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A Study on Slope protection by Grand Anchor for the Toukai Earthquake

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Abstract

The special reinforcement has not been done in the slope of the railroad conventionally. Therefore, we have suffered various damages from the earthquake in the railroad. In recent years, the soil reinforcing is adopted as the slope reinforcement method, when the banking slope is newly excavated. Also, the report is done as for the aseismicity of this soil reinforcing.

On the other hand, the case of the soil reinforcing as the slope reinforcement method is hardly reported in the cut slope. The main reasons are as follows. We can easily control the circumference face frictional force in the banking slope. On the other hand, it is difficult for us to control the circumference face frictional force in the cut slope.

The authors adopted the ground anchor method without adopting the soil reinforcing about the cut slope inside the earthquake disaster prevention measure strengthened area regarding the Toukai earthquake by The Large Scale Earthquake Countermeasures Act. The ground anchor method is an independent method from the circumference face frictional force. The concept of the soil reinforcing has not settled at the time of the execution, which is also the reason.

If we execute the ground anchor method in the cut slope as the earthquake counter measure, the authors have clarified, that can enlarge the effect of the slope protection.

Keyword: Large Scale Earthquake, Cutting slope, Soil reinforcing, Ground anchor method

1. Introduction

In the near future, quite a lot ranges of the Tohkai areas the large-scale earthquake of the magnitude 8 scale is expected to occur.

The government has established "The Earthquake prediction promotion headquarters" in October, 1976, by the cabinet meeting decision headed by the Director General of Science and Technology Agency according to the public opinion that the government should in-

tend the promotion of the earthquake prediction. The earthquake prediction promotion headquarters have organized "The Toukai area determination meeting" in April, 1977, as the lower part organization of the Coordinating Committee for Earthquake Prediction which was the private advisory organization of the Director General of The Geographical Survey Institute.

The Large Scale Earthquake Countermeasures Act has materialized in June, 1978, that does the legisla-

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tion of the disaster prevention measure in advance that is done on the basis of the earthquake prediction. As shown in Fig.1 as the earthquake disaster prevention measure strengthened area regarding the Toukai earthquake, by The Large Scale Earthquake Countermeasures Act that has been implemented in December, 1978, Chigasaki-shi and westward areas have been designated in Kanagawa Pref.

Below, the authors express about the railroad structure of the Japanese National Railway(JNR).

The JNR was made 1 of the designated public organization by The Large Scale Earthquake Countermeasures Act. The designated public organization was required to make the earthquake disaster prevention strengthened plan. The JNR made the measure that im-

proved the aseismicity, about the structures that existed in the disaster prevention measure strengthened area on the basis of this disaster prevention plan, to carry out.

Tokaido Shinkansen and, the Tokaido main line and the branch line group exist in the earthquake disaster prevention strengthened area. In Shinkansen we need to think giving priority to the security of the travelling safety of the high speed passenger train. On the other hand in the Tokaido line we need to intend the security of the train travelling safety. Also, we need to consider that a social influence is extremely big, if the Tokaido line is blocked for a long time.

We have received large earthquakes several times since the railroad opened in the Tokaido line. There is not the case that the travelling passenger train suffered

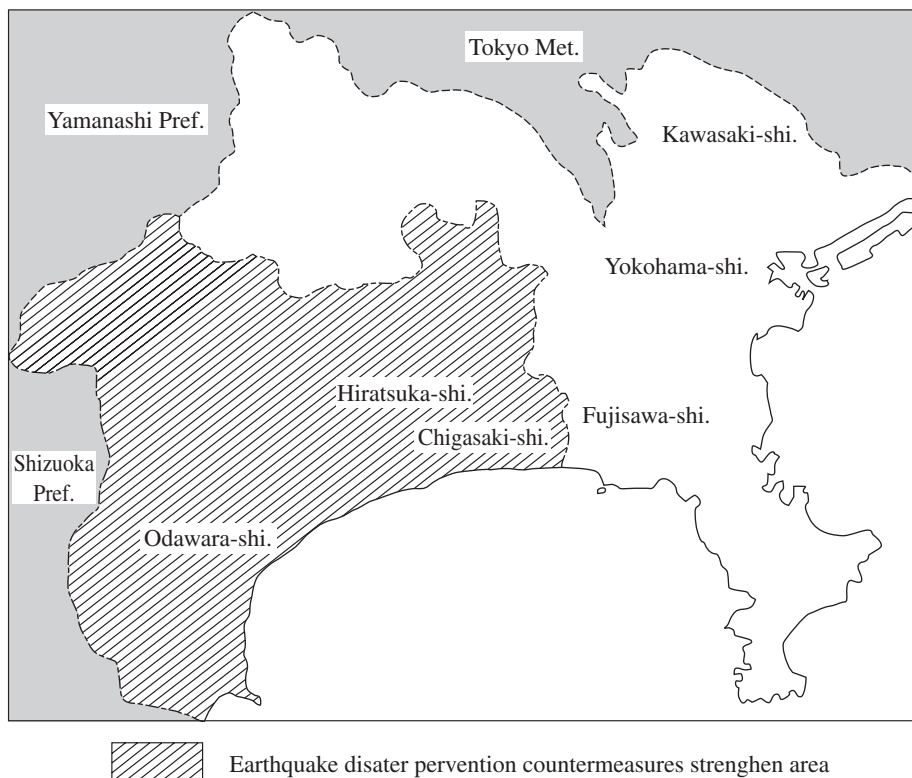


Fig.1 Earthquake disaster prevention countermeasures strengthen area in Kanagawa Pref.

big damage directly from ground shock itself in those earthquakes. In the case of the suffered train the cause was mostly other than the permanent way structure such as the landslide. However, the permanent way structure has received the earthquake damage. It is thought as the cause that there was considerably less train number in comparison with the present, that the train did not suffer big damage directly.

On the other hand, the structures such as the bridges, viaducts, high embankments and tunnels continuously exist in the Tokaido Shinkansen. There is a possibility that these structures suffer the damage from the large-scaled earthquake.

The mission of the railroad as a public shipping organization is to offer the stabilizing shipping service. In other words, it is ideal as a railroad that the usual shipping service is possible, without bringing about the function hindrance even if any large-scaled earthquake occurs.

