3.4 Geological Investigation

3.4.1 General

(1) Objectives of this case study

Suitable methods and procedures were used onsite for the geological investigation and instrumentation of the road slope in Malaysia. The study team selected three representative sites to study the stability of the EW highway. The objectives of the study are as follows.

1) To propose a Guide to Site Investigation and Instrumentation for Slope considering the geology and topography of the national road's slope, and technical level of Malaysian consultants and boring contractors for the site investigation and instrumentation.

2) To grasp the necessary site information for countermeasure design through this case study.

(2) Workflow

The content and sequence of the case study is shown in the figure below, this chapter describes item d). The execution of c) and d) was almost foe the same period, because restriction of work schedule of this case study. It is better to plan and execute based on the foregoing survey result advanced, such as a) - b) - c) - d).

Phase	Work	Period	Relevant Chapters
	a) Preliminary Slope Inspection by study team. Planning case study.	2000/Dec. to 2001/Jan.	Chapter-2 Slope Inspection / Guide to Slope Inspection
Case Study	↓		
Phase 1 Study	b) Execution of topographic survey	2001/May to June	Chapter-4 Other Field Work 4.1 Topographic Survey
	↓		
	c) Detailed geological reconnaissance by engineering geologist and mapping to show the conditions of deformed place, outcrops, etc.	2001/June to July	Guide to Site Investigation and Instrumentation for Slope
	↓		
Phase 2 Study	d) Execution of boring investigation, *geophysical survey and instrumentation.*	2001/June to July	Chapter-3 Geological Investigation and Instrumentation / Guide to Site Investigation and Instrumentation for Slope

Phasing of the study is explained in Guide to Site Investigation and Instrumentation for Slope.

The attention points on applying several investigation and instrumentation methods for the road slope in Malaysia and the geological information to be cleared by this case study results are given in this chapter.

(3) Selection of the case study

The EW Highway is about 117km long and crosses mountainous terrain with difference of elevation (relief) exceeding 1000m. Most of the routes lie adjacent to rock slopes.

From the phase I study, the following locations were selected for detail study of collapse, landslide and embankment.

CH 27.0km - Collapse

CH 30.3km - Embankment

CH 81.3km - Landslide

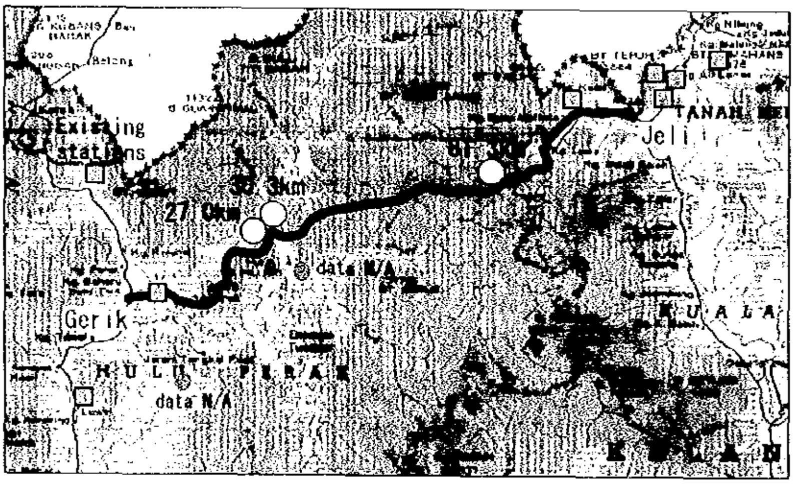
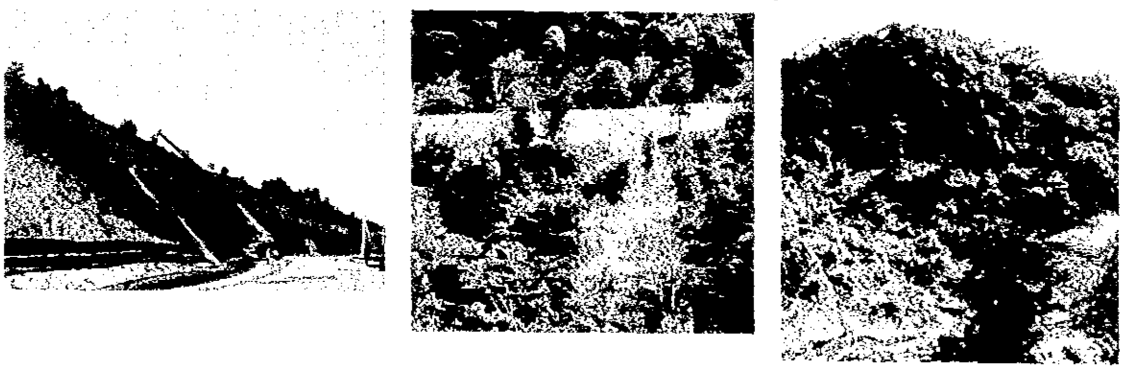


Figure 3.4.1 Location of Case Study



Ch27.0km - Collapse

Ch30.3km - Embankment

Ch81.3km - Landslide

Figure 3.4.2 Site Photos

3.4.2 Method of Geological Investigation

The purpose of the phase 2 studies is to provide adequate data and economical design of countermeasure road slope. Achievement of the geological investigation will minimize the total cost of road slope management project.

A standard investigation method which is effective, fast, economical and needs no special technique and equipment was desired, considering a lot of high-risk road's slopes nation wide.

Thus in this case study, the boring investigation and the geophysical survey employed popular methods in Malaysia.

(1) Investigation Boring

<Purpose>

The purpose of the boring investigation was to estimate the scale, the area, and the mechanism of slope failure. To confirm the ground and geological information, visual observation of boring cores and samples was done by an engineering geologist. Boreholes can be used for instrumentation.

<Methods>

Boring investigation work was carried out using a hydraulic feed rotary boring machine and metal or diamond's crown bit as cutting tool that is attached at the end of the rods and core barrel or sampler (e.g. Maizer core barrel). Boring was performed to below the failure surface and into the bedrock. A rotary boring machine, which is commonly used in Malaysia was used, the tools and materials were also popular item. For basic investigation method, a 100 percentage core recovery was required on core boring. Standard penetration test was used depending on ground conditions.

(2) Geophysical Survey

Utilization of geophysical survey for the road slope investigation is expected to allow grasping the geological structure in a relatively large area and faster, and 2-dimensional survey method is effective to analyse rugged topography such as road slope.

Most of geophysical survey equipment came from outside of Malaysia; so far, it was rare case in use for surveying method of road slope maintenance case. Recently, introducing easily handling equipment with reasonable cost, now advancing computer technology produces a remarkable improving geophysical analysis.

1) Seismic Refraction Survey

Seismic refraction survey is a traditional method, to estimate the geological structure from elastic velocity model in the slope, and one of the highest application geophysical survey method for road slope.

Detail of the method is attached in Guide to Site Investigation and Instrumentation for Slope.

Subjects revealed through this case study are as follows.

a. To spend more than one month to apply for a permit to use explosives.

b. Total work schedule depends on a blaster-man who has a gunpowder license. The Explosives Control Act requires to carry the explosives from explosive warehouse in the dealer to the site each day. He must go in the early morning and return in the evening.

2) Resistivity Image Profiling

Electric survey is also a traditional method in geophysical survey, the resistivity image profiling is its developed derivatives, analysing two-dimensional image profiling technique using topographic reduction i.e. FEM is popular to use in computerization now. Resistivity image profiling can be used to estimate the geological structure from the resistivity model in the slope, and a good application geophysical survey method for road slope.

The resistivity depends on the proper resistivity of mineral which consists of ground, consisting soil particles and void ratio of rock and groundwater conditions. Therefore, the resistivity is not identified one-to-one correspondence to the stiffness of ground as the elastic velocity of refraction survey. The interpretation of resistivity should consider the geological geophysics of the ground and the groundwater.

In this case study three sites, mainly sedimentary rocks as mudstone or metamorphic rocks as schist laid, low resistivity means high consolidation or concretion, high resistivity means high void and loosened ground. Low resistivity often shows clayed ground or high groundwater level.

Detail of the method is attached in Guide to Site Investigation and Instrumentation for Slope.

Subjects revealed through this case study as follows;

a. The accuracy of measurement is influenced by a buried power cable laid along the road crossing the measuring line.

b. There are few executable firms for this resistivity image profiling method in Malaysia, so more geophysical engineer are needed.

3) Geological Investigation Results

Results of geological investigations for this case study are summarized from geological mapping, borehole logs, geological cross sections and geophysical survey models for each study locations, and further detailed data are shown in Geological investigation report.

The quantity of geological investigations and instrumentations are summarized in Table below.

Table 3.4.1 Summary of Quantities for Geological Investigation and Instrumentation

	Item	Unit	CH 27.0km		CH 30.3km				CH 81.3km						Total
			BH-1	Total	BH-1	BH-2	BH-3	Total	BH-1	BH-2	BH-3	BH-4	BH-5	Total	Total
Boring Investigation	SPT	no.	17	17	61	43	35	139	18	41	32	24	42	157	313
	Coring	m	6.3	6.3	3.5	0	3.0	6.5	15.0	3.3	8.5	7.7	0	34.5	47.3
	Total	m	15.0	15.0	35.0	24.0	31.0	90.0	25.0	24.0	25.0	20.0	21.0	115.0	220.0
Geophysical survey	Seismic	m	1 line	200.0	0 line			0.0	2 line x 230m					460.0	660.0
	Resistivity	m	1 line	200.0	1 line			200.0	2 line x 230m					460.0	860.0
Instrumentation	Inclinometer	m	15.0	15.0	0.0	25.0	0.0	25.0			0.0	35.0	0.0	35.0	75.0
	Piezometer	m	0.0	0.0	20.0	0.0	0.0	20.0			15.0	0.0	15.0	30.0	50.0
	Rain Gauge	no.		1				1						1	3
	Extensometer	no.		1				0						2	3
Sound-ing	Dynamic cone penetration test	no		4				0						6	10

3.4.3 Landslide (CH 81.3km)

This slope is located on the left-hand side of EW Highway and at present, there is a 150m long rock gabion already constructed at the toe from approximately CH 81.2km to CH 81.35km.

The results are summarised as follows;

1) Landslide mass consist of several active small blocks as shown in Figure 3.4.2. When the slope inspection was carried out, block-A movement was considered more important to the road, so the study program was planned along main section A-A'. As the study progressed, the behavior of main landslide mass is also considered important, so the program of boring investigation was modified to do additional borings BH-4 and BH-5 in section B-B' (CH81.28km). The geophysical survey was executed in section A-A' (CH 81.33km) and section C-C' (CH 81.30km).

2) The landslide surface is estimated as line A-A' in Figure 3.4.4 and as line B-B' in Figure 3.4.5. Estimated sliding surface A-A' and b-e-b' is occurring the zone of shearing failure in the rock, that zone is classified highly weathered class-V, the movement and velocity is small, since lowering of the groundwater level now.

3) A line A-A' in Figure 3.4.4 and a line B-B' in Figure 3.4.5, are estimated talus deposits surface that SPT N-value is relatively low (less than 10blows/450mm), from ground level to depths of about 6.0m for the following boreholes: BH-1, BH-2, BH-4, and BH-5. The materials generally consist of silty CLAY and clayey SILT with some gravel including completely weathered rocks.

4) Survey result of elastic velocity in each stratum is given in the following Table 3.4.2.

Table 3.4.2 Elastic Velocity Stratum Summary (Site: CH 81.3km)

Elastic Velocity Stratum	Range of Velocity	Estimated Ground Condition
First stratum	0.3km/sec	Subsurface soil and very loosened ground.
Second stratum	0.4～0.5km/sec	Relatively stiff soil or cohesive soil.
Third stratum	1.0～1.2km/sec	Highly to moderate weathered rock zone. Estimating above the groundwater.
Forth stratum	1.6～1.8km/sec	Moderate to slightly weathered rock zone. Estimating below the groundwater.
Fifth stratum	3.8～3.9km/sec	Fresh and low weathered rock zone. Very hard.

5) The landslide mass might repeat movement many times under the construction of the road. It is anticipated that the movement of the sliding mass will be continued after rain or other activities, little by little and small collapses may occur from the high elevation continuously, it becomes additional source of sliding mass.

6) Based on the instrumentation results during July and August 2001, there was some rain but groundwater level is still low. The behaviour of landslide has been stable from the data of the extensometer and the inclinometer. The instrumentation should be monitored carefully in the coming rainy season.

3.4.4 Collapse (CH 27.0 km)

This cut slope located on the right-hand side of EW HIGHWAY had undergone a major slope failure.

The boring investigation was carried out on the second berm.

The results are summarised as follows;

(1) The possibility of further expansion of the existing collapsed area studied by the boring investigation and the geophysical survey on the main study section A-A' shown in Figure 3.4.11.

(2) The countermeasure design against the existing collapsed area has already done by JKR, employing the retaining wall. The construction work has also started since end of June 2001.

(3) The boring investigation results show that the rock-type consists of interstratified sandstone and shale. The Standard Penetration Test (SPT) N-value generally shows a relatively high values ranging from 22 to more than 100. The materials generally consist of silty SAND and sandy SILT derived from the weathering of sandstone and shale respectively.

(4) Survey result of elastic velocity in each stratum is given in the following table:

Table 3.4.3 Elastic Velocity Stratum Summary (Site: CH 27.0km)

Elastic Velocity Stratum	Range of Velocity	Estimated Ground Condition
First stratum	0.3km/sec	Subsurface soil and very loosened ground.
Second stratum	0.4～0.8km/sec	Completely to Highly weathered rock zone. Fractured zone.
Third stratum	1.0～1.6km/sec	Highly to moderate weathered rock zone. Estimating above the groundwater.
Forth stratum	1.8～2.2km/sec	Moderate to slightly weathered rock zone. Estimating below the groundwater.
Fifth stratum	3.0～3.2km/sec	Fresh and low weathered rock zone. Very hard.

(5) The site feature around CH 26km to CH 27km is suspected to be the topography of a natural landslide by the photographic interpretation. The road and cut slope is crossing on the top of the landslide topography. The construction of the road by cutting on the natural slope is made to increase the stability of the natural landslide.

(6) A drastic countermeasure should be taken for the slope since the risk of collapse is the same as neighbouring collapse area. The estimated volume of collapse is around 5m deep at boring location on the study section shown in Figure 3.4.11.

3.4.5 Embankment (CH 30.3 km)

The failure at this location is confined to the left-hand side of EW HIGHWAY.

There are three (3) boreholes located here, namely BH-1 and BH-2 (both at km 30.32) and BH-3 (km 30.38).

The results are summarised as follows;

(1) The countermeasure has been executed by JKR since 1999 to January 2001 to the top parts of the embankment that was employed the retaining wall with piling and the fill's re-compaction with geo-textile method, because the road surface had settled around 1998. The purpose of this case study is to grasp the possibilities of further failure, which may be seepage failure, presence of weak zone in the lower part of the embankment, etc.

(2) No significant weak zone in the embankment that most of fill was compacted more than 20 N-value encountered through this boring investigation. Result of resistivity imaging profile means that the slope surface (reddish zone) is very dry, loose or

low density. Based on the instrumentation records during July to August 2001, the groundwater stayed low level in the small rain. The behaviour of inclinometer shows a small lateral displacement downward. The estimated collapsible mechanism is a progressive failure from lower erosion to upper part during rain.

