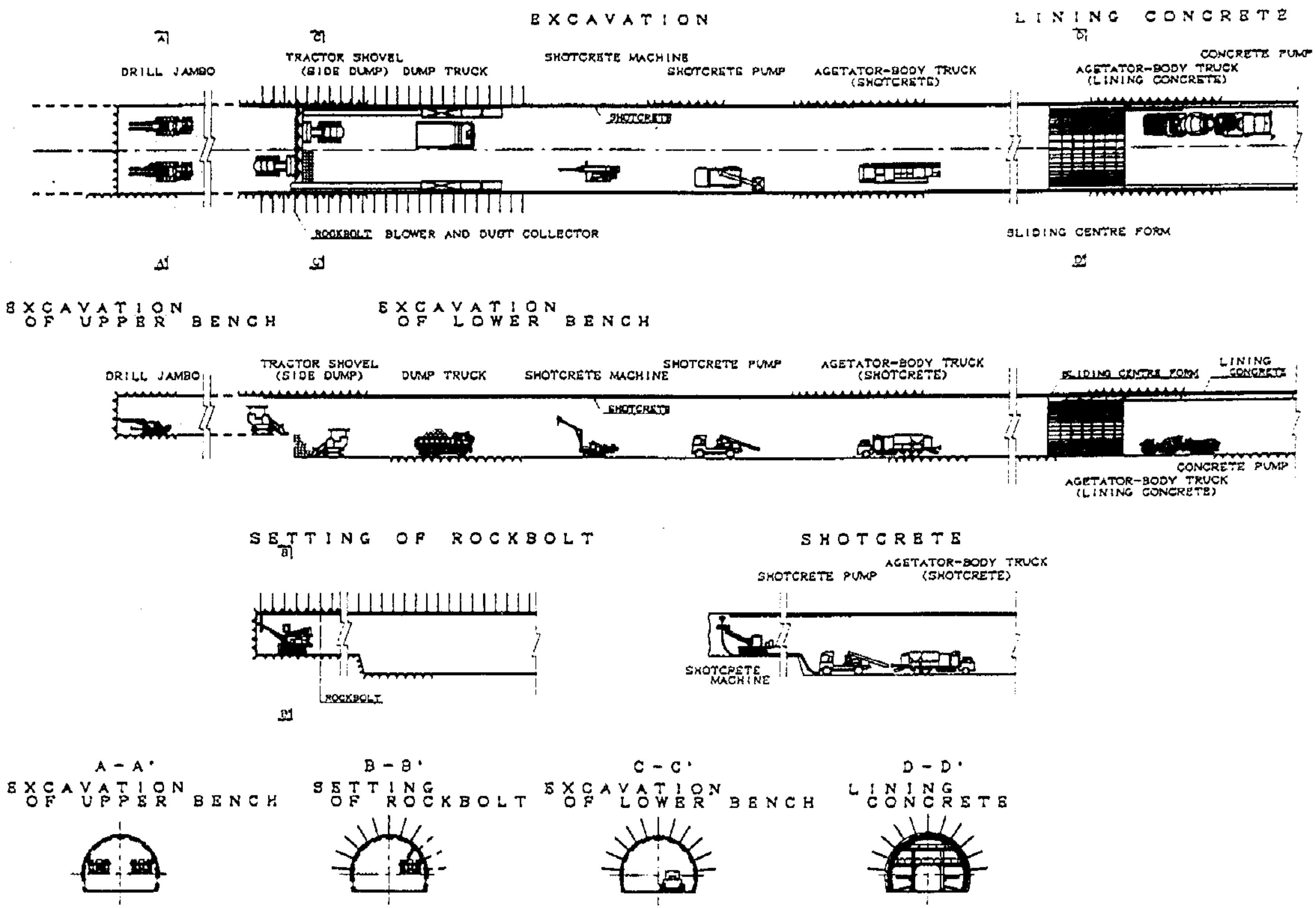


Figure 7-6-2 Tunnel Construction Sequence