Many of the railroad structure already built are surpassing in the aseismicity in comparison with general buildings etc. However, the aseismatic design to the degree that is able to guarantee it that is not carried out to the effect that it does not bring about the function hindrance to the earthquake of the highest level that may happen. Therefore, we need to strengthen the structure to the degree that stays at the damage that can recover as promptly as possible, without bringing about fatal collapse etc. to the structure.

Next the general aseismicity of each railroad structure is shown.

- ① The aseismatic design is done tentatively in the bridge and viaduct.
- ② As for the soil structure etc. the consideration of the aseismicity is not sufficient in comparison with the bridge etc.

In other words, there is difference in the aseismicity of each railroad structure. Accordingly, we need to establish the earthquake proof evaluation method corresponding to each structure.

We need to develop the method of high cost performance, in consideration of the execution condition of the hypothesized earthquake damage pattern and location and hypothesized damage part.

Big earthquakes of high seismic intensity including the South Hyogo earthquake(January 1995) occurred

successively. In Japan, therefore, how to prevent earthquake disasters has become an important concern not only for the civil engineers and architecture engineers but also for the general public.

In this study, the authors investigated the slope protection by ground anchor as a countermeasure against disaster from the Tokai earthquake, choosing as the subject the Hayakawa section of Tokaido line which suffered heavy damage from Kanto earthquake.⁽¹⁾

2. Outline of slope

It is the slope of the area about 60,000 m² angle of inclination about 40°~45° which is located in about 1 km of south of the Tokaido line Hayakawa station, as the location shows it to the **Fig.2**.

This area is the east side piedmont of a Hakone volcano. The hill or mountainous region which incline loosely on the sea side continue and the coast line runs almost north and south. The Tokaido line, the Shinkansen and national road number 135 line go along the coast line.

The topography of the beach neighborhood is the comparatively urgent incline where bumps are a little. This topography is an old sea cliff. The height of the plateau is within the above sea level of 100 m. These hills are dissected by the small and medium size river whose resource is emitted to Hakone volcano somma (Mt. Shirogane and Mt. Taikan). The landslide occurred to, at the time of the Kanto earthquake (September 1st, 1923) in Shiraitogawa of the Nebukawa area in these rivers. The accident that the standing train at the Nebukawa station was washed away by the landslide and fell into the sea, occurred. The wooden house damage rate in the Kanto earthquake⁽¹⁾ is shown to the **Fig.3**.

The main problem of the case that selects the slope protection is as follows. The condition and also the distribution of the geology of the ground are indistinct. Also, the scale of the anticipated collapse is indistinct.