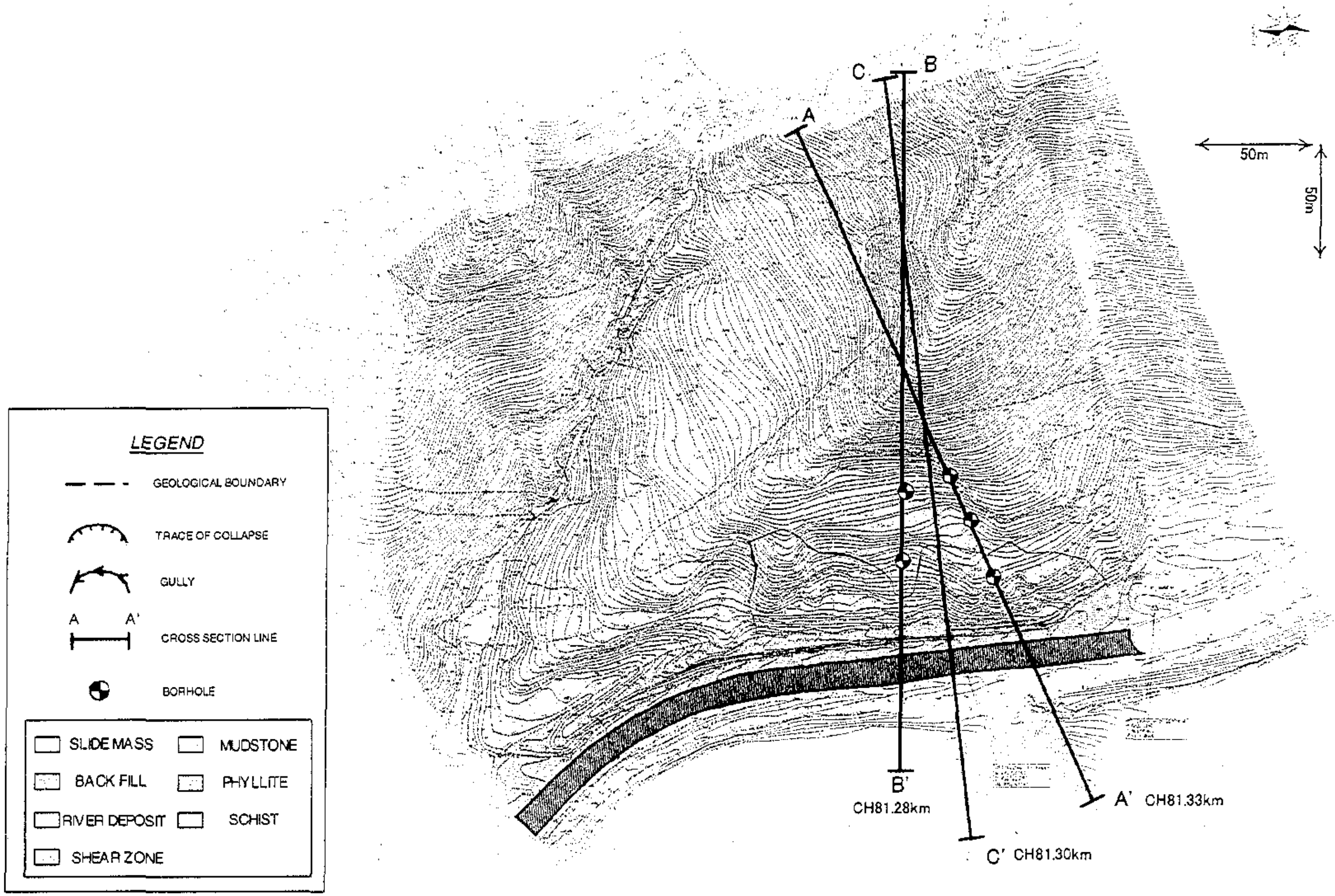


Figure 3.4.3 Geological Map at CH81.33km

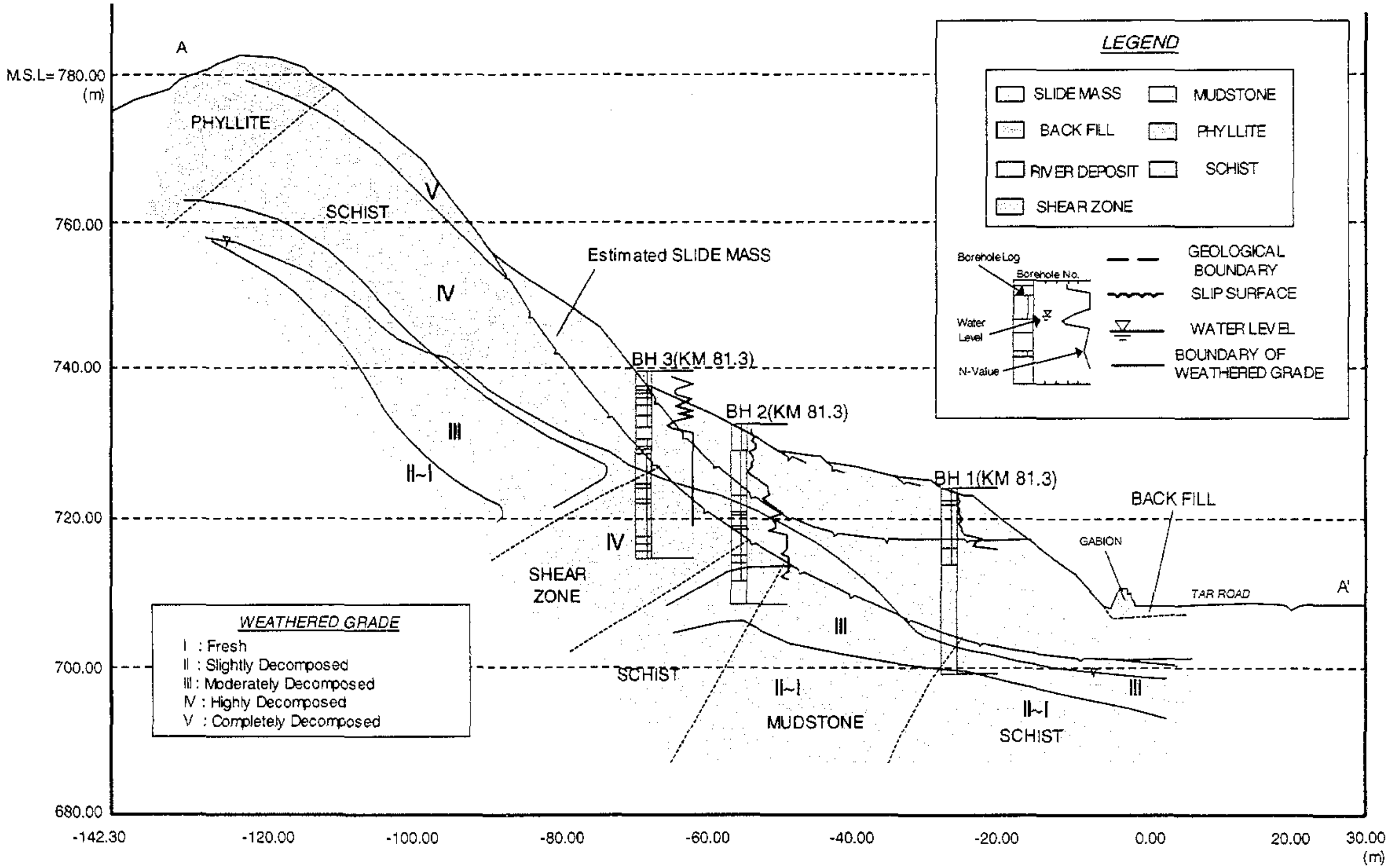


Figure 3.4.4 Geological Cross Section A-A': CH81.33km

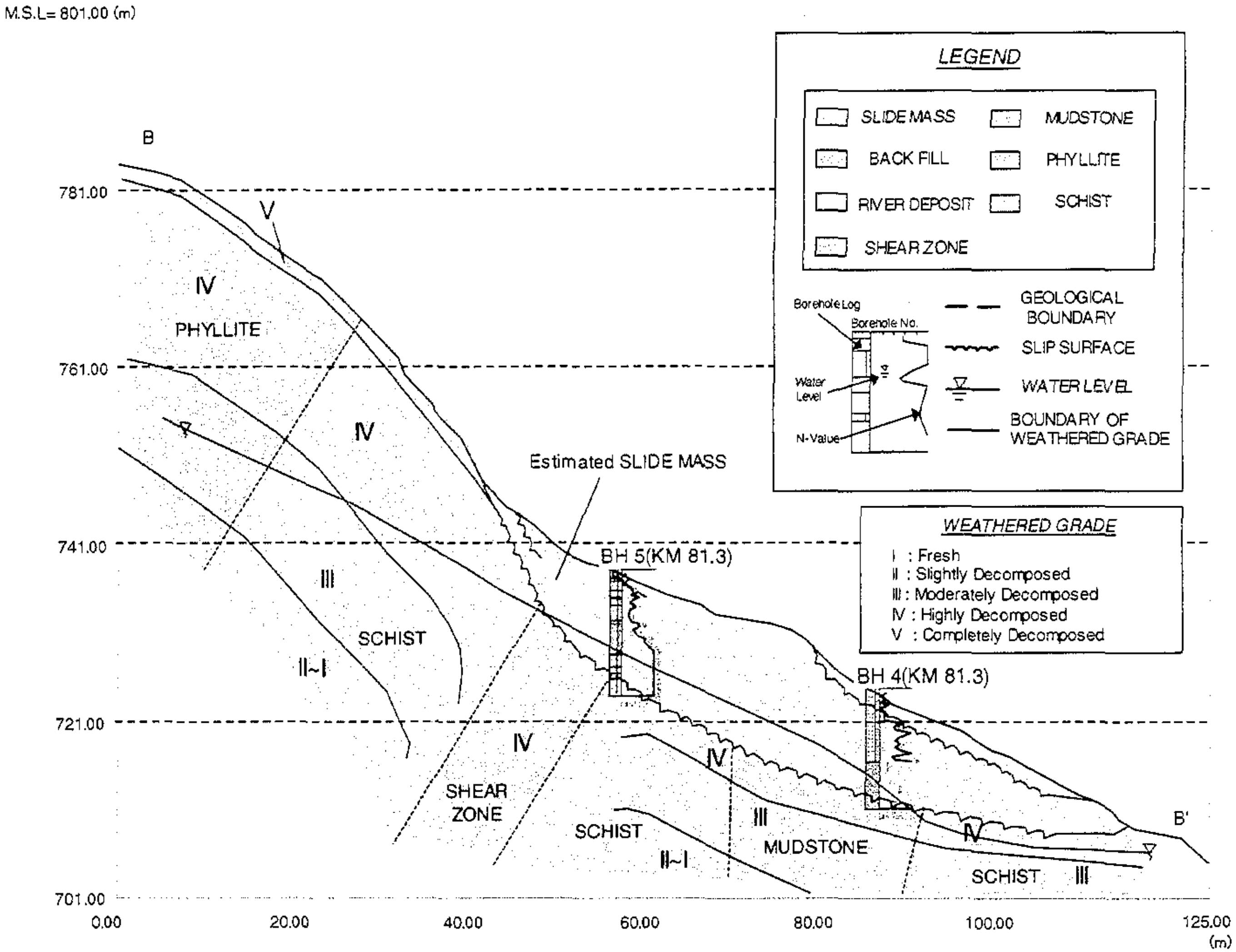


Figure 3.4.5 Geological Cross Section B-B': CH81.28km

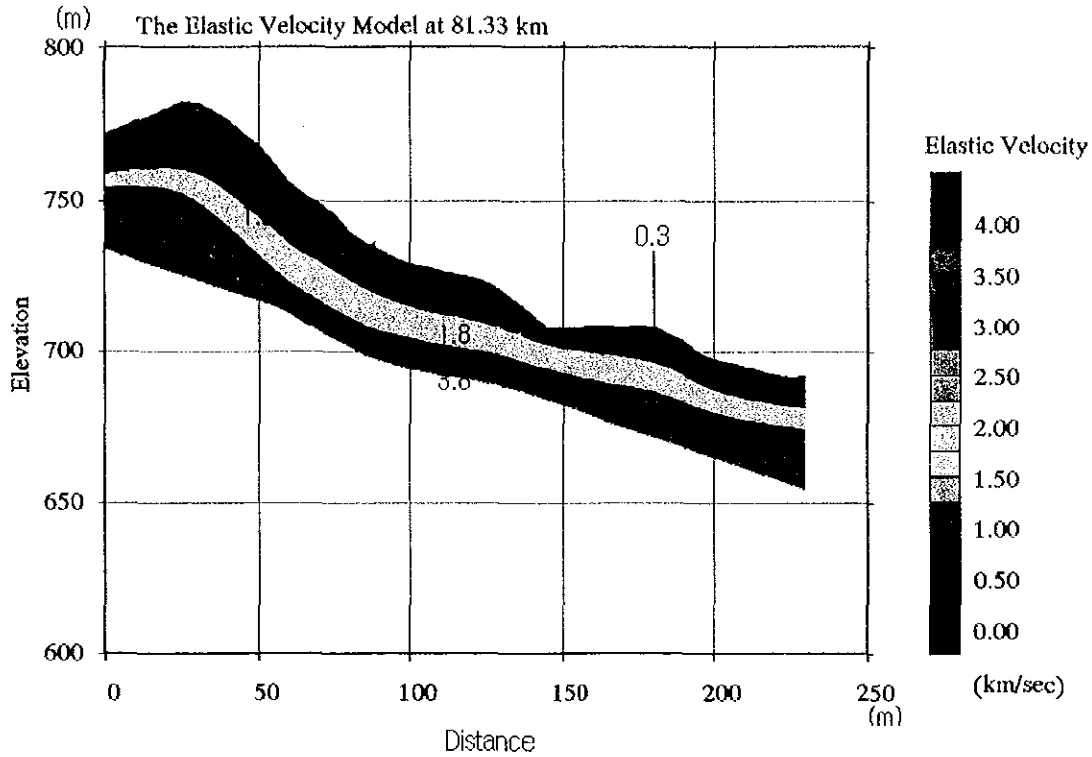


Figure 3.4.6 Elastic Velocity Model Section A-A': 81.33km

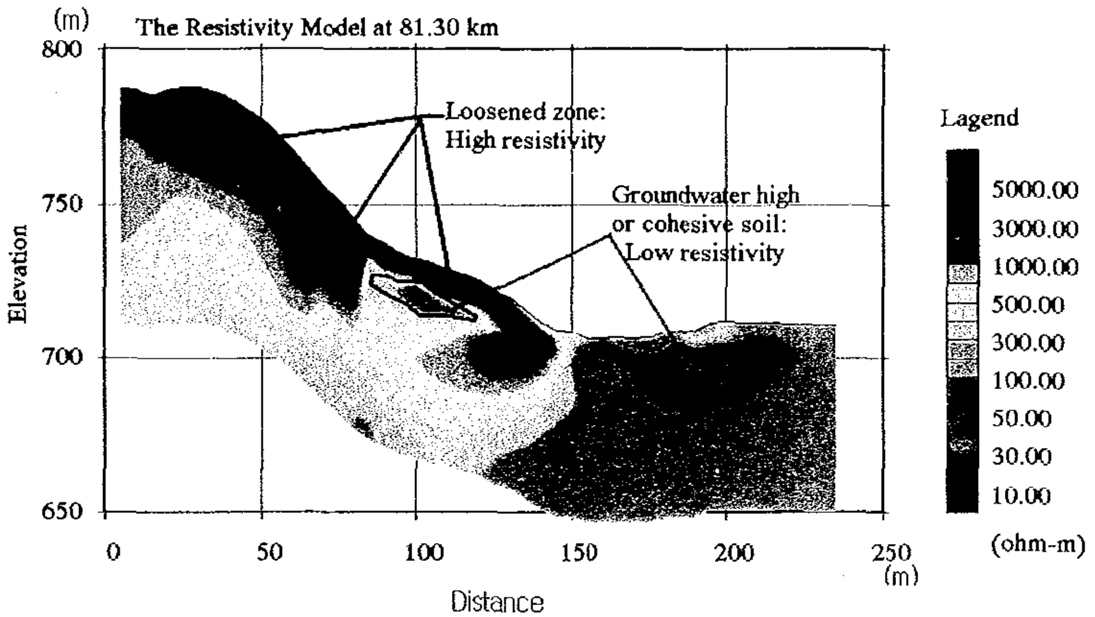


Figure 3.4.7 Resistivity Imaging Model Section A-A': 81.33km

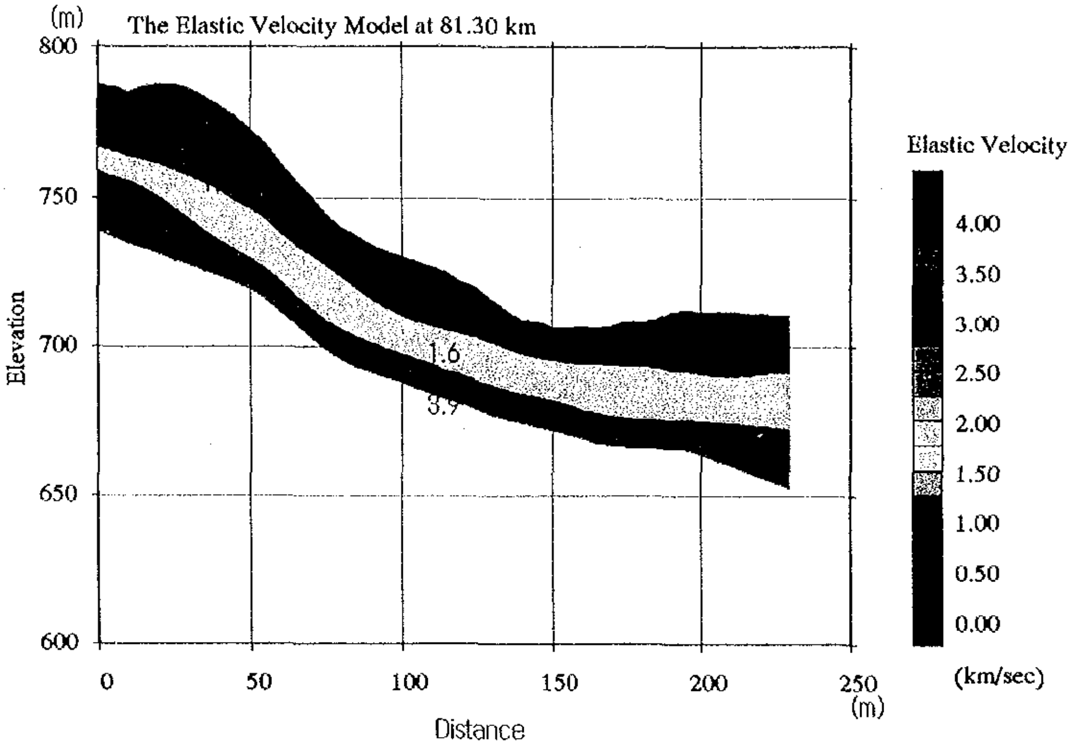


Figure 3.4.8 Elastic Velocity Model Section C-C': 81.30km

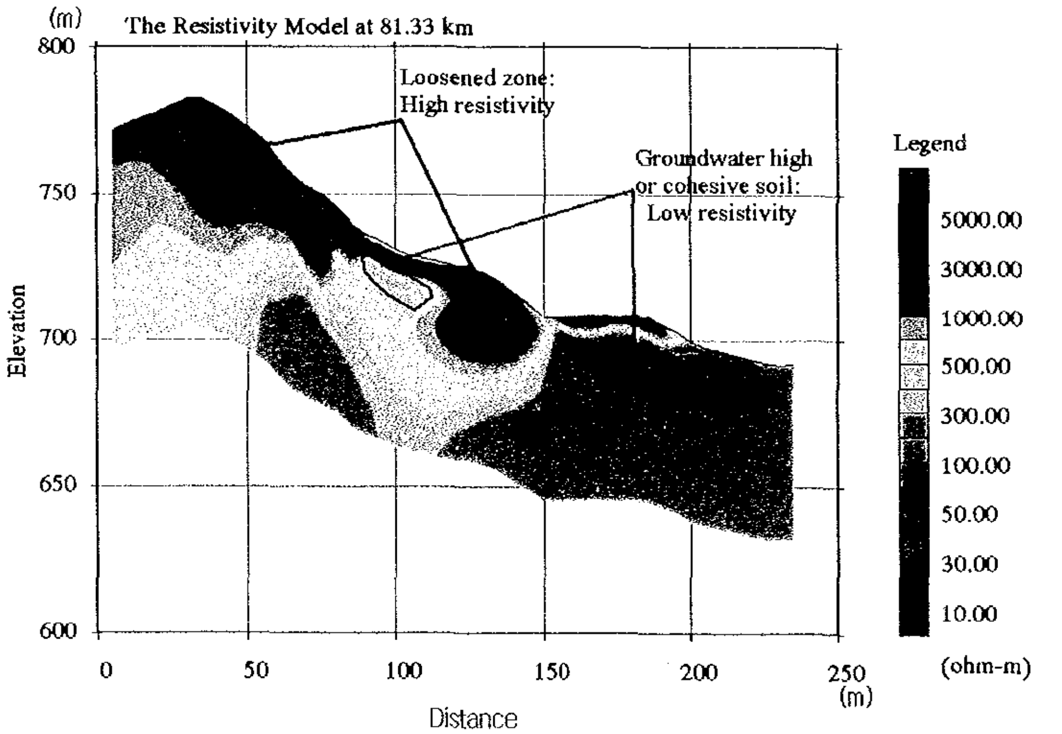


Figure 3.4.9 Resistivity Imaging Model Section C-C': 81.30km

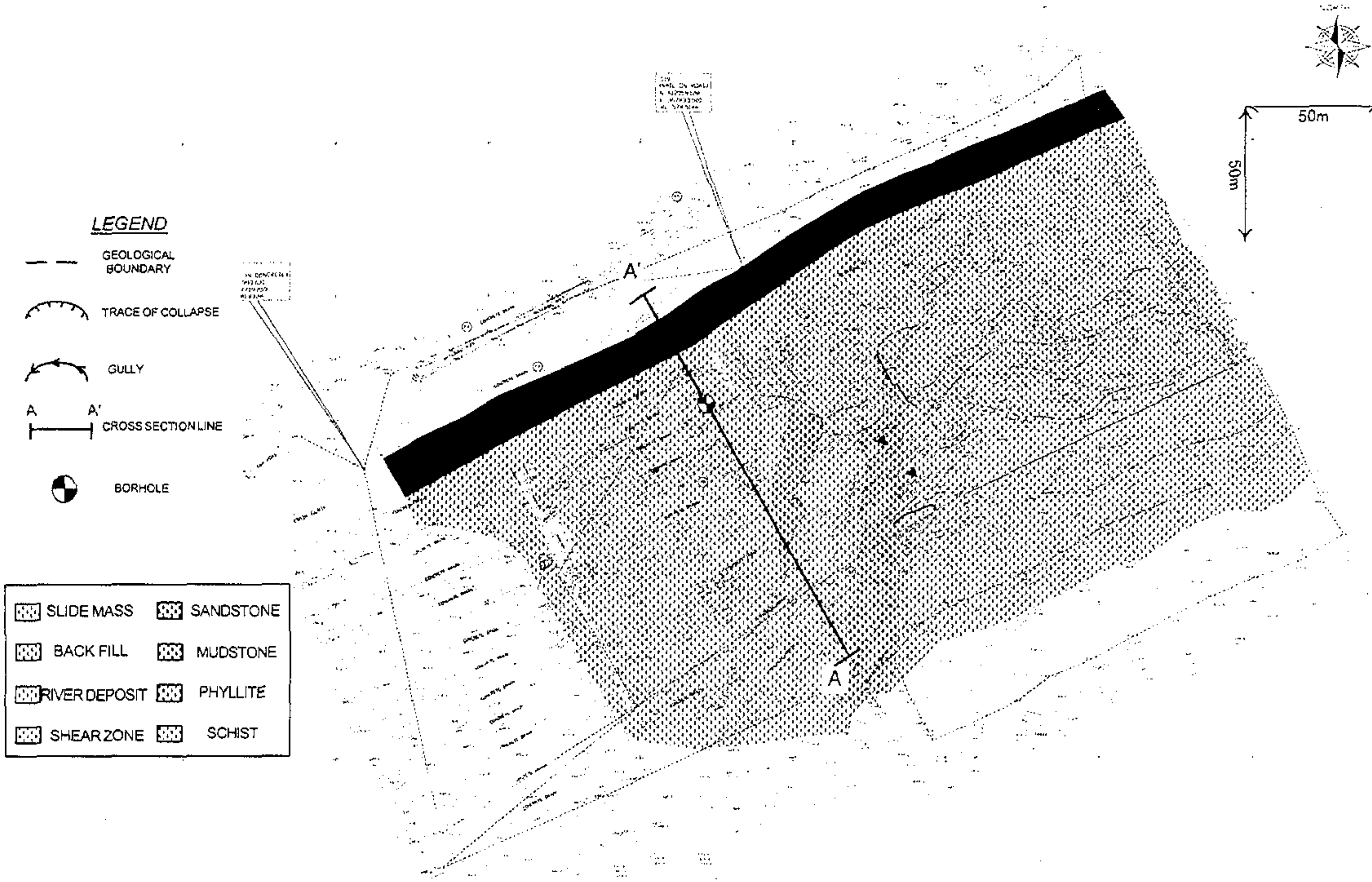


Figure 3.4.10 Geological Map at: CH27.00km

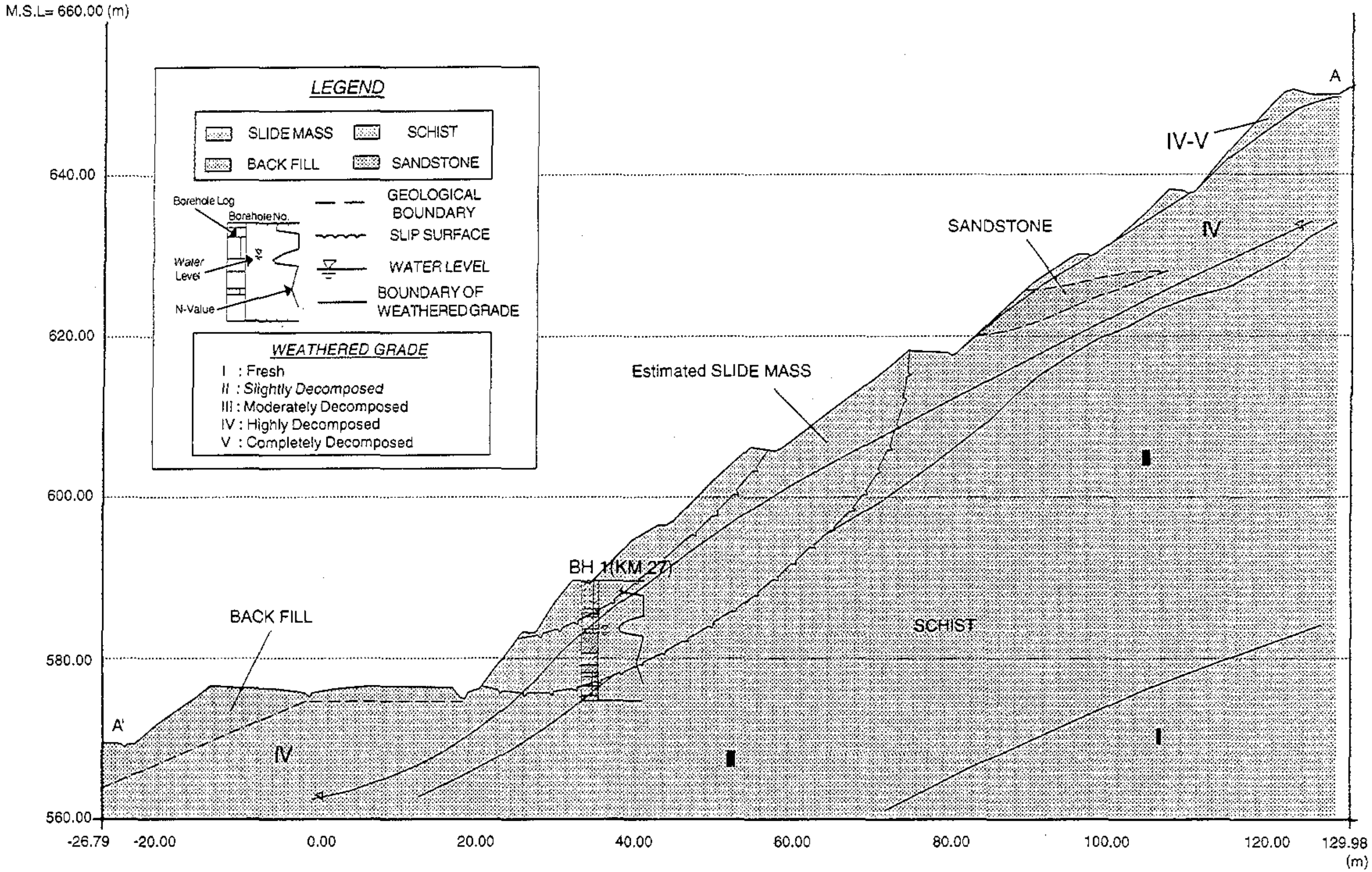


Figure 3.4.11 Geological Cross Section A-A': CH27.00km

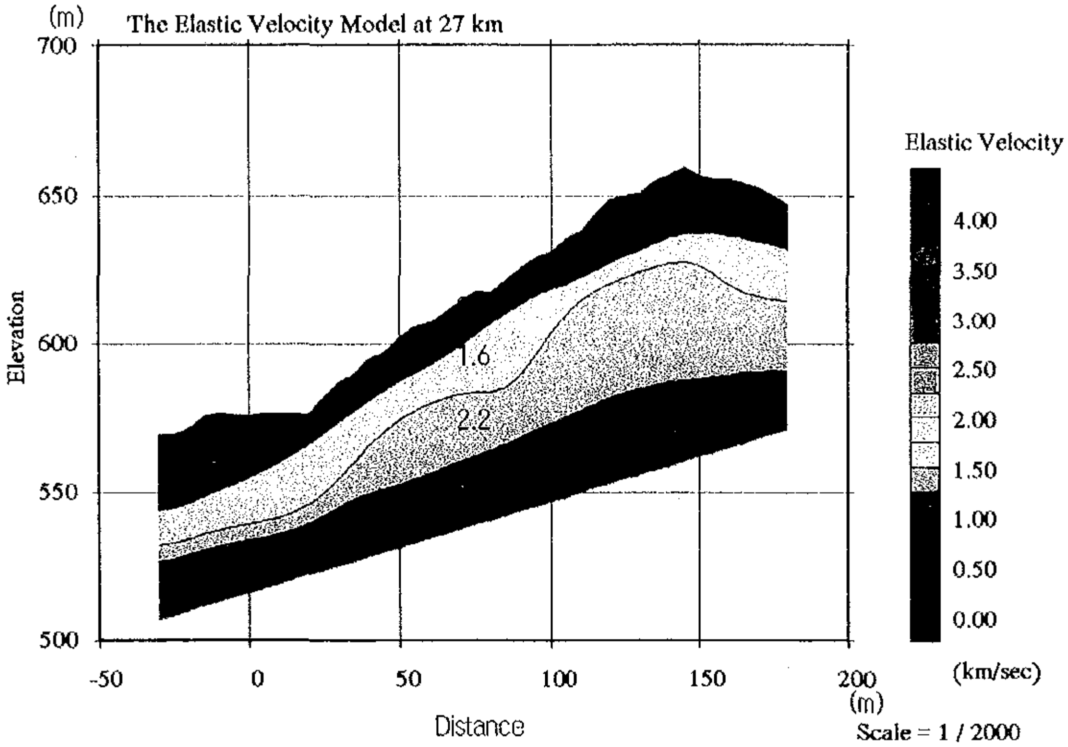


Figure 3.4.12 Elastic Velocity Model Section A-A': 26.95km

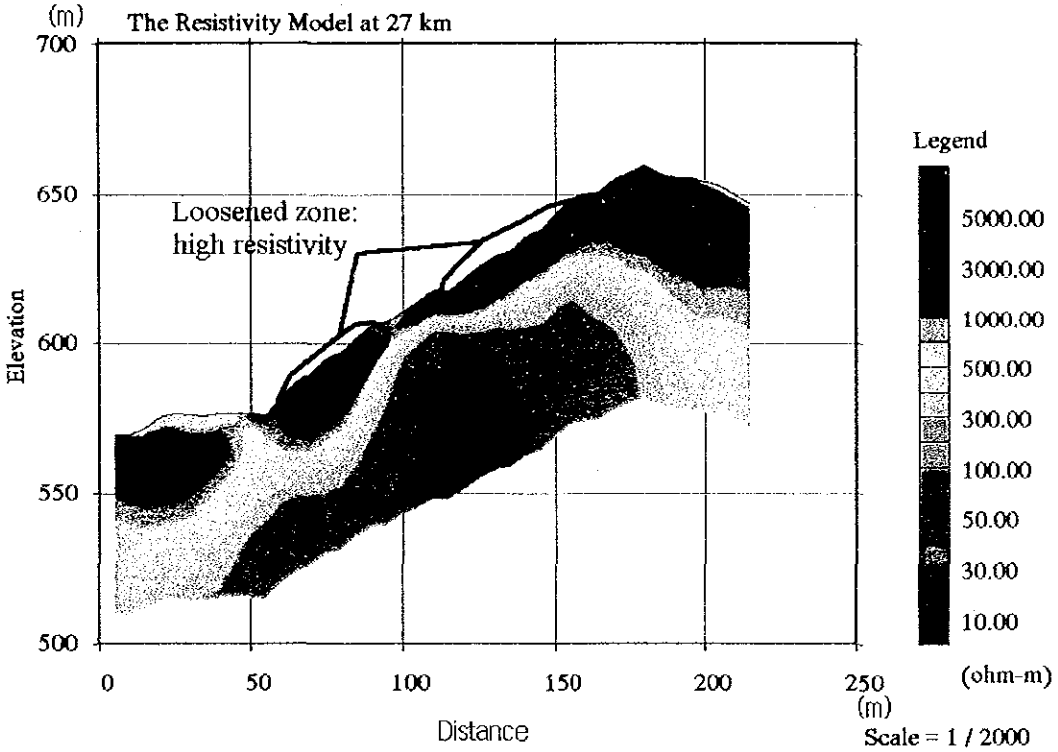


Figure 3.4.13 Resistivity Imaging Model Section A-A': 26.95km

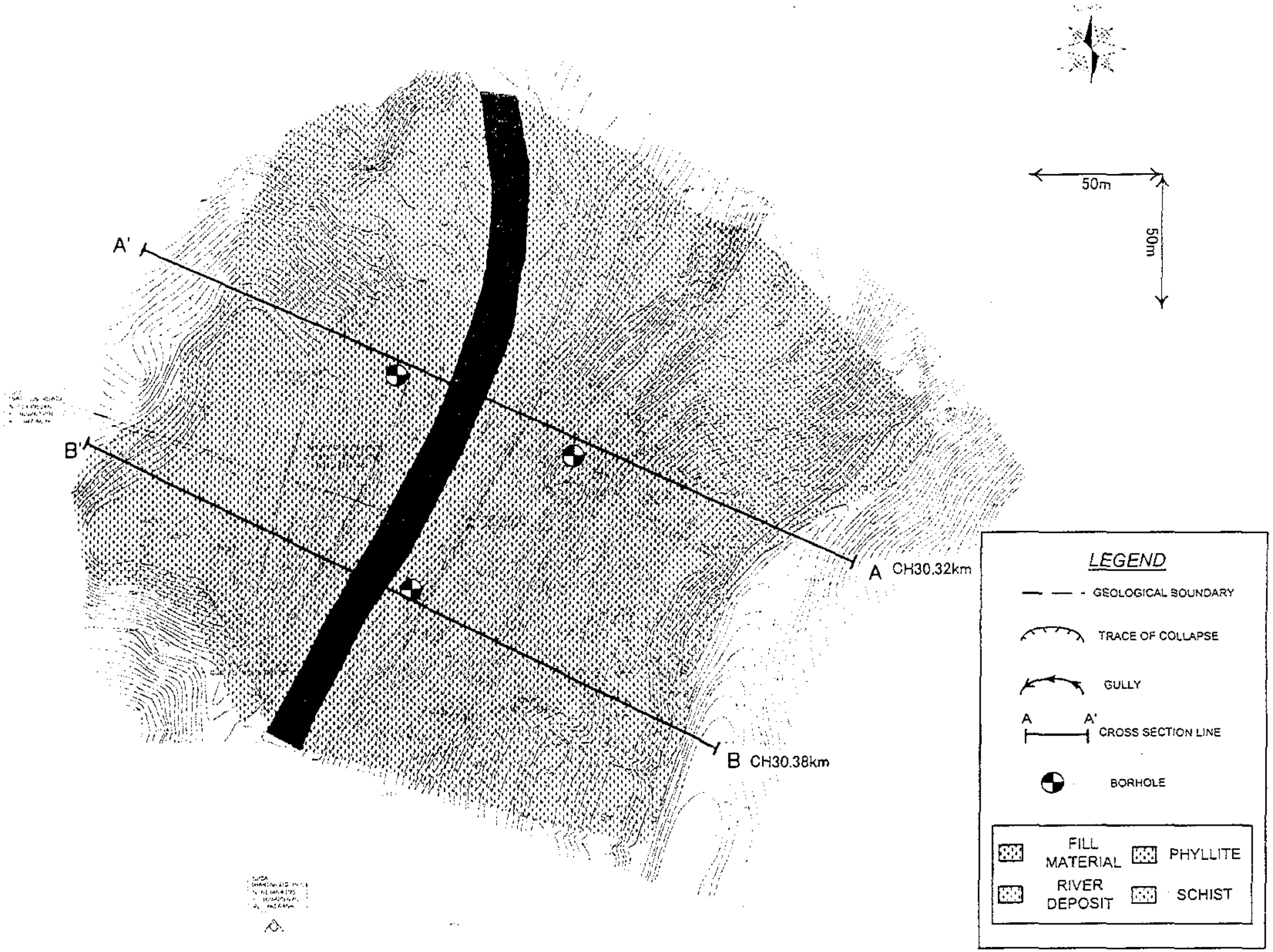


Figure 3.4.14 Geological Mapat: CH81.30km

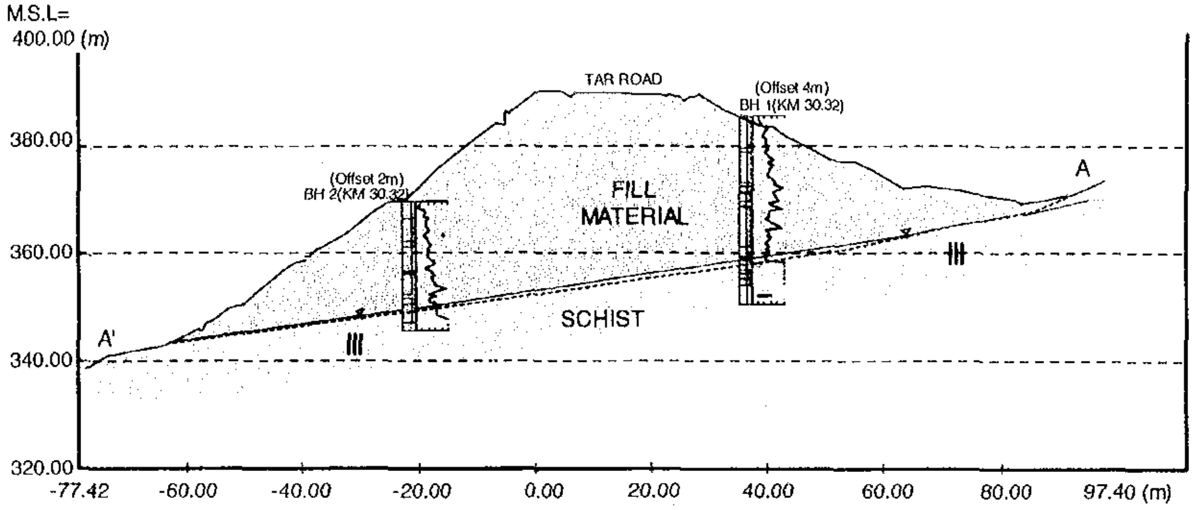


Figure 3.4.15 Geological cross section A-A': CH30.32km

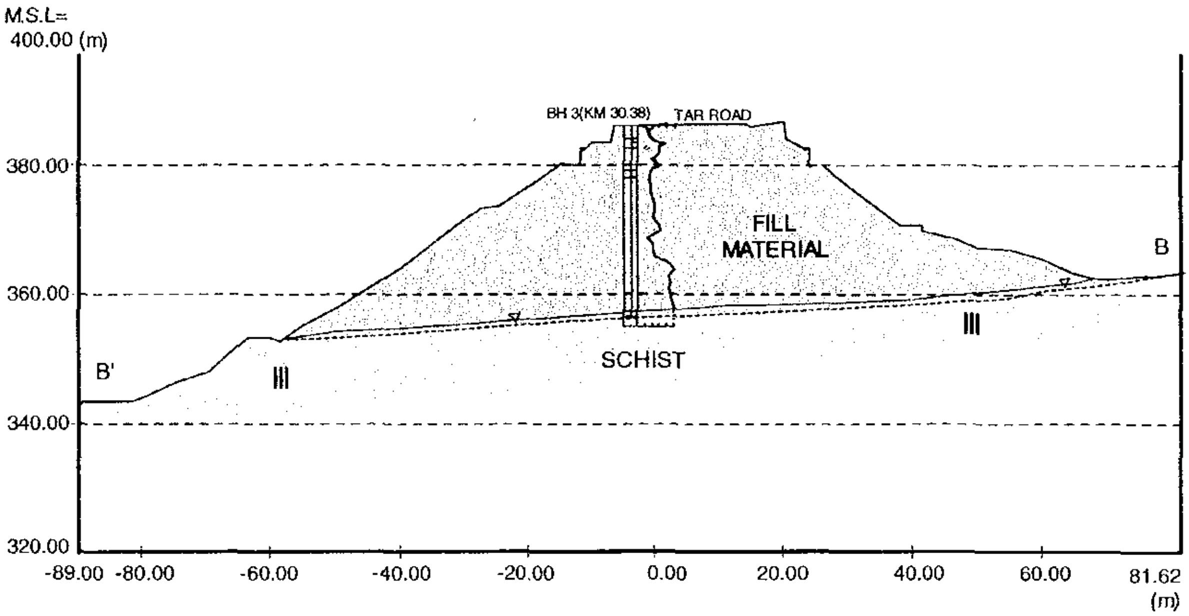
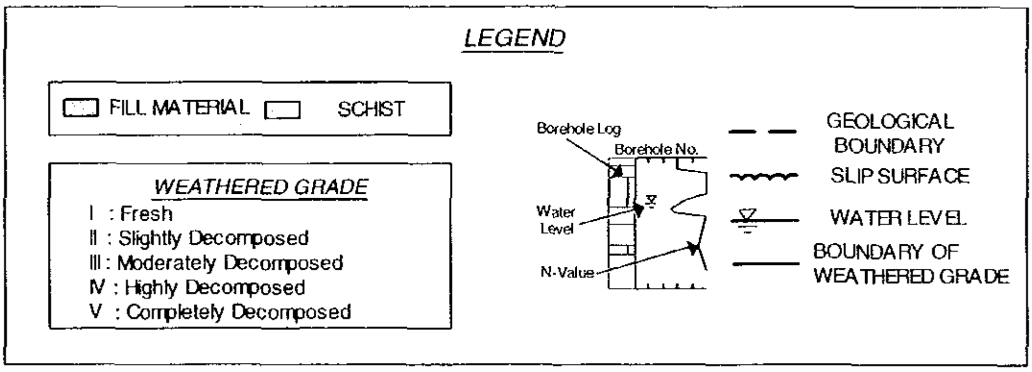


Figure 3.4.16 Geological Cross Section B-B': CH30.38km



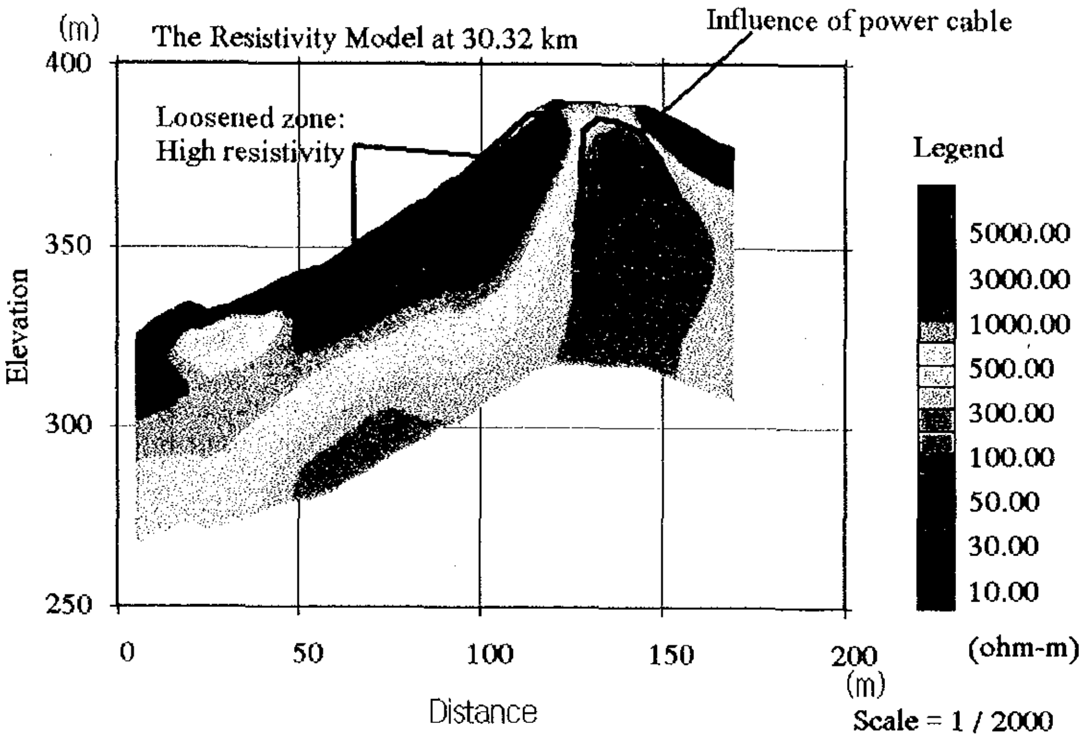


Figure 3.4.17 Resistivity Imaging Model Section A-A': CH30.32km

3.5 Instrumentation and Monitoring

The purpose of the instrumentation in this case study is to observe the behaviour of slope, rainfall, displacement of surface and ground and groundwater. Rain gauge, inclinometer, wire extensometer and VW piezometer were installed at each site as shown in Table 3.5.1.