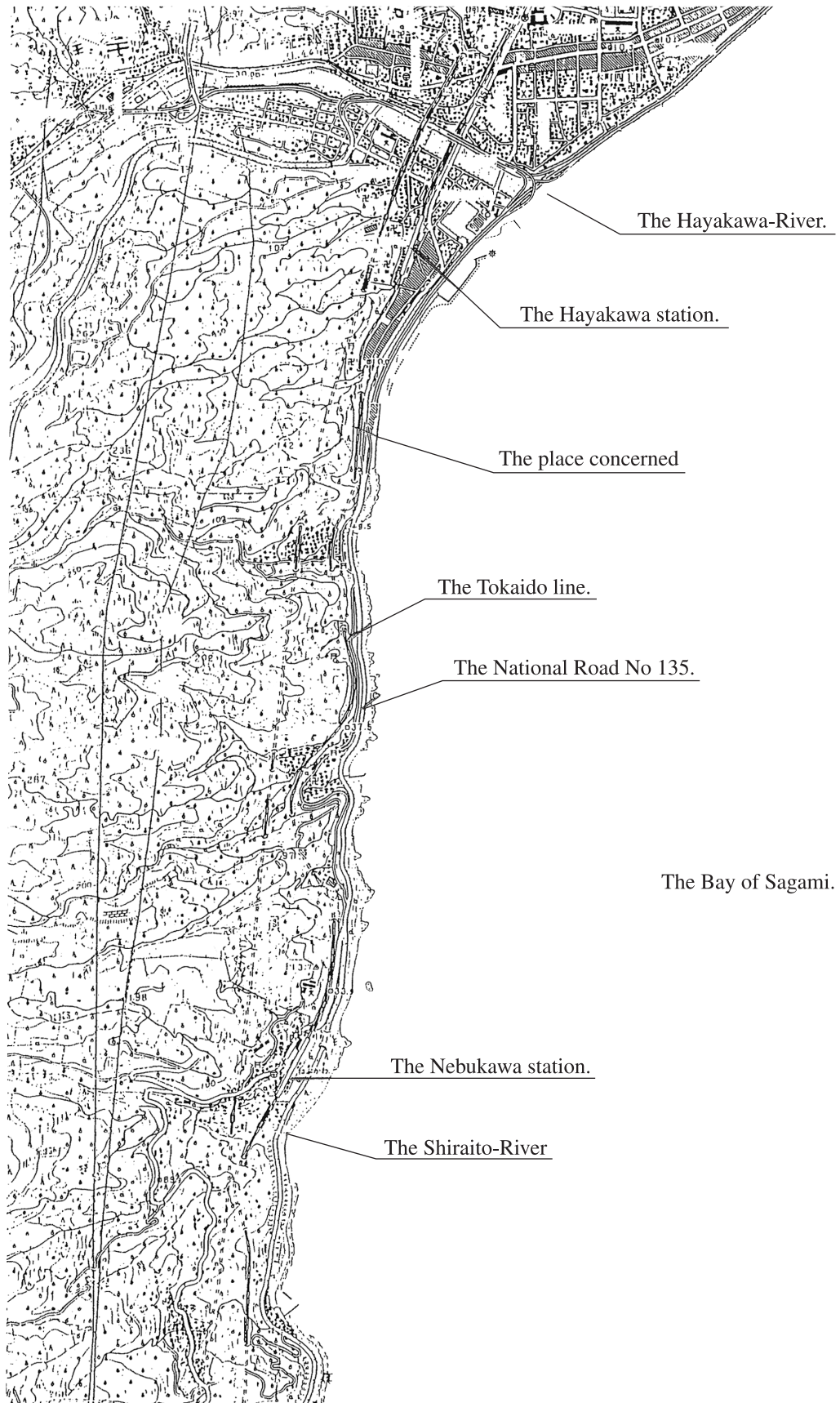


Fig.2 The position figure

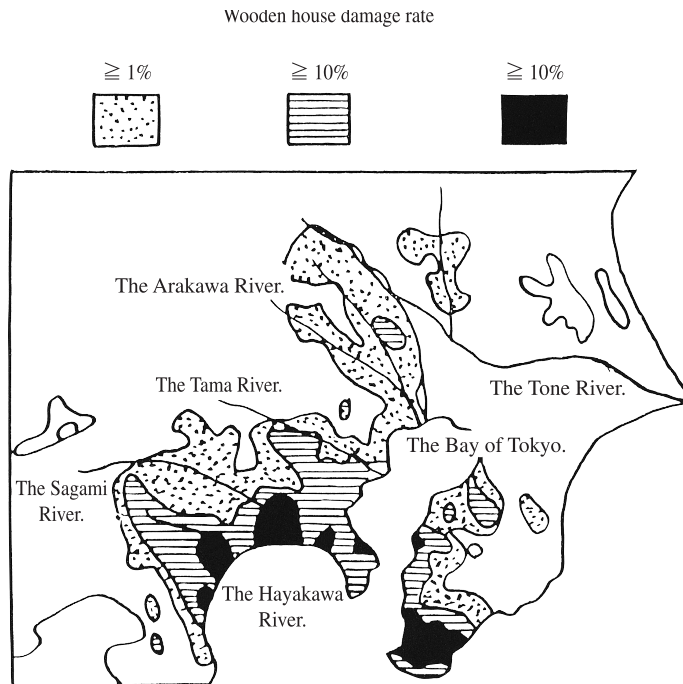


Fig.3 Wooden house damage rate in Kanto earthquake⁽¹⁾

3. Survey

The authors have done such a survey that selected the countermeasure about the slope regarded as the object.

3.1. Contents

3.1.1. Surface of the earth geology reconnaissance and exploration

This reconnaissance and exploration makes the survey of the geology constitution, the geology structure, the weathering condition of the base rock, the distribution of instable soil and sand and the surface topography (collapse, creep ~ landslide and erosion front etc.) main purposes.

3.1.2. Boring survey

The purpose of this survey is the grasp of the situation of the surface soil and sand, the confirmation of the base rock depth and the grasp of the base rock weathering situation. The depth makes 10 m. The authors have

done the standard-penetration test at the time of the boring survey.

3.1.3. Velocity logging in hole

The authors have done the velocity logging by utilizing the above boring hole.

We estimate the physical situation of the stratum from the elastic wave (the P wave) velocity. This physical situation, in other words weathering ~ crush spalling situation of the stratum, did not appear in the N-value of the standard-penetration test.

As the working method we observed the vibration by the shock of the surface of the earth through the receiver that was inserted into the boring hole.

3.1.4. Seismic prospecting

We have done the seismic prospecting, for we compared the surface of the earth geology reconnaissance and exploration result with a proper slope and the boring survey result, by grasping the elastic wave speed

distribution between two sections 86k870 m and 87k320 m from Tokyo Station. (86k900 m section from Tokyo Station = No0 section.)

This survey is aimed to grasp the thickness of layer and distribution of instable soil and sand and obtain the foundation data on the slope countermeasure design. We have carried out it with the line arrangement that showed this survey to the **Fig.4**.

The investigation outline is as follows.

- ① The line. A-Line 275 m, B-Line 50 m, C-Line 50 m.
- ② The oscillating method. By the shock of the hammer.
- ③ The receiver interval. It is 5 m.

3.2. Results

3.2.1. Surface of the earth geology reconnaissance and exploration

The Hakone volcano lava of the Quaternary Period is distributed widely in this area. The Hakone ancient times period somma lava are distributed among Odawara ~ Manazuru. It consists of plate state joint basalt quality andesite of unhypohyaline that progressed or the tuff breccia of the same quality to the andesite of porphyritic that are mainly represented to "the Nebukawa stone".

On the other hand the Yugawara volcanic rocks of the subordinate position than this distribute in the Yugawara area and the andesite of a little bit many hypohyaline is main.

Thin Kanto loam formation is covers either of them and these Kanto loam thickness of layers is around 1 ~ 5 m. Most of the Kanto loam formations has turned the soil.

The house tuff breccia of the above ancient times period somma lava is dark ash-color and include andesite gravel ($\Phi 10 \sim 20$ cm) of the same quality. Also the gravel state part that burnt to the light brown in the periphery department of the andesite lava is observed and from the viewpoint of the engineering the natural state is similar to the tuff breccia. These andesite lava and tuff breccia are a sandwich state. The layers of andesite lava and tuff breccia lean in the east side loosely.

3.2.1.1. Slope conditions

The falling rock protection fence was made in the slope mountainside by using the rail and the leaning type retaining wall was established going along the permanent way.

The topography consists of the steep slope directly than from the permanent way. The elevation 90 m over is mandarin (Japanese orange) field (terraced fields) of a loose dip.

The angle of inclination of the slope is $40^\circ \sim 45^\circ$ and there are over 50° parts. The top half department of the slope is a little bit loose more the half department under the slope in the same section. There is the erosion front in the upper part or mountainside department between No3 section and No14 section. The upper part between No8 section and No12 section has the complex topography that has a difference in level.

The clear mountain stream is not admitted. The accumulation is a little bit thick in the concavity slope of No1 section neighborhood, No15 section neighborhood and No20 section neighborhood. The slope of the No15 section neighborhood is the old landslide topography. There is spring water constantly in the top part of the slope. Two gully erosions are produced by the outflow. The running water continues to the drainage in the gully erosion of the Tokyo direction. There is running water only half of neighborhood of the channel in the gully erosion of the Atami direction. It is infiltrating to the underground under there more half of the channel in the gully erosion of the Atami direction. The thickness of layer of the colluvial deposit of this slope bottom is the 3 ~ 5 m. This colluvial deposit makes the flat topography including the boulder gravel of the biggest diameter of 150 mm. This colluvial deposit stabilizes in the present condition.