Table 3.5.1 Summary of instrumentation

Location / Instrumentation	81.3km	27.0km	30.4km
Rain gauge	1	1	1
Inclinometer	2 (BH-2, BH-4)	1 (BH-1)	1 (BH-2)
Wire extensometer	2	1	0
VW piezometer	2 (BH-1, BH-3)	0	1 (BH-1)
Water standpipe	1 (BH-5)	0	1 (BH-3)

3.5.1 Installation

Four (4) types of instruments were installed in this project:

(1) Inclinometer: To measure ground movement in the slope and the location of movement with respect to the vertical depths.

(2) Piezometer: To measure the porewater pressure due to ground water.

(3) Extensometer: To measure the surface movement of the slope.

(4) Rain Gauge: To measure the rainfall at the each site.

After the instruments have been installed, measuring will carry out for a period of eighteen (18) weeks.

(1) Inclinometer Casing Installation

This instrument measures the ground movement in slopes. After termination of boring, the hole is filled with cement/bentonite mixed fluid to form from the bottom using grouting hose. This is to ensure that inclinometer casing-pipe reflects to the ground movement. It must be installed where is no void around the casing-pipe. If the cement/bentonite mixed fluid sunk after itself concreted in the borehole, this grouting work must be filled up again up to the ground surface.

(2) Vibrating Wire Piezometer Installation

Two types of groundwater investigation instrument were installed in the project site. The first type was a vibrating wire piezometer that comes with a mini data logger that can be used to record reading automatically. The second was a water standpipe that requires manual measurement.

The purpose of the piezometer is to monitor the pore water pressure in ground for the effective earth pressure.

After boring, 20cm of bentonite plug was first used to seal the lower part of the borehole. The piezometer pipe with a porous section at the bottom was inserted into the borehole. The porous part was subsequently covered with 0.5m of sand and followed by another 20cm of bentonite plug. Finally, the hole was backfilled to the surface. Installation procedure is shown in Figure 3.5.1.

Installing Piezometer in Borehole

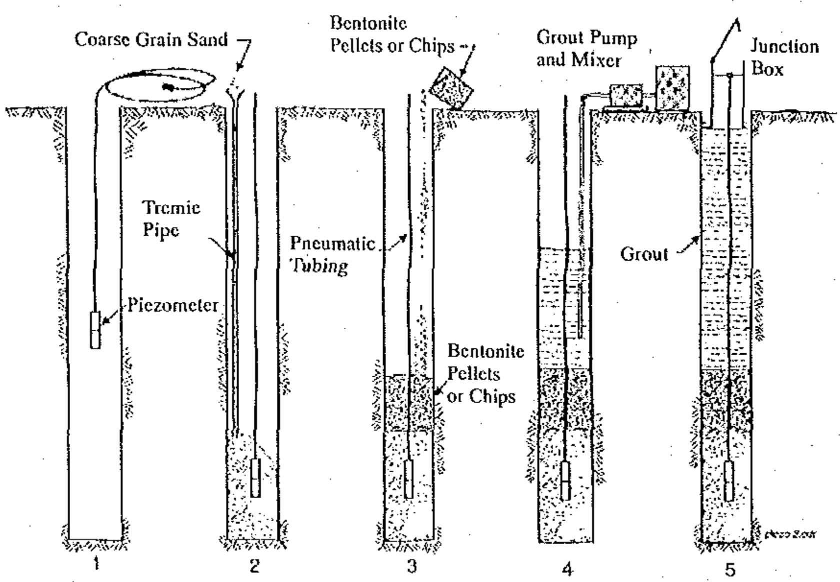


Figure 3.5.1 VW Piezometer installation procedures

Procedures:

i) Saturate filter and piezometer. Flush borehole with clean water. Check that piezometer functions correctly and note "zero" readings of VWP. Lower piezometer into borehole to required depth.

ii) Place sand around tip. Sand should be wet to easily sink to the bottom of borehole. Check that piezometer functions.

iii) Place bentonite seal. Drop pellets or chips into borehole slowly to avoid bridging. Add water as needed for hydration of betonite. Allow time for bentonite to swell.

iv) Fill remainder of borehole with bentonite-cement grout.

v) Top off borehole with grout and install protective cover.

(3) Water standpipe Installation

The installation procedures for the standpipe is similar to the vibrating wire type. However, the standpipe type requires a PVC pipe that allows the direct measurement of water head for the conversion to pressure.

Installing Standpipe Piezometer in Borehole

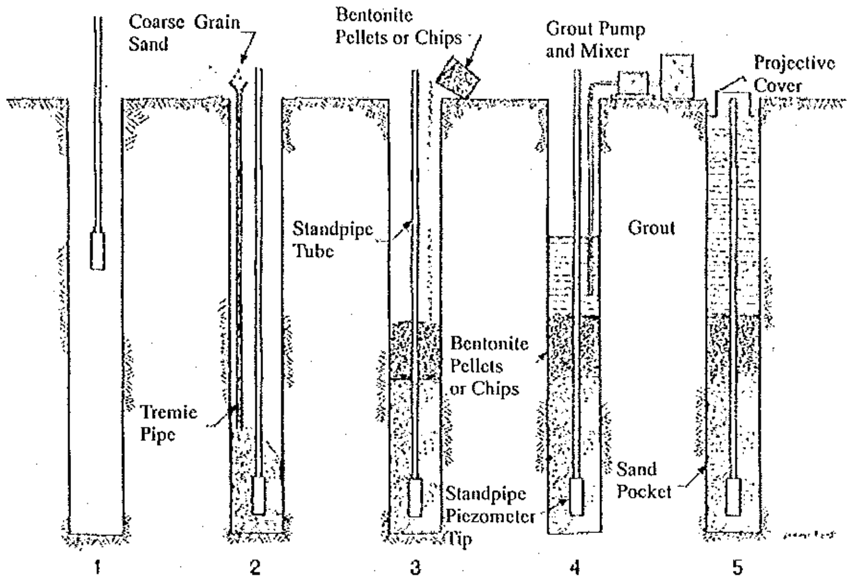


Figure 3.5.2 Water standpipe installation procedures

(4) Rain Gauge Installation

Rain gauges were installed to measure rainfall near the slope of interest. The location of installation was in open space that has no aerial obstruction. For automatic measurement, a tipping bucket automatically tips when a certain amount of precipitation accumulates inside the bucket. Total precipitation is determined by the number of tips.

The standard rain gauge is comprised of two basic elements: a funnel or standard 8-inch opening at the top and a collection device at the bottom. Precipitation measurements may be obtained by counting the number of tips after the bucket containing a fixed amount of precipitation. For the instrument at site, the accuracy is 0.2mm.

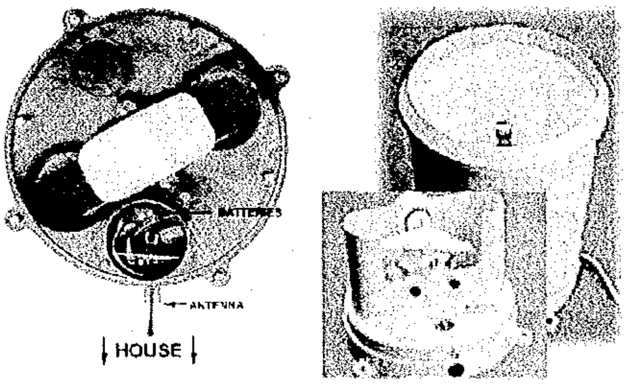


Figure 3.5.3 Rain Gauge Equipment Tipping Bucket Type

Table 3.5.2 Specification of Rain Gauge

Receiver	:	Diameter 200mm (within ±0.3mm)
Sensitivity	:	One switch closure per tip of bucket (0.2mm)
Measuring Range	:	Max. rainfall intensity, 500mm/hr
Measuring Accuracy	:	Better than 2% @ 100mm/hr
Sensor	:	Tipping bucket (material gold plated brass) with syphon to improve linearity of counting with varying rain fall intensity
Contact System	:	Dual reed switches (glass encapsulated normally open contact)
Contact Capacity	:	12VA (Max. current 0.5amp)
Contact Time	:	0.1 second

(5) Wire Extensometer

Wire extensometer detect movement of the slope at the surface. This is carried out by fixing the instrument at a place where it is most unlikely to move and putting a marker at the position where the slope is most likely to move. Connect the two places using invar line. If movement occurs at the slope, the invar line will be in tension and caused the recorder to record the movement.

Installation procedure is shown in Figure 3.5.4.

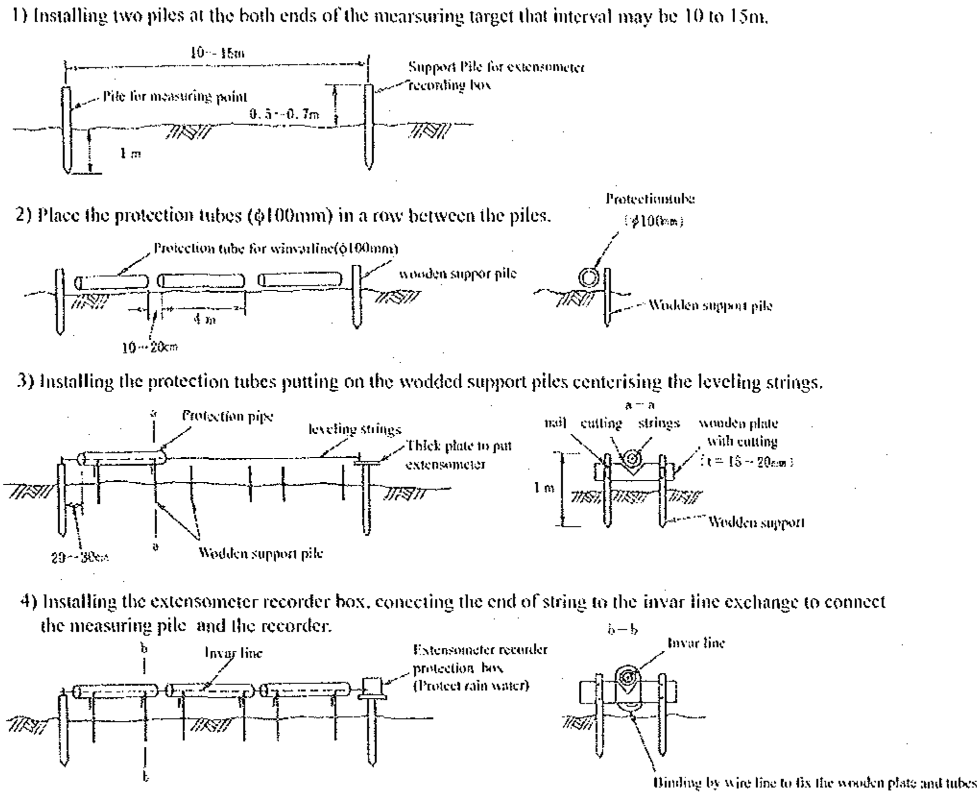


Figure 3.5.4 Wire extensometer installation procedures

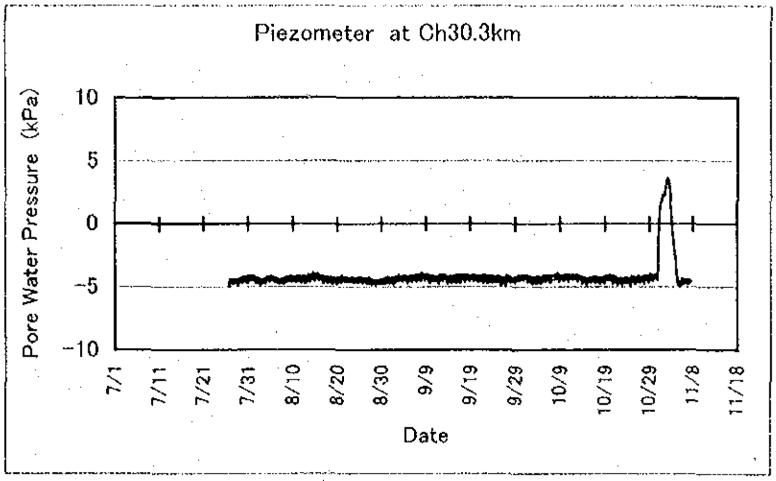
The instrument installed at the site is manufactured by Sakata Denki of Japan, Model SRL-5. The instrument records on paper to carry out continuous monitoring and plotting of the slope movement. The paper can take recordings for a period of 30 days. The drum turns at 0.25mm/h.

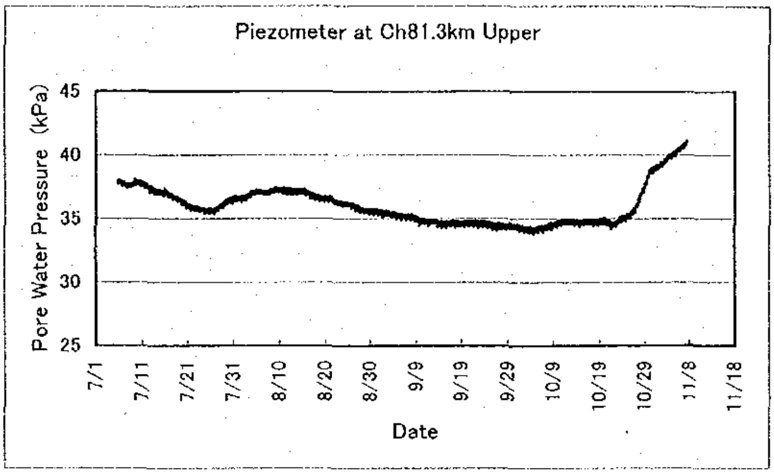
3.5.2 Monitoring Result

(1) Extensometer, Inclinometer, Piezometer

The monitoring on extensometers, inclinometers, piezometers at Ch27.0km, Ch30.2km and Ch81.3km has been monitored from July to December 2001.
The results are summarised in Figures 3.5.5 to 3.5.7.
The points of monitoring results are as follows:

i) Three Surface Wire Extensometers were installed at Ch27.0km and Ch81.3km and no clear movement has been seen at three extensometers
ii) Three Piezometers are installed at Ch27.0km and Ch81.3km. They shows high pore pressure in late October to November and low pore pressure in September.
iii) Very stable ground temperatures were monitored by thermometers in piezometers as shown on Figure 3.5.6.
iv) Inclinometers show no clear movement.





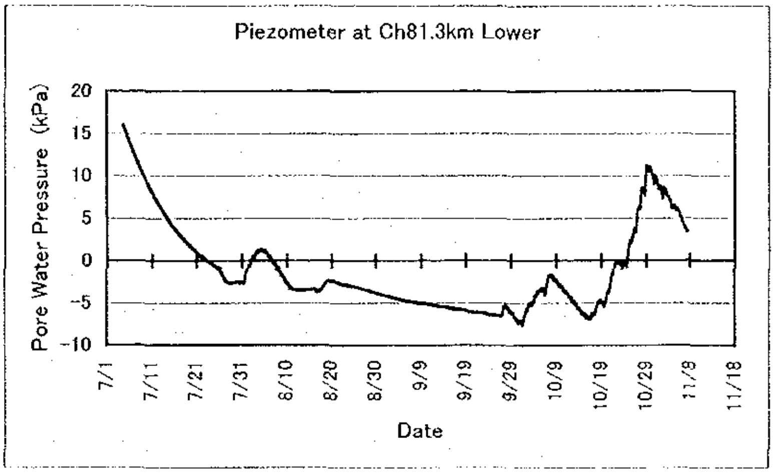
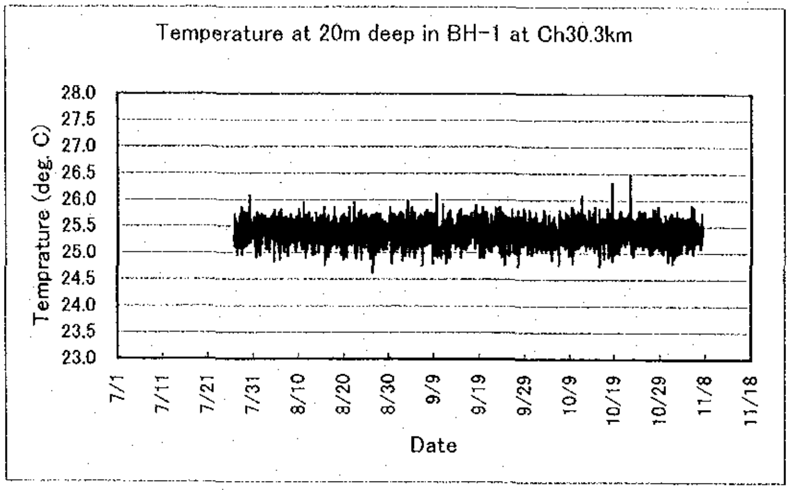
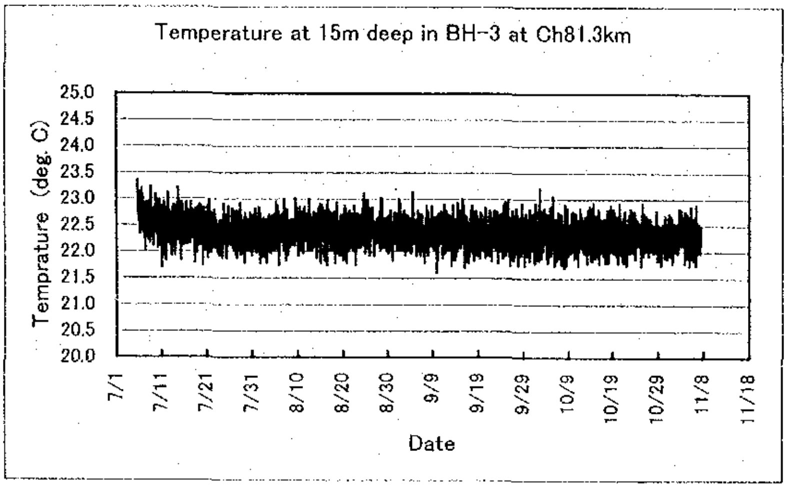


Figure 3.5.5 Piezometer Monitoring Result





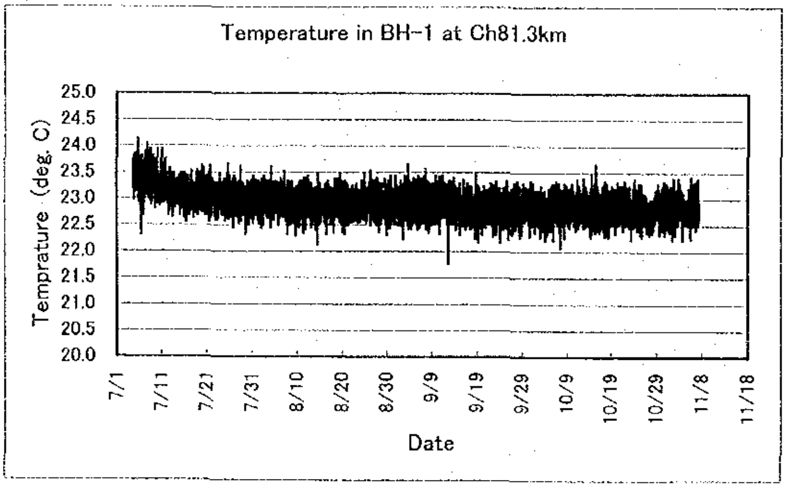
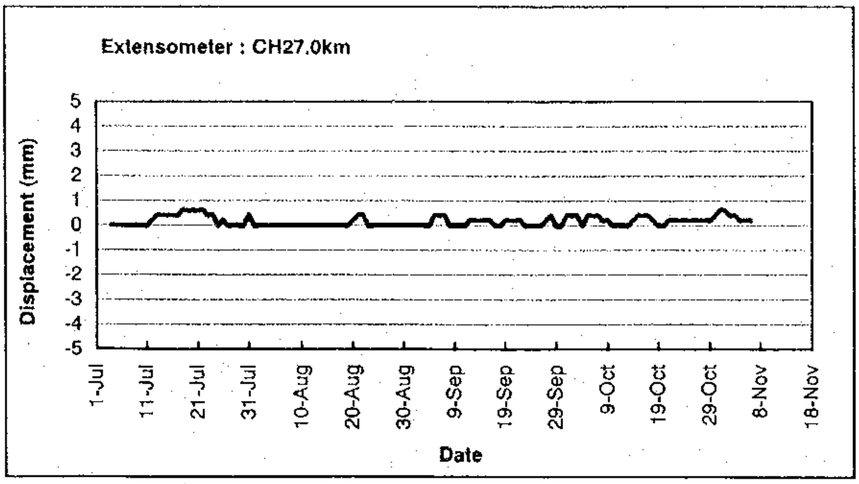
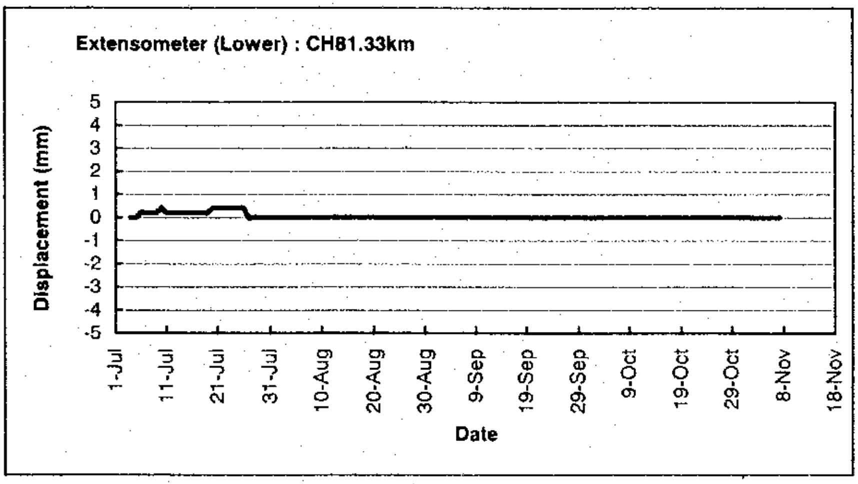


Figure 3.5.6 Temperature in Boreholes





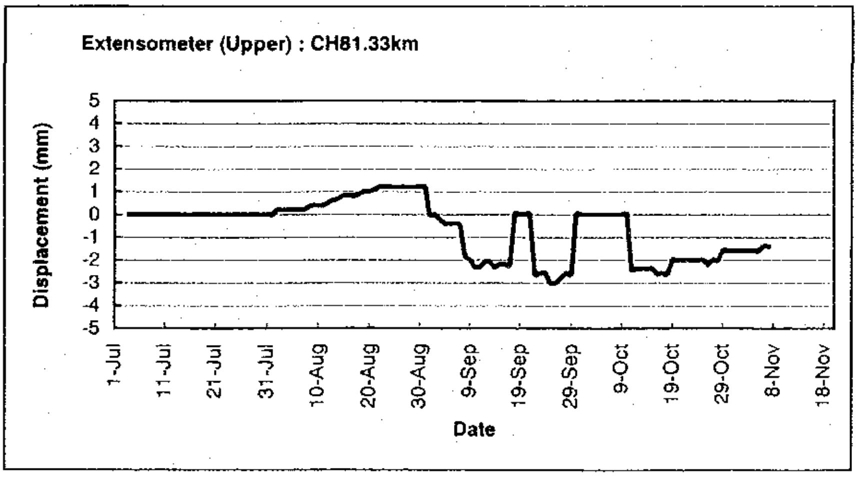


Figure 3.5.7 Extensometer Monitoring Results

(2) Rain Gauge

The rain gauge at Ch27.0km, Ch30.2km and Ch81.3km has been monitored from July to December 2001.
The results are summarised in Figures 3.5.8 to 3.5.15.
The points of monitoring results are as follows:

i) Three rain gauges were installed at Ch27.0km, Ch30.3km and Ch81.3km. Monitoring has been done for five months from 6th July 2001 to 6th November 2001.

ii) The amount of Rainfall at three location are different even between Ch27.0km and Ch30.3km which are close by each other as shown on Figures 3.5.8 and 3.5.10.

iii) Maximum hourly rainfall was,
Ch27.0km 33.2 mm/hour on 29th Oct. 23:00-24:00
Ch30.3km 34.4 mm/hour on 24th Jul. 18:00-19:00
Ch81.3km 36.4 mm/hour on 10th Jul. 21:00-22:00.

iv) Figure 3.5.14 shows the accumulate hourly rainfall at above time. It shows rainfall patterns at only two locations, Ch27.0km and Ch30.3km are similar. Sometimes the rainfall patterns at all locations are similar as on 7th September, 27th October as shown on Figure 3.5.15.

As mentioned above, the rainfall are deferent at location by location even if the locations are close each other.

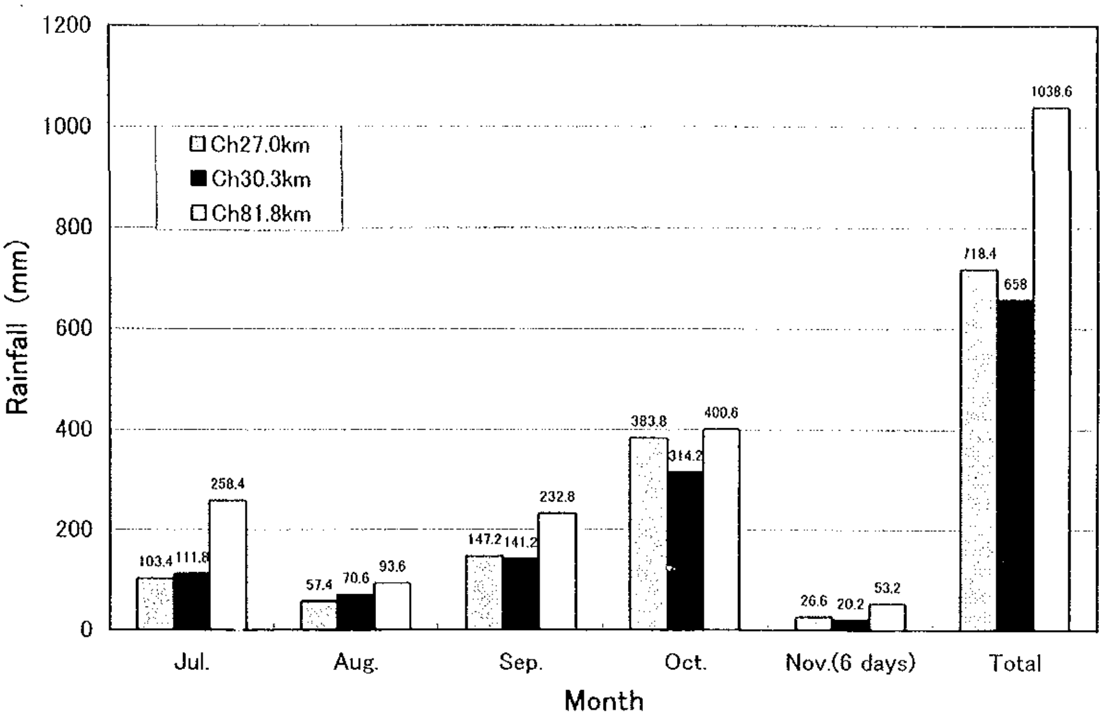


Figure 3.5.8 Monthly Rainfall

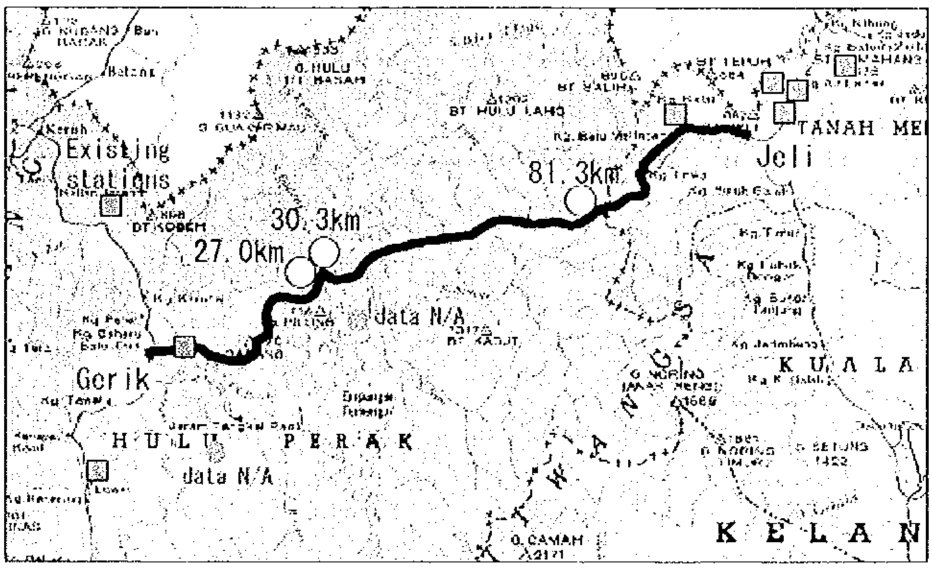
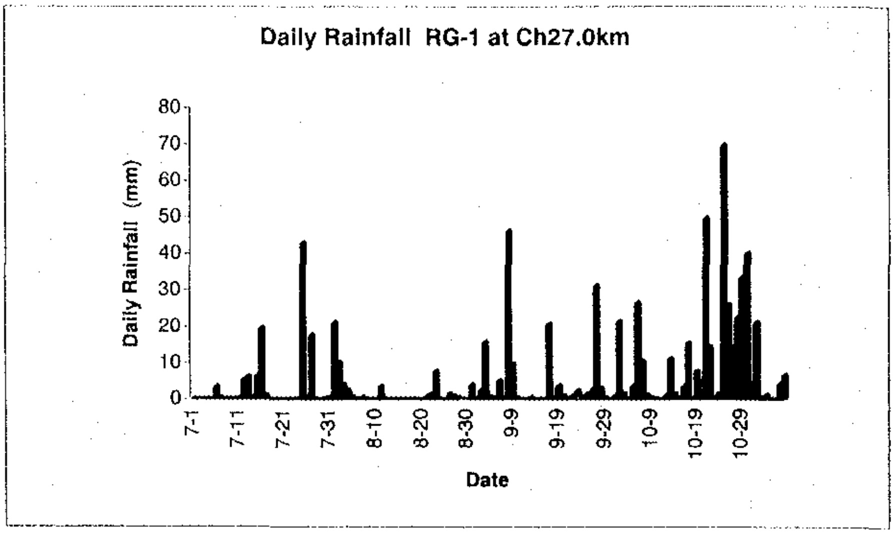
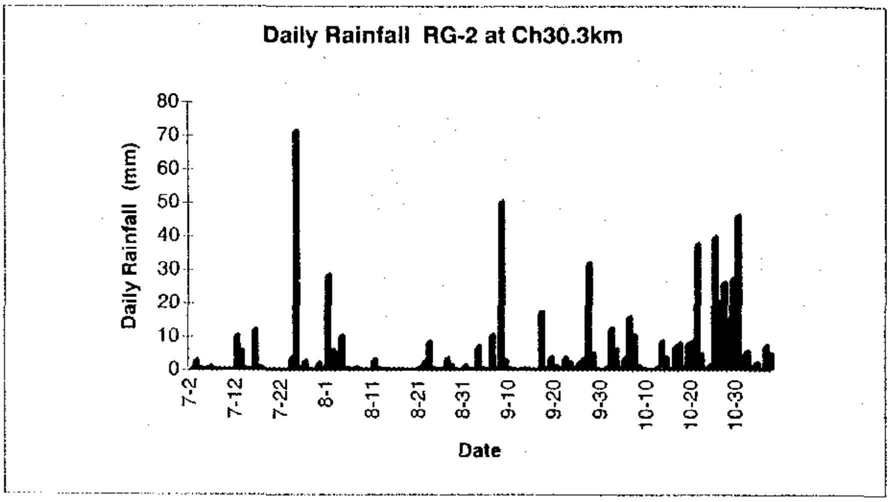


Figure 3.5.9 Location of the Rain Gauge Stations





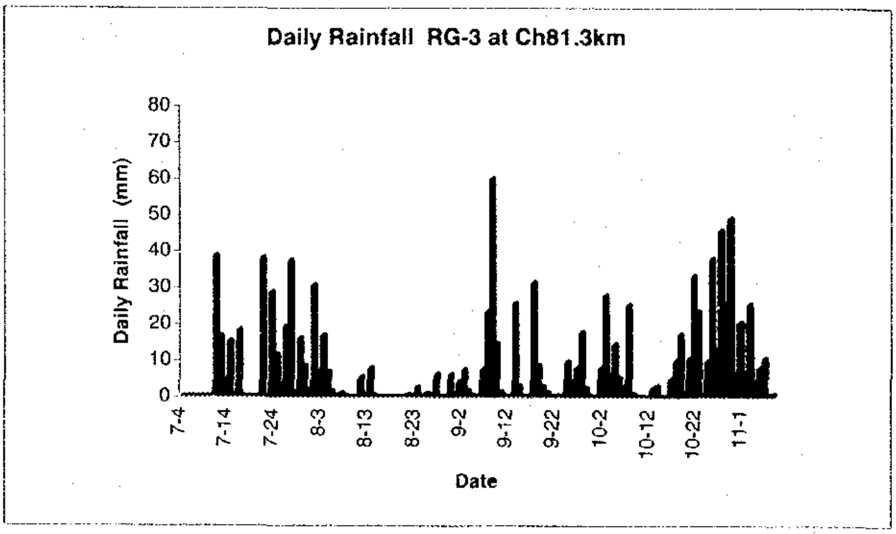
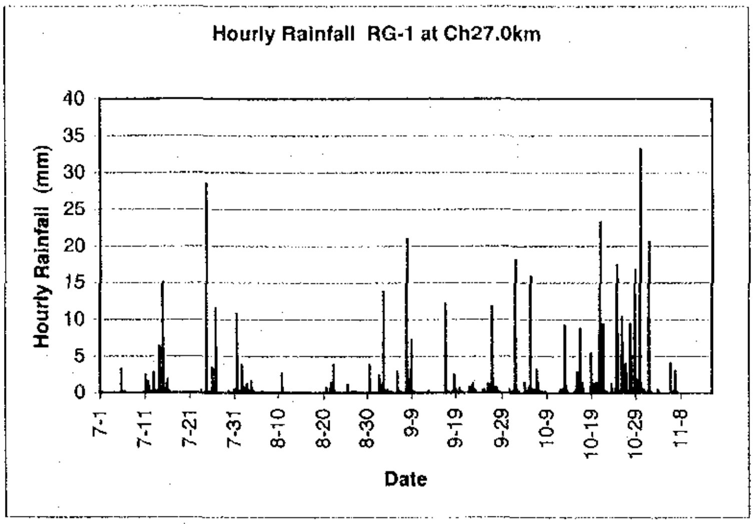
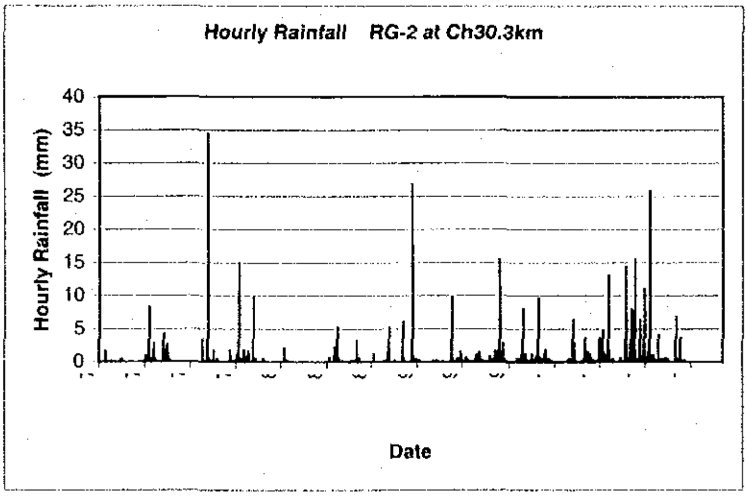