There is the collapse topography in the mountainside of the No12 section neighborhood. This is conceivable as the collapse of surface soil and sand and heavy weathering layer of the base rock. The collapse depth is a few m. The remainder product soil is accumulating to the collapse topography a little bit thickly.

The stratum of this area inclines loosely toward Tokyo and the tuff breccia distributes mainly extending from the middle of the stairs through the upper row of the slope between No1 section and No11 section. The exposure of a hard andesite is conspicuous in the lower berth of the slope.

The tuff breccia is being put to the andesite in the mountainside of the slope between No11 section and No17 section and the tuff breccia is observed well with the grid frame periphery. It is a tuff breccia in the slope lower part after the No15 section neighborhood.



Fig.4 Line arrangement figure in the seismic prospecting

3.2.1.2. Topography geology situation that has the possibility of a disaster

① The surface soil and sand and heavy weathering layer of the entrance of tunnel right side slope compose the erosion front between No0-10 m section and No0+5 m section. The surface slip style collapse of the small-scale from this erosion front neighborhood is conceivable.

② The concavity topography in the No1 section neighborhood has a little bit big collection water area. The Kanto loam formation accumulates in the slope bottom of the No1 section neighborhood thickly. However, this topography is conceivable as stability.

Because it depends on that there is a leaning type retaining wall in the exit of the concavity topography and that there is no running water at all in the concavity topography.

③ There is a little bit thick talus soil and sand in the slope mountainside among No3 section and No4+10 m section. This section is thought to be the slope of the creep nature generally. And the surface of the ground of the Tokyo direction is the andesite that has weathered to the decomposed granite soil. Also, a glance cobble form andesite outcrop lies scattered to the department on the slope of period from this to No6 section. From now on the slip and falling rock of the surface soil and sand are conceivable. However, it is considered to be prevented with the falling rock protection fence of the middle of the stairs and also lower berth whichever.

④ Between No6 section and No11+10 m section the top half department of the slope is a little loose dip than the under half department. However the erosion front where it is observed with the collapse site is continuously admitted. There is thick talus soil and sand in the slope upper end of around the No8 section. The clear surface slip style collapse site is observed in the erosion front neighborhood. The collapse soil and sand creeps under the erosion front.

Also, the loose dip ~ flat topography is seen in the erosion front upper part between No8 section and No11 section. However the movement of the collapse or landslide etc. is not seen.

⑤ The crack has occurred to a small-scale leaning type retaining wall with the slope mountainside ~ upper part between No11 section and No12 section, and we estimate this section as the slope of the creep nature. However there is comparatively little movement soil.

⑥ The surface is collapsing between No12 section and No12+15 m section and the collapse soil of this section is a little bit thick. It is conceivable from now on that the surface collapse expands upward and that the surface slips. The scale of the case is conceivable that the width is about 15 m and the length is about 15 m and the depth is about 2m.

⑦ Between No13 section and No14 section, the ground from the lower slope berth to mountainside protrudes toward the permanent way side as the topography. The top half department is loose dip (mandarin)field. The gravity type grid frame is executed from the lower slope berth to the mountainside. The geology of the gravity type grid frame execution place is a fairly fragile tuff breccia. The upper side of gravity type grid frame is a fairly hard andesite, although there is cracking a little. It is conceivable that it has already become such topography due to the erosion resistance of the andesite.

Even if there is a collapse of small-scale in the department on the slope of this section at the time of the earthquake, accordingly, the possibility that the shoulder department of the urgent cliff collapses extensively is low.

⑧ The contents with regard to the landslide topography of the No15 section neighborhood are expressed with **Paragraph 3.2.1.1.**

⑨ There is the soil and sand of talus nature in the concavity part of the leaning type retaining wall upper part slope between No16+15 m section and No19 section and conceivable as the slope that creeps. Also there is a cracking of opening nature between the upper end of the leaning type retaining wall and the channel concrete. It is conceivable that it does not collapse largely from this part. However the measure of the reinforcement etc. is necessary.

⑩ The talus soil and sand is accumulated to the respectable amount within the concavity topography in the No20+10 m section neighborhood. Especially several collapse topography exists toward Tokyo though it is small-scale. From now on, the landslide is expected to occur. However it is conceivable that the damage would not result from it to a permanent way because there are channel (there is no flow of water) and debris barrier.

3.2.2. Boring survey

Fig.5. Columnar section No-1.

The result that the authors have carried out in 2 places is shown in Table 1.

3.2.2.1. Br-No1. (87k020 m neighborhood from Tokyo Station, depth :10 m)

The authors have carried out it in the mountainside of the slope.

- ① Depth : 0 m to 1.6 m.

It is loose, black brown surface soil . The N-value is 3 or less.

- ② Depth : 1.6 m to 10.0 m.

It is the tuff breccia with andesite quality in dark white. The gravel state ~ sand state is presented to the whole.

Subangular gravel ~ circle gravel of a porphyritic andesite (A part is porous.) to diameter 3 cm ~ 5 cm is included in the middle of volcanic ash base quality of sand quality ~ silt quality in this stratum.

There is a part that is about 60 cm are the core of the short bar of the several cm between the depth 6.9 m ~ the depth 7.5 m. It is conceivable as this short core bar was the boulder gravel of originally diameter about 60 cm. Such a boulder gravel sometimes is often included in this rock phase. The N-value is 10 with the depth 3 m the neighborhood. The N-value is 50 or more in except for the depth 3 m the neighborhood.