Figure 3.5.10 Daily Rainfall





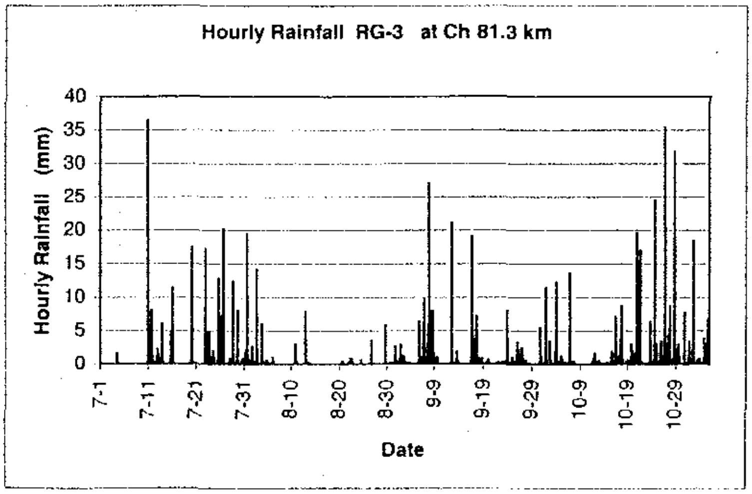
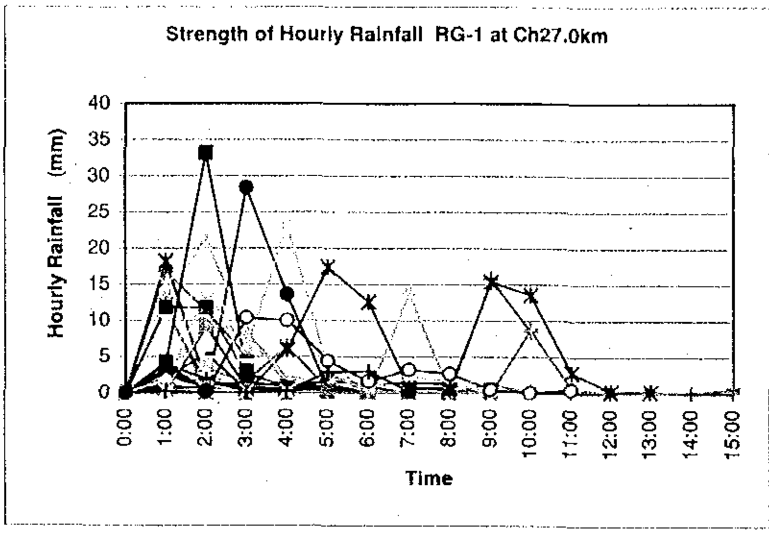
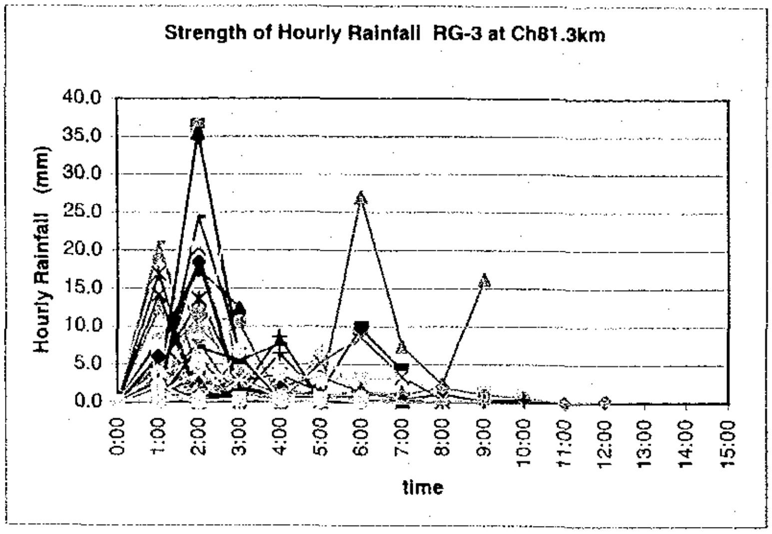


Figure 3.5.11 Hourly Rainfall from 1st Jul. 2001 to 6th Nov. 2001





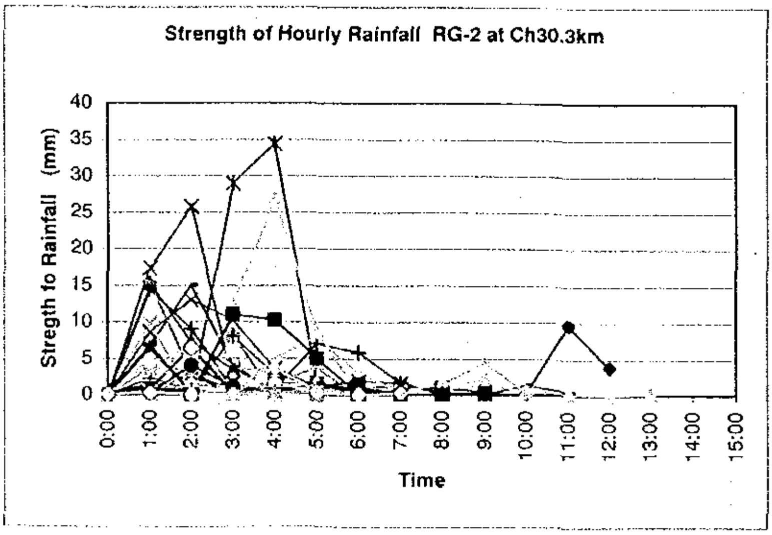
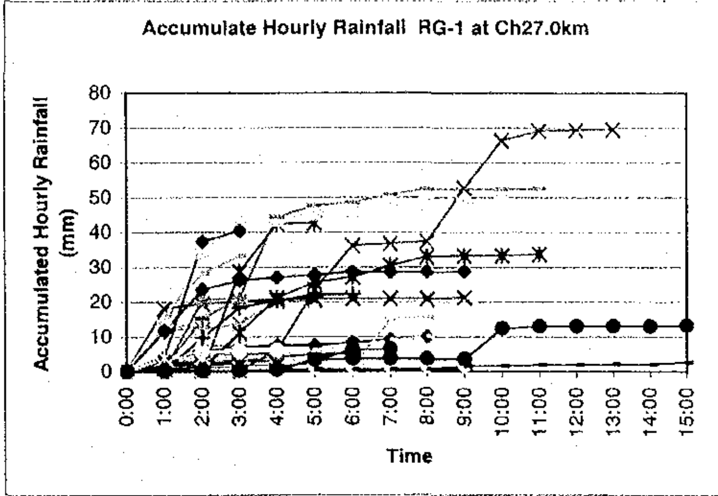
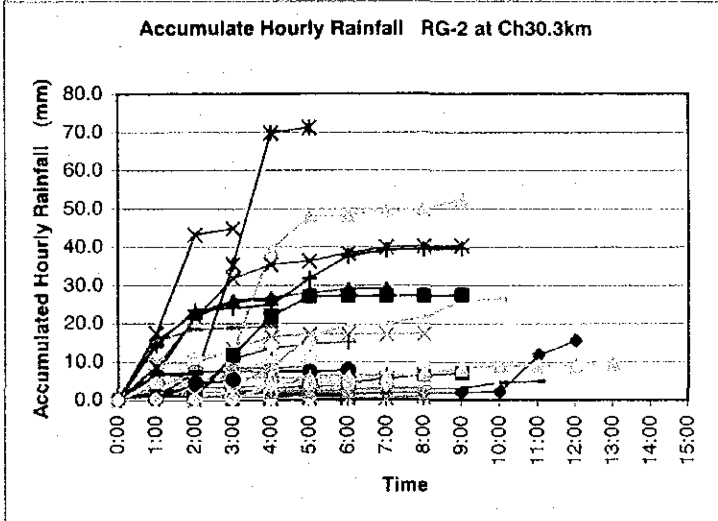


Figure 3.5.12 Strength of Hourly Rainfall





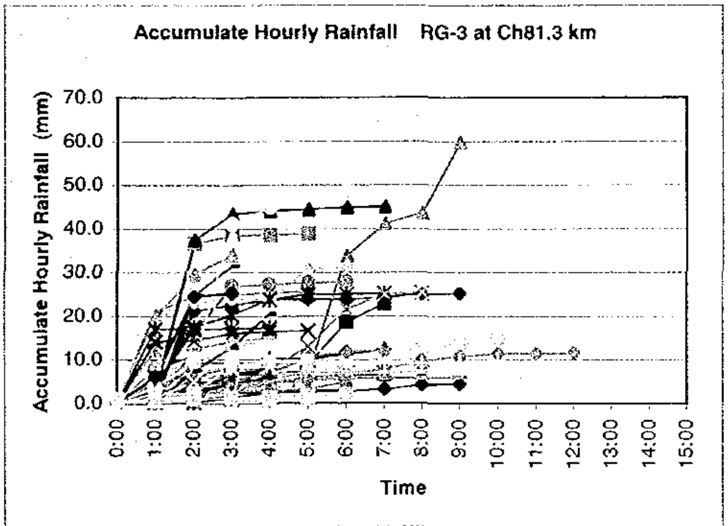
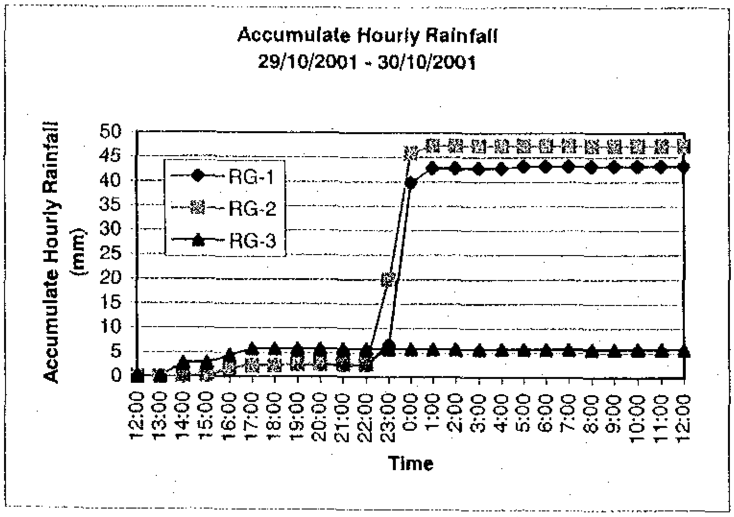
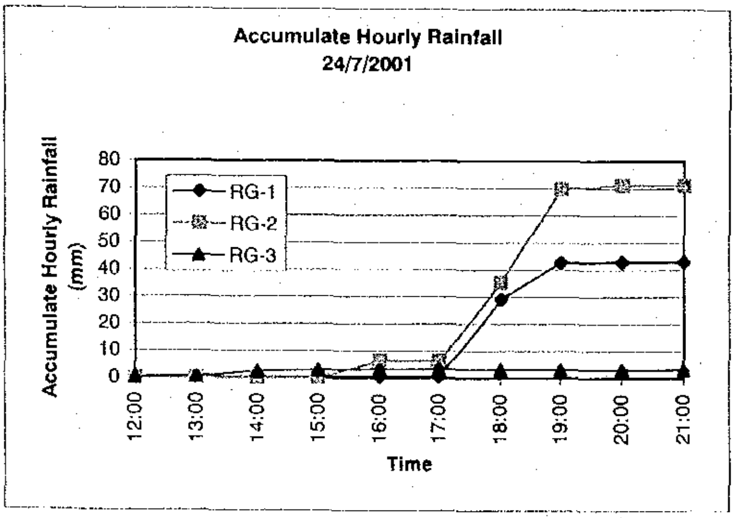


Figure 3.5.13 Accumulate Hourly Rainfall





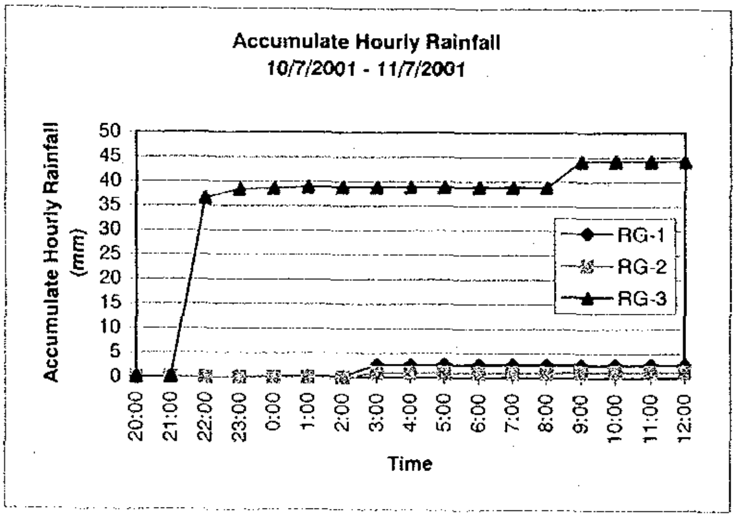
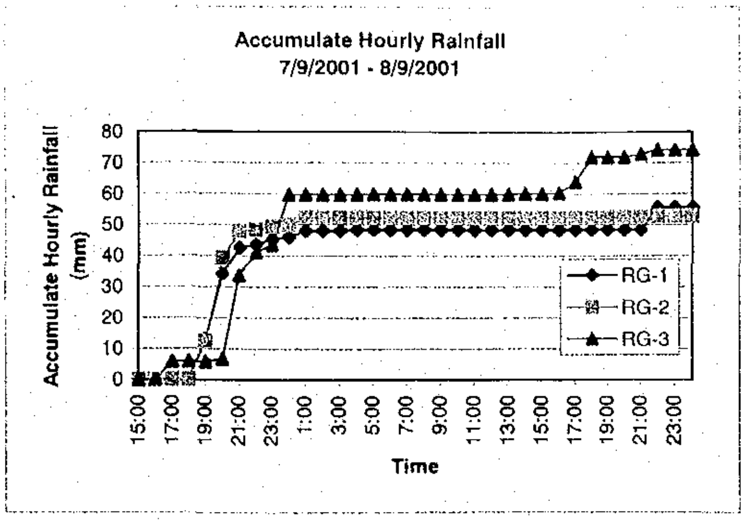


Figure 3.5.14 Comparison of Accumulate Hourly Rainfall at Rain Gauge



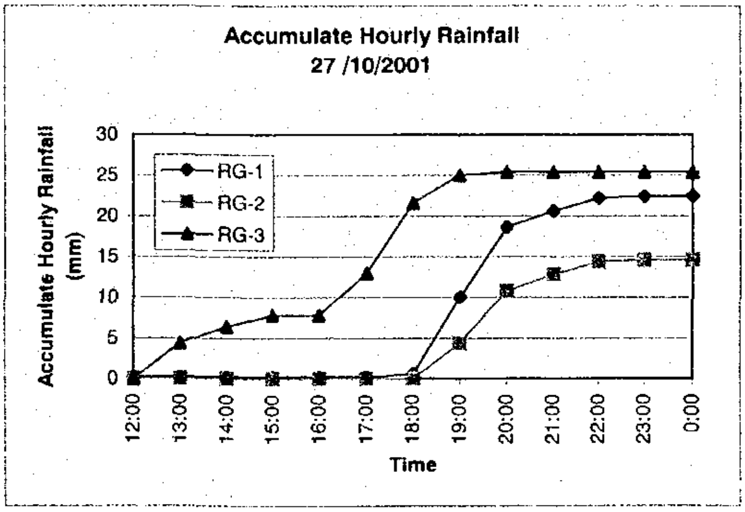


Figure 3.5.15 Comparison of Accumulate Hourly Rainfall at Rain Gauge

3.5.3 Site Phto of Instrumentation

Figure 3.5.16 Inclinometer (27.0km BH-1)

Figure 3.5.17 Rain Gauge(CH27.0km)

Figure 3.5.18 Wire Extensometer (CH27.0km)

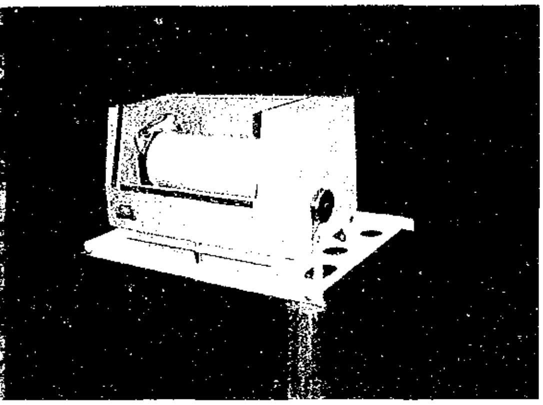


Figure 3.5.19 Extensometer Recorder

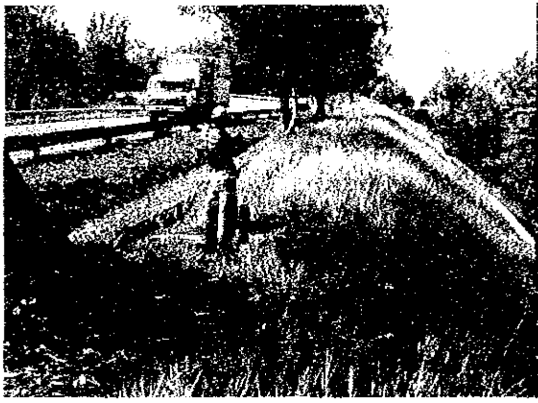


Figure 3.5.20 Water standpipe (30.38km BH-3)

Figure 3.5.21 VW Piezometer (30.32km BH-1)

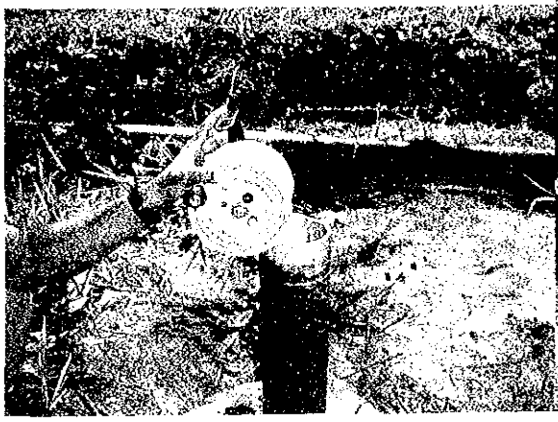


Figure 3.5.22 Water Standpipe (30.38km BH-3)


Figure 3.5.23 Rain Gauge (CH30.3km)

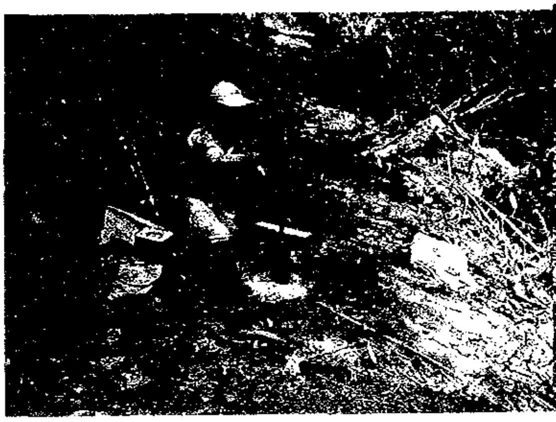


Figure 3.5.24 Inclinometer (81.28km BH4)


Figure 3.5.25 Inclinometer (81.33km BH2)

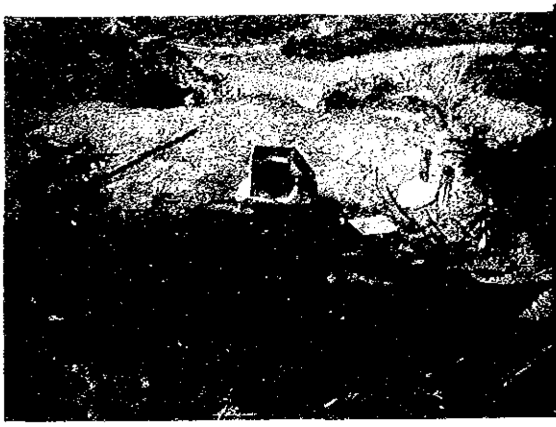


Figure 3.5.26 VW Piezometer (81.33km BH-1)


Figure 3.5.27 VW Piezometer (81.33km BH-3)

Figure 3.5.28 Water Standpipe(81.28km BH-5)

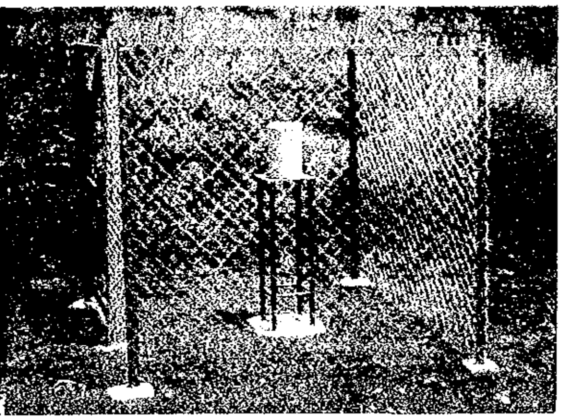


Figure 3.5.29 Rain Gauge (CH81.3km)

Figure 3.5.30 Wire Extensometer
(81.33km Lower)

Figure 3.5.31 Wire Extensometer
(81.33km Upper)

3.6 Preliminary Design of Countermeasure

The study team selected three representative sites for detail study of collapse, embankment failure, and landslide.

CH 27.00 km (Collapse, landslide)

CH 30.32 km (Embankment failure)

CH 81.30 km (Landslide)

3.6.1 CH 27.00 km (Collapse, Landslide)

The studied area locates at the end of high cut slope of road toward Jeli, and it also locates the slide of large landslide area. The feature of this slope failure has both feature of collapse and landslide. In this case, the countermeasure of slope failure should be selected against landslide for the safety of traffic. The boring investigation results show that the rock-type consists of interstratified sandstone and shale. The Standard Penetration Test (SPT) N-value generally shows a relatively high ranging from 22 to more than 100. The materials generally consist of heavy weathered sandstone and shale.