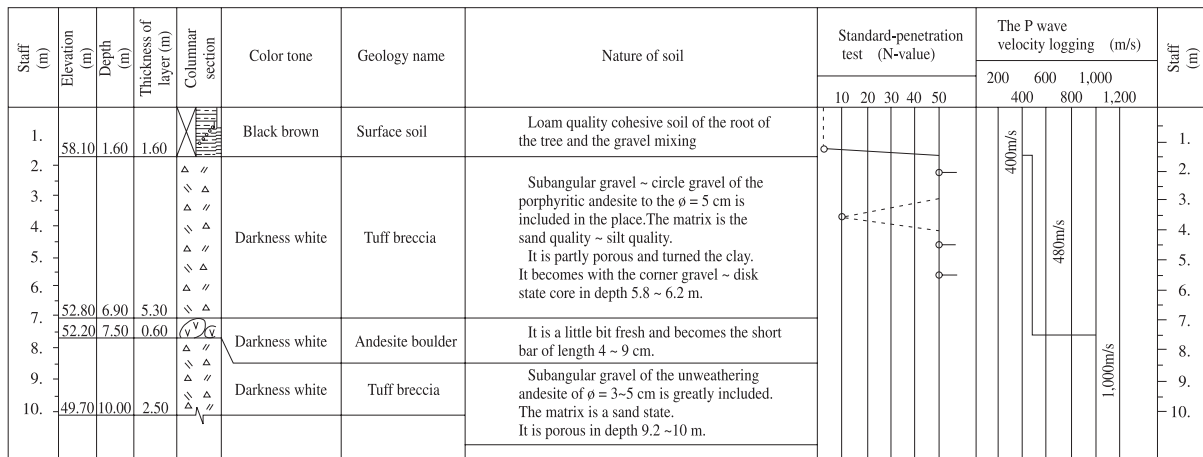


Fig.5 Columnar section No-1

Table 1 Boring Results

	1st layer	2nd layer and 3rd layer	Remarks
Boring survey	0 m~1.5 m	1.5 m~10 m	
	Surface soil	Dark ash tuff breccia	
	N-value<3	N-value>50	

3.2.2.2. Br-No2. (Tokyo Station from the 87k205 m neighborhood, depth : 10 m)

Fig.6. Columnar section No-2.

The authors have carried it out in the direct upper part of the exit and around the erosion front of concavity topography (landslide).

① Depth : 0 m to 1.5 m.

The others of the black brown surface soil of about 40 cm thick are the cobble of the andesite of porphyritic. The lower part is the weathered rock. The N-value is 50 or more.

② Depth : 1.5 m to 1.9 m.

It is the porphyritic andesite which is dark white and a little bit soft. This porphyritic andesite layer becomes the short bar of the length up to 15 cm. The lower part is converting to the light brown.

③ Depth : 1.9 m to 3.2 m.

It is the light brown andesite. The gravel (state part) partially becomes the short bar of the length 6 cm ~ (the length) 7 cm. The rest shows the sand state. The N-value is 50 or more.

④ Depth : 3.2 m to 10.0 m.

It is the dark white tuff breccia. The sand state ~ gravel state is shown. The subangular gravel ~

subrounded gravel of the andesite of diameter less than 12 cm is included into the volcano base quality of the sand quality. The part of the depth 9.0 m ~ (the depth) 9.6 m is a little bit fragile. Around 5 cm core of short pillar has continued in this part. It is conceivable that this is similar to Br-No1 as the boulder gravel. The part of the depth 9.6 or more m is the same quality as that in the upper part. However it is observed as there is more gravel quantity a little than expected. The N-value is 30 in the upper part. The N-value is 50 or more except for the upper part.

3.2.3. Velocity logging in hole

P-wave velocity logging was conducted, making use of the boreholes. The results are given in **Table 2**.

3.2.3.1. Br-No.1(87,020 m neighborhood from Tokyo Station, depth :10 m)

The P-wave velocity is less than 400 m/s in surface soil(0-1.6 m deep), 480 m/s in tuff breccia at depths up to 7.5 m and 1,000 m/s at depths of 7.5-10.0 m.

There is no significant difference in velocity between surface soil and weathered rock at depths up to 7.5 m.

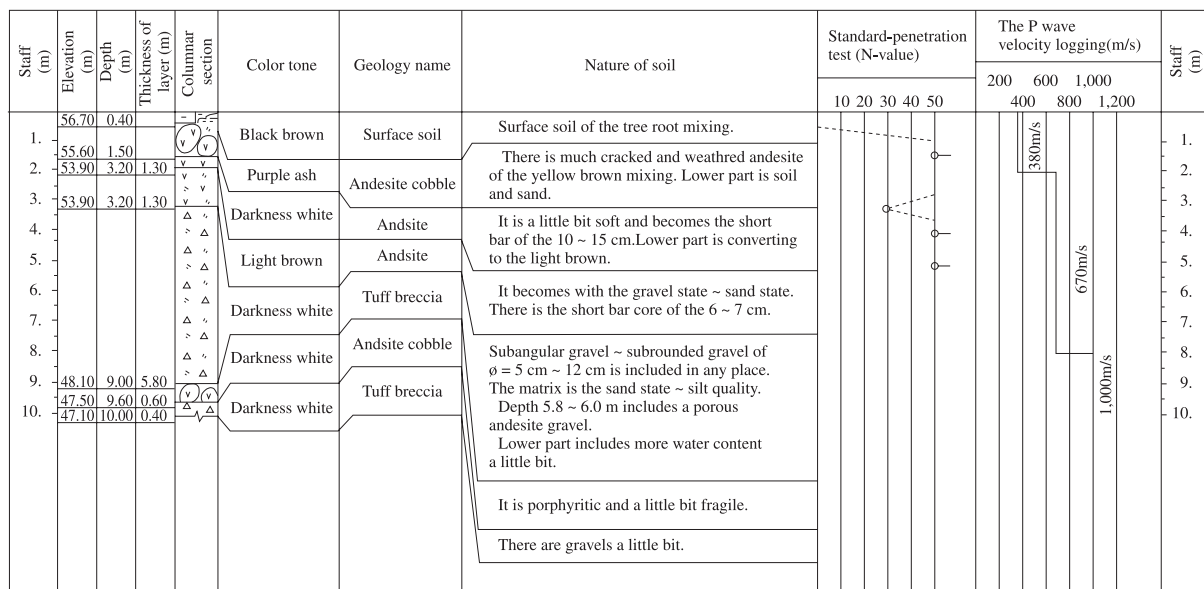


Fig.6 Columnar section No-2

3.2.3.2. Br-No. 2 (87,205 m neighborhood from Tokyo Station, depth :10 m)

The P-wave velocity is 380 m/s in surface soil and cobble layer(0-2.0 m deep), 670 m/s in the intermediate layer of andesite and tuff breccia (2.0-8.0 deep) and 1,000 m/s in the lower part of tuff breccia(8.0-10.0 m deep).

3. 2. 4. Seismic prospecting

The values of elastic wave velocity in the survey area can be classified as shown in **Table 3**, based on the results of the geological reconnaissance and exploration

of the ground surface and of the boring survey.

The survey results are described below with reference to **Table 3**.

① 1st velocity layer (300-450 m/s)

This layer is a heavily weathered part of base rock. It is composed of sandy and gravelly cores containing much surface soil or clay of unconsolidated deposits of tuff etc. Distribution of this layer is detected on all of the lines.

The unconsolidated part and weathered part of base rock are distinctly different in geology as seen from the columnar section obtained by the boring survey. However, it is judged that 2 layers are physically equal and also be equal in strength per unit area from the results

Table 2 Classification of Results of Velocity Logging in Hole

	1st layer	2nd layer	3rd layer	Remarks
No1	0 m~1.6 m v = ~400 m/s	1.6 m~7.5 m v = 480 m/s	7.5 m~10 m v = 1,000 m/s	
No2	0 m~2.0 m v = ~380 m/s	2.0 m~8 m v = 670 m/s	8 m~10 m v = 1,000 m/s	
	Surface soil	Tuff breccia	Tuff breccia and andesite	

Table 3 Classification of Results of Seismic Prospecting (Unit: m/s)

	1st layer	2nd layer	3rd layer	Remarks
A-Line	300~400	550~900	1,100~1,300	Main line
B-Line	300~400	700~800	-	Sub-line
C-Line	350~450	800~900	-	Sub-line
	Heavily weathered part of base rock composed of sandy and gravelly cores containing much surface soil or clay of unconsolidated deposits of tuff etc.	Heavily weathered part of base rock composed largely of sandy and gravelly core of tuff breccia or andesite lava.	Heavily weathered part of the base rock of tuff breccia or andesite lava.	3rd velocity layer detected on main line only. Its depth is GL 15 ~20m. 3rd velocity layer not determined by boring survey.

of the seismic prospecting and velocity logging.

The velocity of 300-450 m/s in this layer is considered appropriate as the elastic wave velocity in the ground, compared with the values of 380-670 m/s obtained by the velocity logging in hole.

The thickness of this layer is generally 5-10 m in the survey area. The layer thickness is about 4 m in the steep slope in the vicinity of A-0 m. While in the vicinity of A-65 m, the former site of collapse, and in the vicinity of 0 m (retaining wall) on the B- and C-lines, the layer thickness is about 1 m.

② 2nd velocity layer (550-900 m/s)

The 2nd velocity layer corresponds to the heavily weathered part of base rock composed of sandy and gravelly cores of tuff breccia or andesitic lava. Between A-90 m and A-175 m, somewhat lower velocity values like 550-600 m/s are detected. It is judged from this fact that weathering has progressed further in this layer.

The velocity in this layer is considered appropriate as the elastic wave velocity in the ground, compared with the value of 1,000 m/s obtained by the velocity logging in hole.

The thickness of this layer is 8-10 m on the A-line. On the sub-lines (B and C), the 3rd velocity layer below this layer is impossible to detect from the extensions of the survey lines. The layer thicknesses on the B- and C-lines are estimated at about 15m and 10 m, respectively, from the intersections with the A-line.

③ 3rd velocity layer (1,100-1,300 m/s)

This layer is detected only on the A-line as described in ② above. Entered in the "sub-line" columns are the values of velocity and depth at the intersections of each sub-line with the A-line.

From the values of elastic wave velocity and the results of the geological survey of the ground surface, this layer is assumed to be a weathered part of tuff breccia or andesitic lava. The depth is 15-20 m from the ground surface. The geological conditions of the layer have not been determined by the this boring survey.

4. Investigation of Countermeasures

Based on the ground surface conditions and geologic features, this area is divided into four blocks as shown in Fig.7. Countermeasures for each of these blocks are investigated below.

4.1. Block I (between No. 0 and No. 1+9 m sections)

Fig.8 I block representative section

4.1.1. Site conditions

In this block, Countermeasures are already provided on the entire slope. That is, a leaning type retaining wall is built there from the formation of the permanent way. A rockfall prevention fence is set up above the retaining wall. In a 20 m long section of the leaning type retaining wall, no rockfall prevention fence is provided. Concrete is sprayed on the slope surface in the elevation range from 35 to 45 m. A leaning type retaining wall is provided in a section nearer to Atami. A leaning type retaining wall is also placed on the top of the concrete-sprayed surface.

From the tunnel exit to the vicinity of the No. 0+5 m section, there is a continuous steep slope running up from the uppermost retaining wall to the level of 10 m above. A somewhat flat plateau exists above the steep slope.

Between the No. 0+5 m and No. 1+9 m sections, a retaining wall is provided on the erosion front. The topography above the erosion front is a concave having a relatively large water collection area. In the event of a downpour of rain, however, water running from the weep holes made in the retaining wall and slope can hardly be confirmed.

The slope is geologically composed of porphyritic andesite containing tuff breccia that is somewhat fragile compared therewith. The location of the concrete-sprayed surface and leaning type retaining wall is considered to have been a cut slope at the time of construction of the tunnel opening. The slope is stable as far as the vicinity of the upper retaining wall.

In the plateau above the slope, a 1-2 m thick Kanto loam layer is deposited on porphyritic andesite.