The results of elastic velocity survey show that the completely to highly weathered rock zone develop 15 m to 20m under surface. And highly to moderate weathered rock zone develop 50 m to 70 m under surface.

The results of resistivity survey show that high resistivity zone develop at cut slope of roadside and high part of cut slope. It is considered that these high resistivity zones developed loosened rock zone.

The results of slope investigation and shape of deformation of slope, slip surface is assumed like Figures 3.7.2 and 3.7.4.

The stability factor of this slope is determined Fs=1.00 because of this slope is stable on normal condition, except heavy rain situation.

The countermeasure of this slope failure can be selected crib work and ground anchors because of steep grade of slope and slip surface. It is also available the diversion of route, because of existence of open space at opposite of road, and counterweight embankment can be done at open space of shifted old road. So we will study the countermeasure of this slope at two cases, 1) counterweight embankment and road diversion, 2) crib work and ground anchors.

Table 3.6.1 Result of stability analysis (CH 27.00 km)

	Unit weight γ_t (kN/m^3)	Cohesion C (kN/m^2)	Internal friction angle ϕ (degree)	Safety factor Fs
CH 27.00 km	18	15	26.879 ($\tan\phi = 0.5069$)	1.00

Case I Counterweight embankment and road diversion

Designed safety factor : Fsp = 1.20

The road should be shifted to riverside about 15 m and counterweight embankment should be done in open space. The required dimension of counterweight embankment is

Gradient :	1:2.0
Width of crest :	9m
Height of embankment :	7m
Volume :	$80m^2*200m = 16,000m^3$

Case **II** Crib work and ground anchors

Designed safety factor : Fsp = 1.20

The summarized results of analyses is follows

Nos. of steps :	6
Horizontal interval :	3 m
Strength of tendon :	492.1 kN/unit (VSL Type E5-5)
Fixed length :	4.5 m
Total numbers :	384
Total length of anchors :	6720m
Width of crib :	60 cm

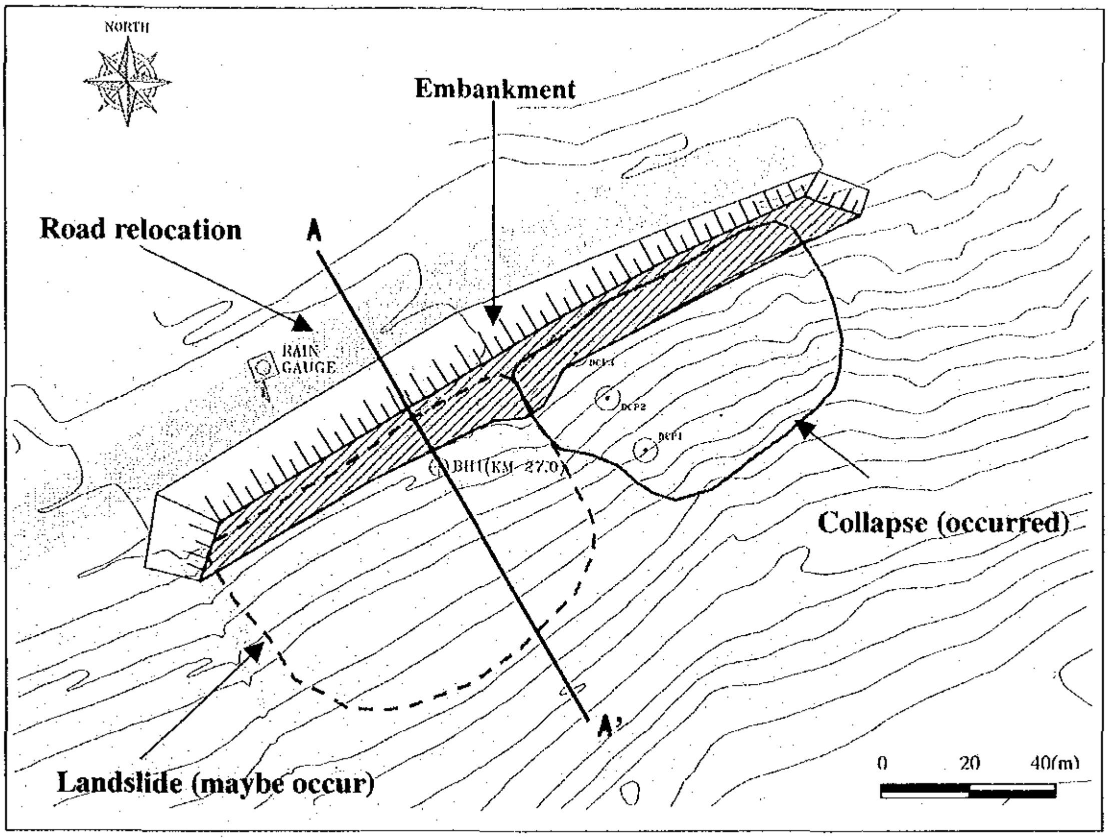


Figure 3.6.1 Plan of countermeasure embankment in CH 27.00 km

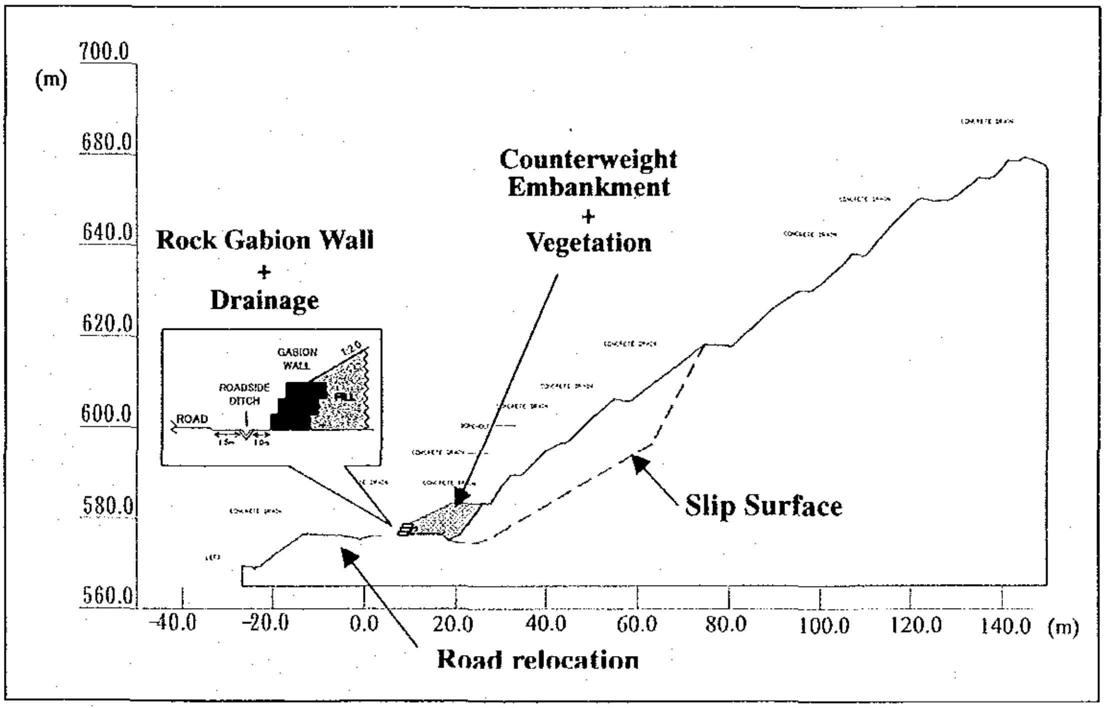


Figure 3.6.2 Cross section of countermeasure embankment CH 27.00 km

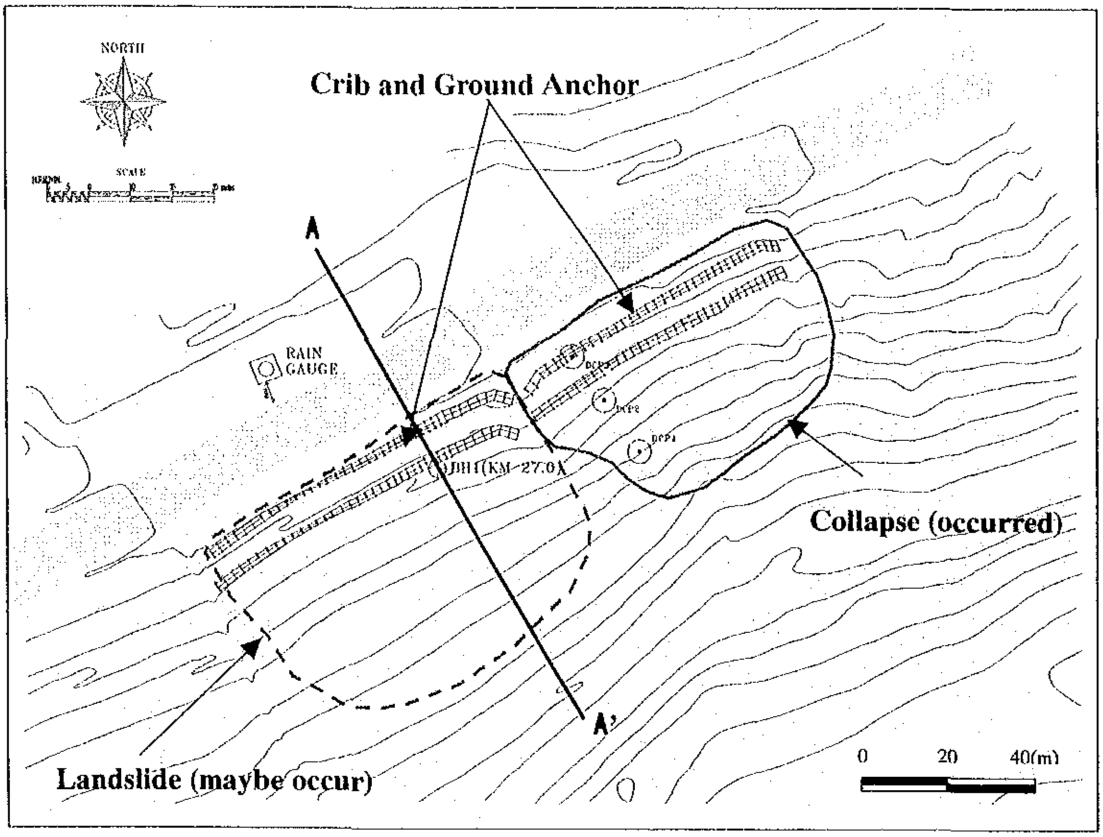


Figure 3.6.3 Plan of ground anchor works in CH 27.00 km

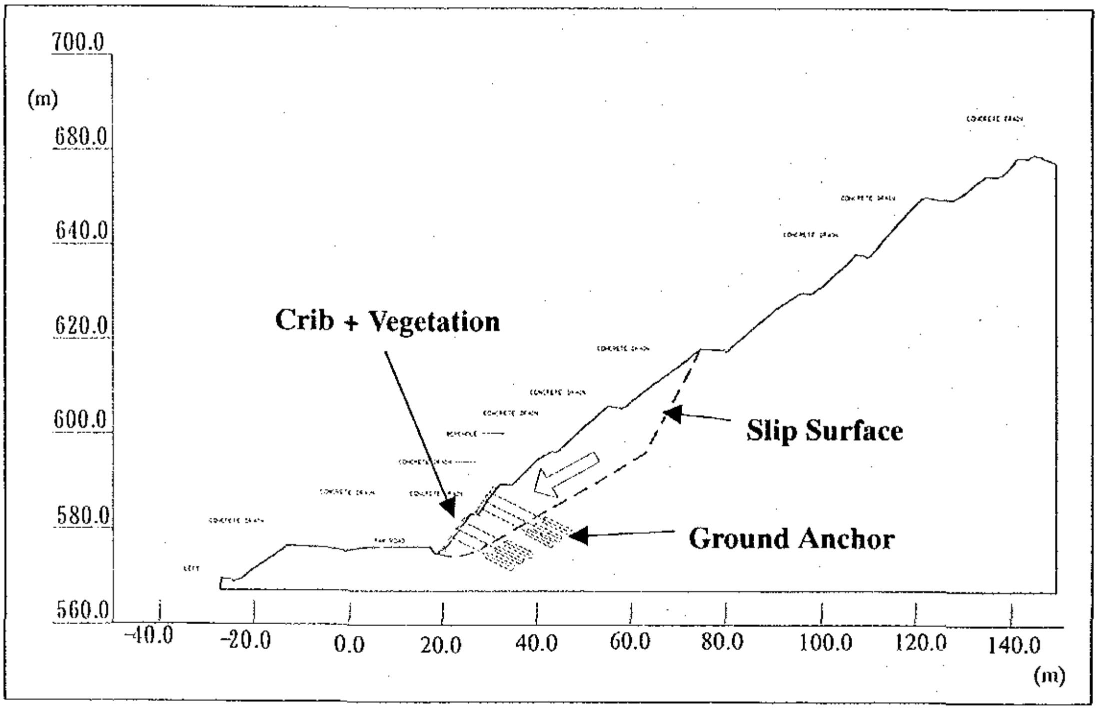


Figure 3.6.4 Cross section of ground anchor works in CH 27.00 km

3.6.2 CH 30.4 km (Embankment Failure)

The results of slope investigation show that maximum height of fill is 35 m and the groundwater level stayed almost original ground surface. Materials of fill are mainly silt and contain small amount of sand and gravel. Most part of fill was compacted more than 20 N-Value. The estimated collapse pattern is a progressive failure from lower erosion to upper part. So countermeasure of this site is complete drainage of berm and boundary of ground and fill.

Drain ditch and cascade :	180m
Surface drainage :	330m

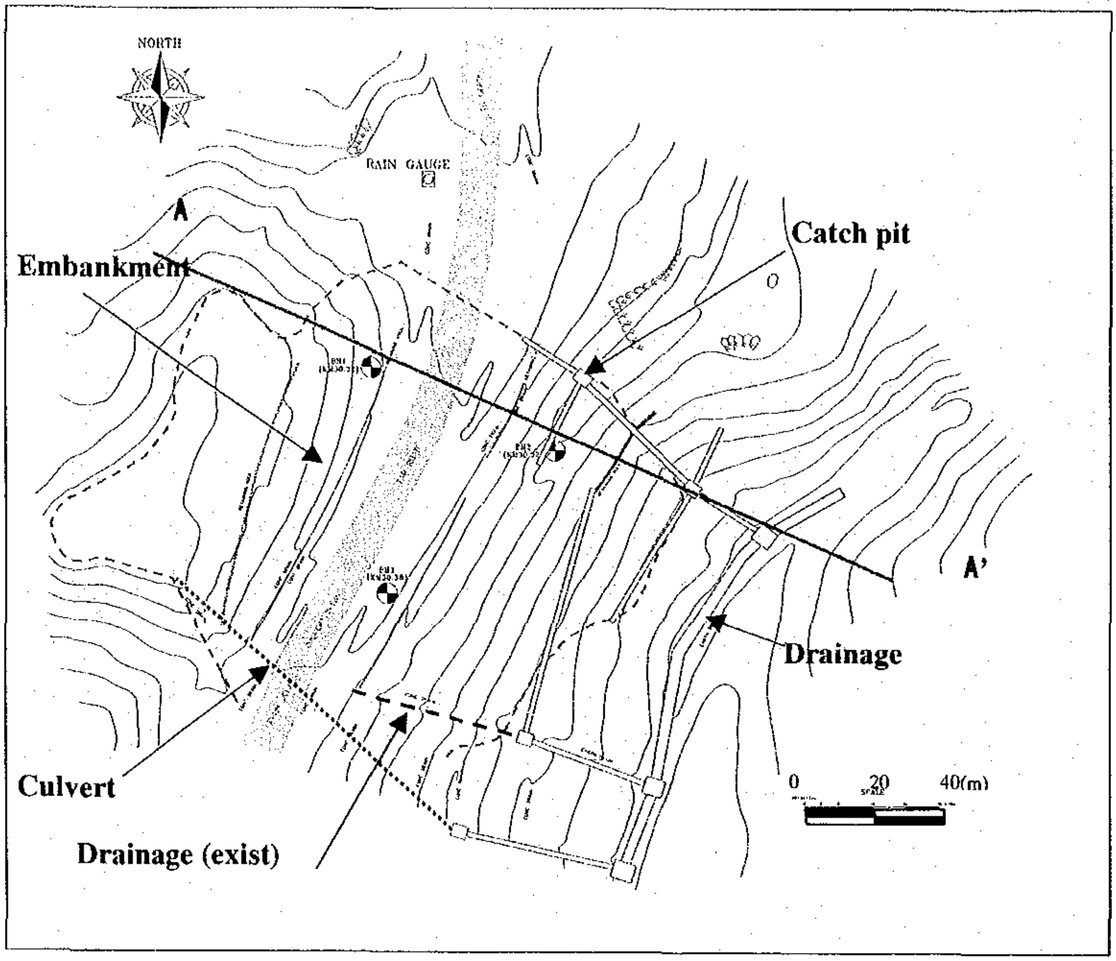


Figure 3.6.5 Plan of CH 30.30 km

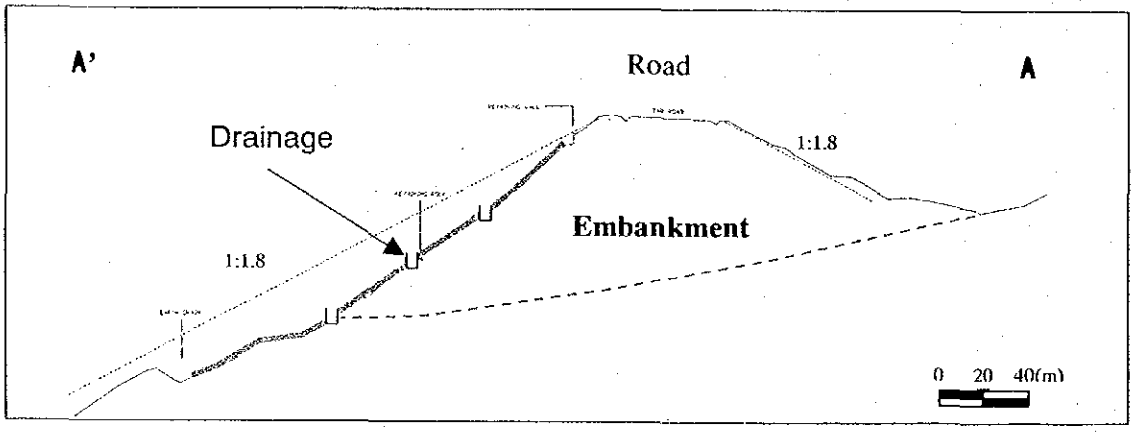


Figure 3.6.6 Cross section of CH 30.30 km

3.6.3 CH 81.3 km (Landslide)

This slope is located on the left-hand of E-W Highway and there is a rock gabion wall with length of 150 m. In this area, the top scarp of landslide is obvious, and this rock gabion has been lift up at the centre of landslide.

The boring investigation results show that the rock-type consists of interstratified sandstone and schist. The Standard Penetration Test (SPT) N-value generally shows a relatively high ranging from 22 to more than 100.The materials generally consist of heavy weathered sandstone and shale. At the boring No BH-5, sheared shale is developed until bottom of borehole.

The results of elastic velocity survey show that the highly to moderate weathered rock zone develop 15 m to 20m under surface. And highly to moderate weathered rock zone develop 30 m to 40 m under surface.

The results of resistivity survey show that high resistivity zone develop at cut slope of roadside and upper part of scarp, and low resistivity zone developed middle part of landslide. It is considered that this low resistivity zone developed high groundwater zone.

The results of slope investigation and shape of deformation of slope, slip surface is assumed like Figures 3.6.8 and 3.6.10.

The stability factor of this slope is determined Fs=1.00 because of this slope is stable on normal condition, except heavy rain situation.

Table 3.6.2 Result of stability analysis (CH 81.30 km)

	Unit weight γ_t (kN/m^3)	Cohesion C (kN/m^2)	Internal friction angle ϕ (degree)	Safety factor Fs
CH 81.30 km	18	20	17.352 (tan ϕ = 0.3125)	1.00

Case I Road relocation counterweight embankment, and horizontal drain holes

Designed safety factor ; Fsp = 1.20

The road should be shifted to opposite side of landslide about 15 m and counterweight embankment should be done in open space. The required dimension of counterweight embankment is follows:

Table 3.6.3 Result of stability analysis (CH 81.30 km)

	Unit weight γ_t (kN/m^3)	Cohesion C (kN/m^2)	Internal friction angle ϕ (degree)	After Embankment Fs	After Embankment and Drain holes Fs
CH 81.30 km	18	20	17.352 (tan ϕ = 0.3125)	1.12	1.19

- Embankment
  Gradient : 1:1.5
  Width of crest : 17m
  Height of embankment : 10.5m
  Volume : $120m^2*150m = 18,000\ m^3$
- Vegetation
  The gross area : $19m*150m = 2,850\ m^2$
- Horizontal drain holes
  Length : 50 m
  Nos. : 9
  Total length : 50 m * 9 = 450 m
- Rock gabion wall
  Volume : $H * D * L = 3 * 3 * 150 = 1350\ m^3$
- Drain ditch
  Length : 150m
- *Road relocation*
  Length : 200m
  Cutting : $V = 100 * 120 = 1200\ m^3$

Case II Restraint works by steel pipe piles and horizontal drain holes

Designed safety factor ; Fsp = 1.20

The summarized result of analyses is follows

Table 3.6.4 Result of stability analysis (CH 81.30 km)

CH	Unit weight γ_t (kN/m^3)	Cohesion C (kN/m^2)	Internal friction angle ϕ (degree)	After Drain Holes Fs	Required Shearing Resistance for Fs=1.20 (kN/m)
81.30 km	18	20	17.352 (tan ϕ = 0.3125)	1.086	630.36

- Horizontal drain holes
  Length : 50 m
  Nos. : 9
  Total length : 50 m * 9 = 450 m
- Steel Pipe Piles
  ϕ ; 400 mm
  t ; 24 mm
  τ sa ; $490\ N/mm^2$ (SKK 490)
  Pitch : 2.0m (Horizontal)
  Length : L = 17.5 m
  Nos. : n = 150/2 = 75
  Total length : 17.5 m * 75 =1,312.5 m

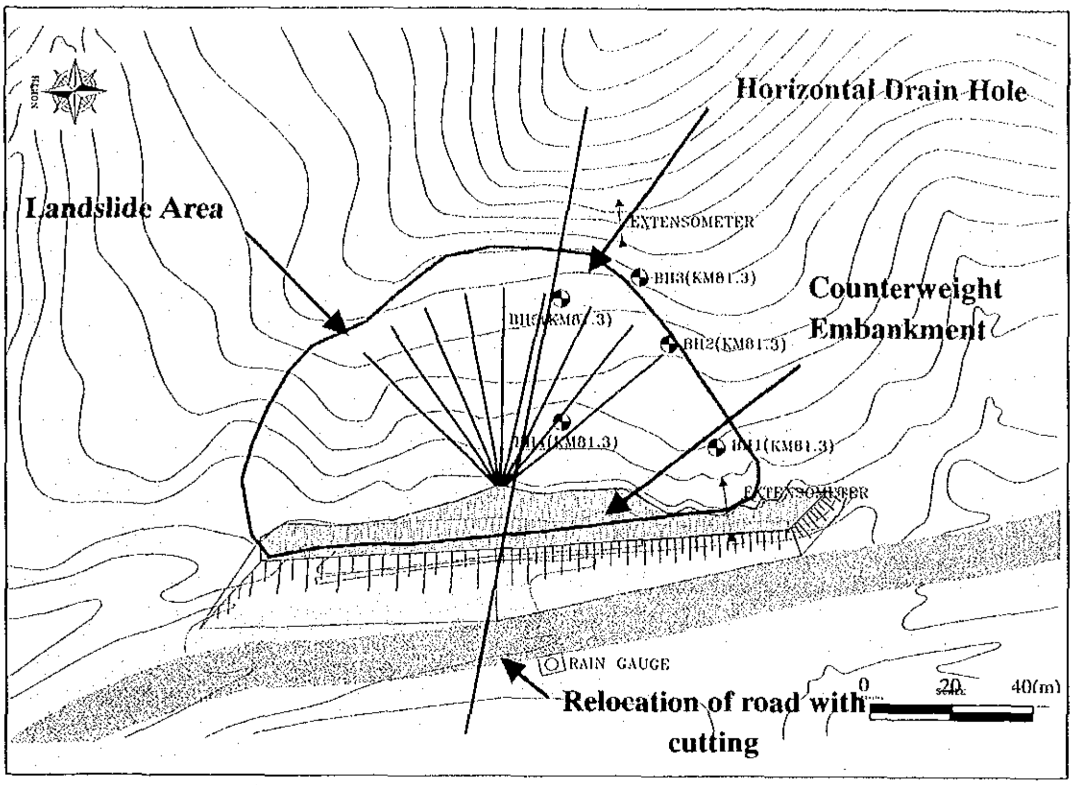


Figure 3.6.7 Plan of counterweight embankment in CH 81.30 km

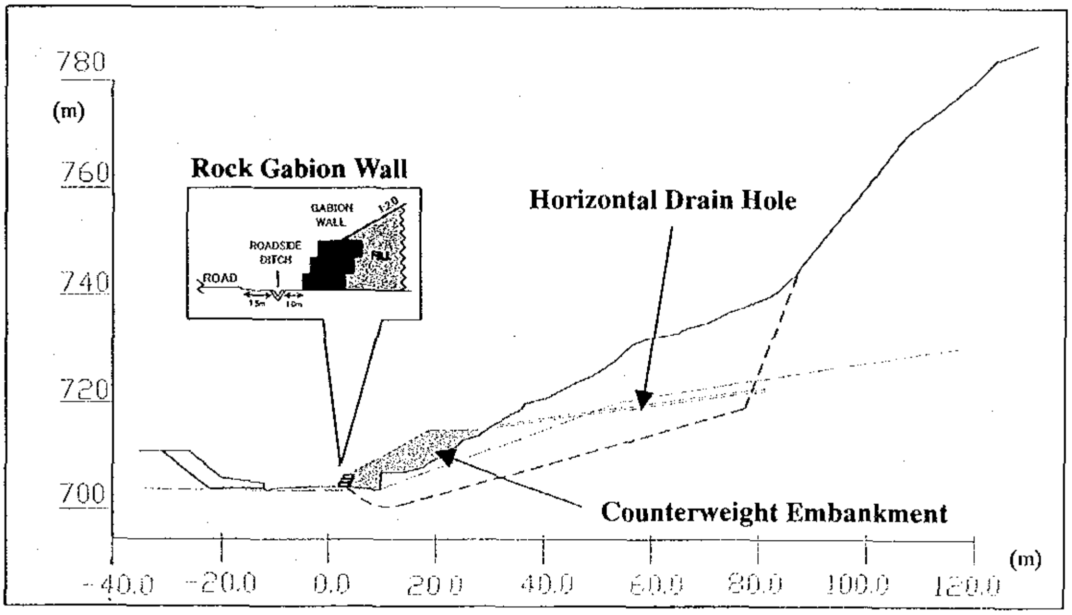


Figure 3.6.8 Cross section of counterweight embankment in CH 81.30 km

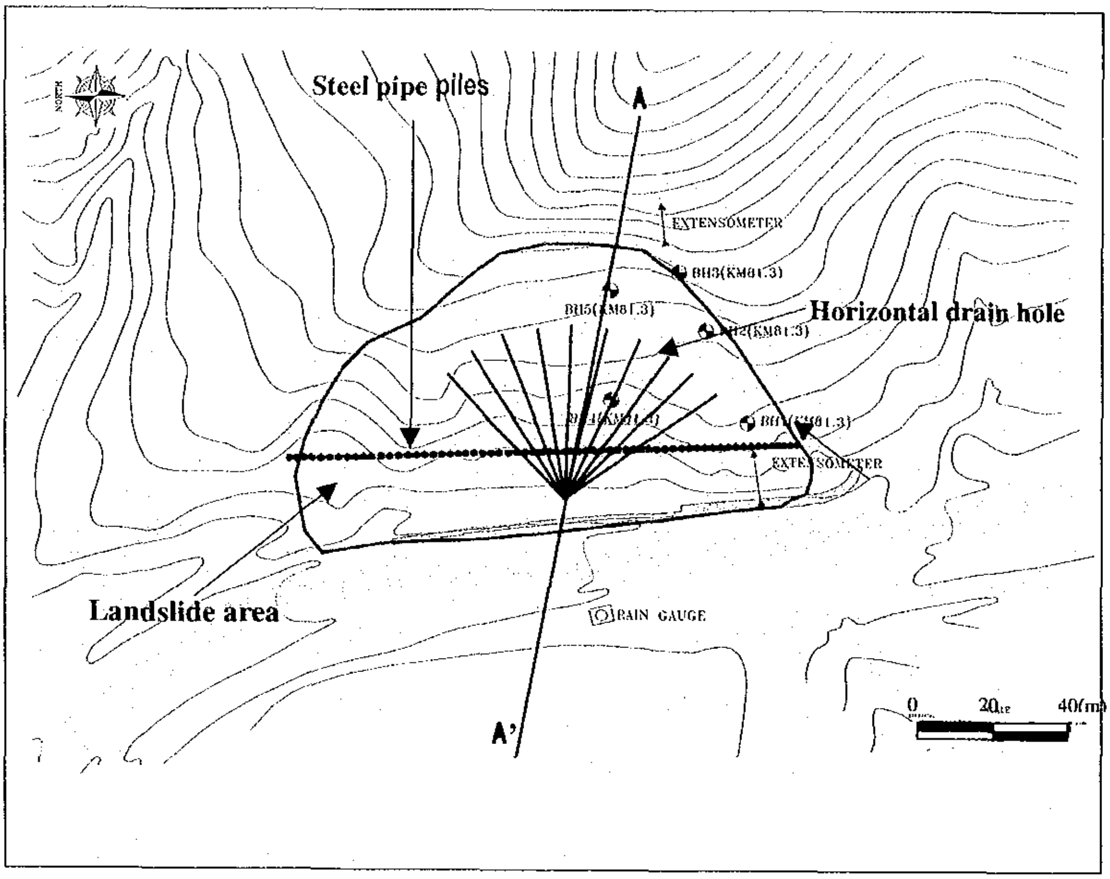


Figure 3.6.9 Plan of Steel-pipe pile works in CH 81.30 km

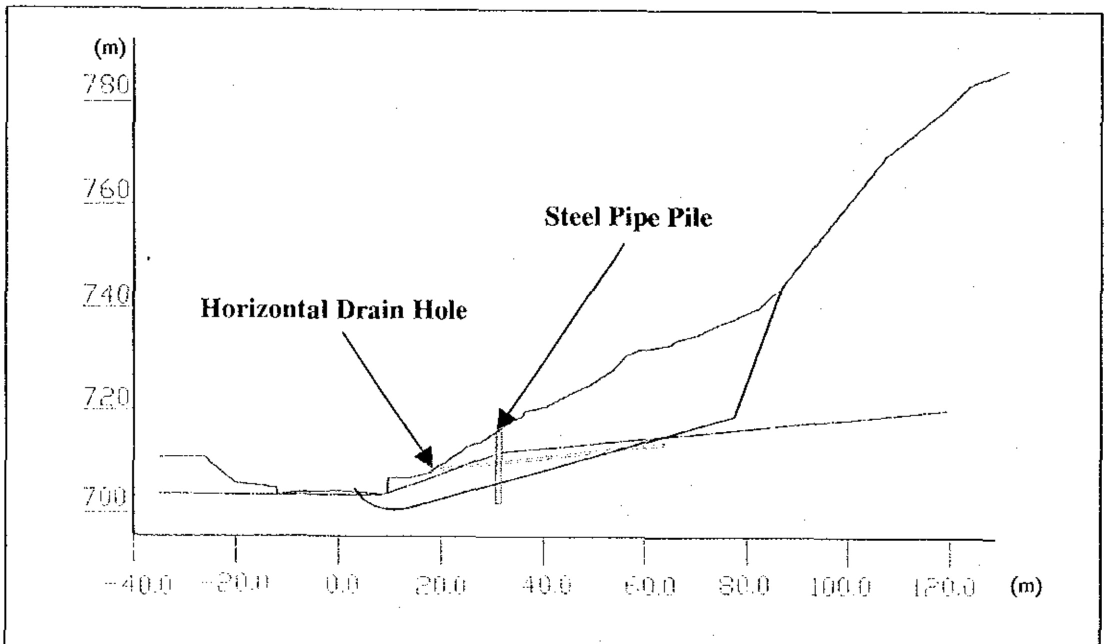


Figure 3.6.10 Cross section of Steel-pipe pile works in CH 81.30 km

3.6.4 Cost Estimate of Countermeasures

Estimated cost of the each case study site is following:

(1) CH 27.00 km (Collapse, Landslide)

• **Case I** Road relocation and counterweight embankment

Table 3.6.5 Cost of countermeasure

	Countermeasure	Unit	Unit Rate	Quantity	Amount
1	Road Relocation	m	300	200	60,000.00
2	Counterweight Embankment	m^3	8	16,000	128,000.00
3	Rock Gabion Wall	m^3	45	1,800	81,000.00
4	Surface Drainage	m	30	200	6,000.00
5	Vegetation	m^2	3	2,400	7,200.00
TOTAL					347,000.00

• **Case II** Ground anchor

Table 3.6.6 Cost of countermeasure

	Countermeasure	Unit	Unit Rate	Quantity	Amount
1	Ground Anchor (492.1 kN/unit)	m	550	6,720	3,696,000.00
2	Crib (Width 60 cm)	Nos.	5,000	384	1,920,000.00
3	Vegetation	m^2	3	2,400	7,200.00
TOTAL					5,623,200.00

(2) CH 30.40 km (Embankment failure)

Table 3.6.7 Cost of countermeasure

	Countermeasure	Unit	Unit Rate	Quantity	Amount
1	Drain Ditch and Cascade	m	80	100	8,000
2	Surface Drainage	m	30	330	9,900
TOTAL					17,900

(3) CH 81.30 km (Landslide)

• **Case I** Road relocation, counter embankment and drain hole

Table 3.6.8 Cost of countermeasure

	Countermeasure	Unit	Unit Rate	Quantity	Amount
1	Road Relocation	m	300	200	60,000.00
2	Counterweight Embankment	m^3	8	18,000	144,000.00
3	Horizontal Drain Hole	m	70	450	31,500.00
4	Rock Gabion Wall	m^3	45	1,350	60,750.00
5	Surface Drainage	m	30	150	4,500.00
6	Vegetation	m^2	3	2,850	8,550.00
TOTAL					305,250.00

• **Case II** Restraint works by steel pipe piles and horizontal drain holes

Table 3.6.9 Cost of countermeasure

	Countermeasure	Unit	Unit Rate	Quantity	Amount
1	Steel Pipe Pile	m	3,000	1,312.5	3,937,500.00
2	Horizontal Drain Hole	m	70	450	31,500.00
TOTAL					3,969,000.00

3.6.5 Preliminary Design of Countermeasure of Penanpang-Tambunan-Keningau Road

The study team also selected S05 slope as a landslide and S19 slope as a rock fall for case study of preliminary design of counter measure at Sabah.

(1) S05 Slope

Disaster occurred as subsidence of road, and the pavement has been broken by landslide, and now sinkage reached more than 2 m. The cause of landslide is assumed that the fill of road added the normal force of landslide, the stability of landslide decreased a little, and ground water level rose in the landslide mass, and then landslide moved toward down slope.

The stability factor of this landslide is estimated almost Fs = 1.00 on actual condition, and filled condition of first construction of road is less than 1.00.

Table 3.6.10 Result of stability analysis (Sabah S 05)

S 05	Unit weight γ_t(kN/m^3)	Cohesion C (kN/m^2)	Internal friction angle ϕ(degree)	Stability factor construction stage	Stability factor on actual condition
	18	20.0	20.545	0.986	1.00

Case I Counterweight embankment and horizontal drain holes.

Designed safety factor : Fsp = 1.20

On the assumption that the ground water level decrease 3 m at drain hole of ground water by construction of horizontal drain hole, the safety factor of landslide is follows;

Table 3.6.11 Result of stability analysis after construction of drain hole and counterweight embankment

S 05	Unit weight γ_t(kN/m^3)	Cohesion C (kN/m^2)	Internal friction angle ϕ(degree)	Stability factor after drain holes	Stability factor after drain holes and embankment
	18	20.0	20.545	1.127	1.192

The counter weight embankment should be done at the toe of landslide. As the result of stability analysis the required dimension of counterweight embankment is follows;

Crest level :	1465.5m
Width of Crest :	3m
Gradient :	1:1.8
Width of berm :	5m

Height of slope : 10m
Volume : 20,000m^3

- Gabion wall at toe of embankment
  Height : 4m
  Volume : 800m^3
- Horizontal drain holes
  Length : 50m
  Layer : 2
  Total holes : 20 nos.
  Total length : 1000m

Case II Counterweight embankment and drain well.

Designed safety factor : Fsp = 1.20

On the assumption that the ground water level decrease 5 m at drain well by construction of drain well, the safety factor of landslide is follows;

Table 3.6.12 Result of stability analysis after construction of drain well and counter embankment

S 05	Unit weight γ_t(kN/m^3)	Cohesion C (kN/m^2)	Internal friction angle ϕ (degree)	Stability factor after drain well	Stability factor after drain well and embankment
	18	20.0	20.545	1.127	1.212

The counter weight embankment should be done at the toe of landslide. The required dimension of counterweight embankment is follows;

Crest level : 1465.5m
Width of Crest : 3m
Gradient : 1:1.8
Width of berm : 5m
Height of slope : 10m
Volume : 13,000m^3

- Gabion wall at toe of embankment
  Height : 4m
  Volume : 800m^3
- *Drainage well*
  Length : 15m
  Catchment drain holes (layer : 2)
  Total holes : 16 nos.
  Total length : 560m

Case III Counterweight embankment and horizontal drain holes.

Designed safety factor : Fsp = 1.20

On the assumption that the ground water level decrease 3 m at drain hole of ground water by construction of horizontal drain hole, the safety factor of landslide and required shearing resistance are follows;

Table 3.6.13 Result of stability analysis after construction of drain hole and counter embankment

S 05	Unit weight γ_t(kN/m^3)	Cohesion C (kN/m^2)	Internal friction angle ϕ (degree)	Stability factor after drain hole	Required shearing resistance (kN/m)
	18	20.0	20.545	1.127	714.94

Design of steel pipe pile work is contained in Attachment **.

- Steel pipe piles
  - ϕ : 450 mm
  - t : 24mm
  - τ sa : 490 kN/mm^2 (SKK 490)
  - Pitch : 2.0 m (horizontal)
  - Length : L = 17.5m
  - Nos. : n= 40
  - Total length : 17.5*40 = 700 m
- Horizontal drain holes
  - Length : 50m
  - Layer : 2
  - Total holes : 20 nos.
  - Total length : 1000m

Case IV *Counterweight embankment and drainage well.*

Designed safety factor : Fsp = 1.20

On the assumption that the ground water level decrease 5 m at drainage well by construction of drainage well, the safety factor of landslide and required shearing resistance are follows;

Table 3.6.14 Result of stability analysis after construction of drainage well and counter embankment

S 05	Unit weight γ_t(kN/m^3)	Cohesion C (kN/m^2)	Internal friction angle ϕ (degree)	Stability factor after drainage hole	Required shearing resistance (kN/m)
	18	20.0	20.545	1.151	476.01

Design of steel pipe pile work is contained in Attachment **.

- Steel pipe piles
  ϕ : 350 mm
  t : 21mm
  τ sa : 490 kN/mm^2 (SKK 490)
  Horizontal pitch : 2.0 m
  Length : L = 17.5m
  Nos. : n= 40
  Total length : 17.5*40 = 700 m
- Drainage well
  Length : 15m
  Catchment drain holes (layer : 2)
  Total holes : 16 nos.
  Total length : 560m