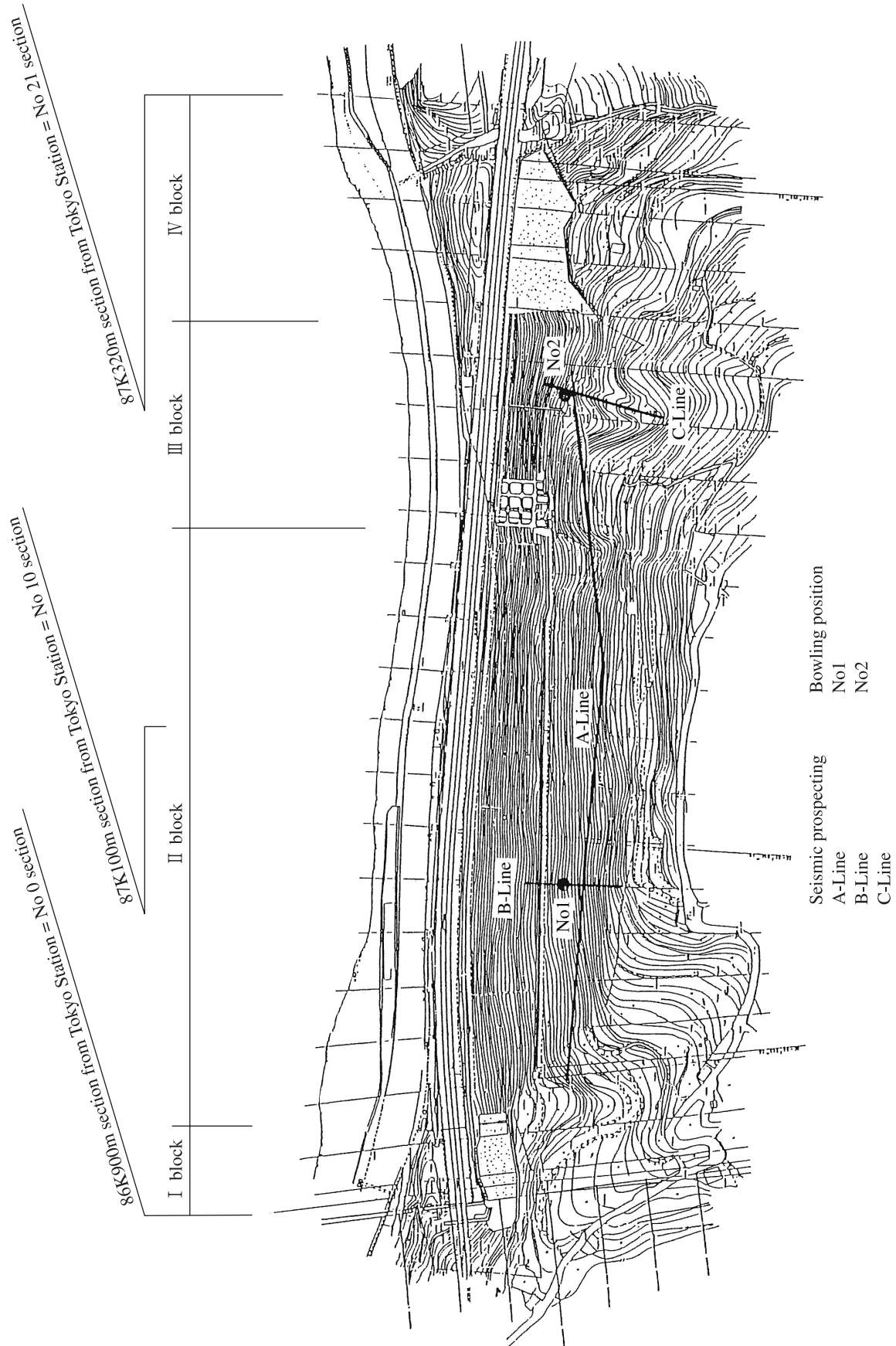


Fig.7 Classification figure

4.1.2. Disaster mechanism and scale of disaster

Between the No. 0-10 m and No. 0+5 m sections, the surface soil and heavily weathered layer of rock in the slope above the upper retaining wall are expected to collapse when a large-scale earthquake occurs. This collapse is considered to be small in scale, being confined to the surface layer of the slope. The margin behind the retaining wall placed above is limited. Therefore, it is readily expected that even a small-scale col-

lapse will cause fragments of rock etc. to get over this existing retaining wall. The slope below the retaining wall is steep, so a rockfall prevention fence is set up by the side of the permanent way. However, it is considered highly possible from the slope gradient and surface irregularities that the soil and cobbles getting over the existing upper retaining wall will also get over the rockfall prevention fence beside the permanent way, resulting in directly hitting the permanent way.

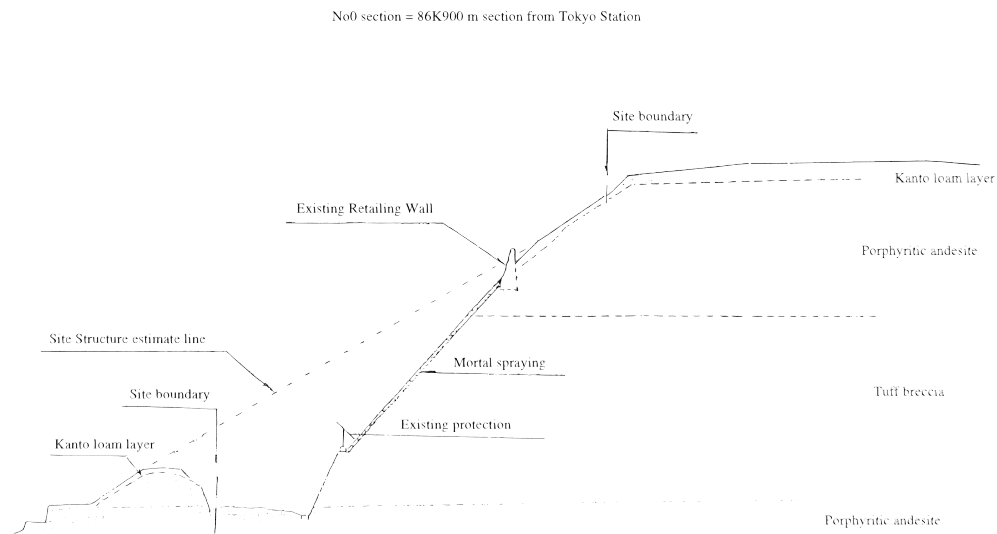


Fig.8 I block representative section

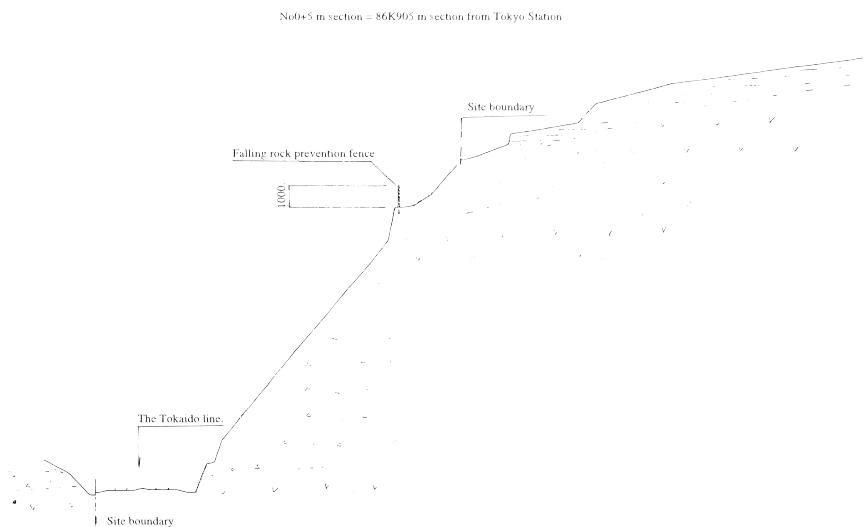


Fig.9 I block falling rock prevention fence

4.1.3. Policy for selecting Countermeasures

The cutting is done from the existing retaining wall department neighborhood through lower part of the upper part at the time of the permanent way construction, when we conjection from the topography of the neighborhood in this block. The stability of the slope protection that is established along with this cutting is high. Accordingly, the measure is to prevent the bad influence to the permanent way at the time of the surface collapse of unmeasure slope of the upper part from an existing retaining wall.

Even a small-scale collapse will cause the cobbles etc. getting over the existing retaining wall to directly hit the permanent way. Therefore, it is necessary to select measures to prevent collapsed soil etc. from getting over the existing retaining wall.

Measures to be taken are as follows:

- ① Measures to prevent occurrence of collapse
- ② Measures to prevent progress of collapse

Subject to collapse is the very thin part of the surface layer. From the viewpoint of the possible scale of disaster, therefore, methods to stabilize the entire slope, e.g., ① cut + concrete spraying and ② grid frame +

ground anchor, are considered excessive. Only the cobbles or soil caused by collapse to get over the retaining wall will have a great energy by the side of the permanent way. This energy is small in the vicinity of the upper retaining wall.

In conclusion, the rockfall prevention fence on the top of the retaining wall will be excellent in both effectiveness and cost efficiency.

Fig.9 I block falling rock prevention fence

4.2. Block II (Between No. 1+9.0 m and No. 12+16 m sections)

Fig.10 II block representative section

4.2.1. Site conditions

In this block, a retaining wall and a rockfall prevention fence are provided by the side of the permanent way. Between the No. 5+15 m and No. 12+16 m sections, there are a leaning type retaining wall and a mortal-sprayed surface. However, either of these slope protections is fairly old and exhibits conspicuous cracks and exfoliations. The slope is thickly wooded with trees and other vegetation.

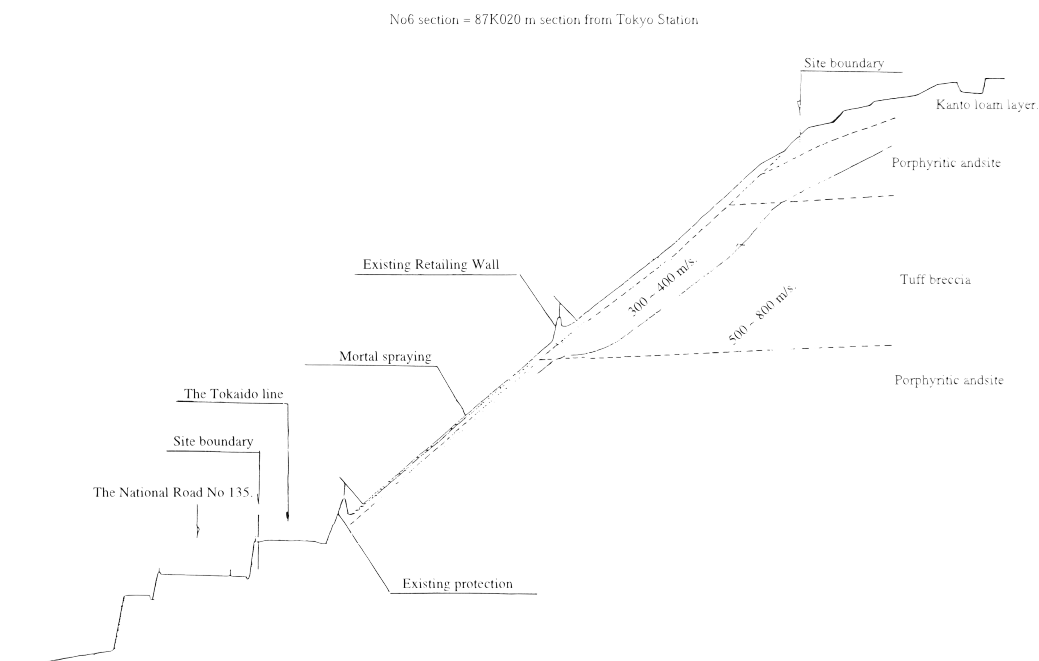


Fig.10 II block representative section

At the elevation of about 50 m, there is a 5 m high rockfall prevention fence beneath which a retaining wall is provided. The rockfall prevention fence nearer to Tokyo from the vicinity of the No. 10+10 m section is new. The rockfall prevention fence, about 80 m long, nearer to Tokyo from the No. 6 section was constructed in March 1982.

The topography here is a slope with an average gradient of 40-45°. At elevations above 70-80 m, there are gently sloping mandarin fields.

Below the elevation of about 50 m, the slope is composed of stable porphyritic andesite. Though there is some tuff breccia made fragile by the action of consolidation from the upper part, the slope is stable as a whole. On the other hand, the upper slope is composed largely of more fragile tuff breccia than the lower slope. Above the elevation of 80 m, porphyritic andesite is distributed. The layer of tuff breccia is thick and occupies almost all parts of the steep slope.

On the slopes in the vicinity of the No. 2+10 m. in the vicinity of No. 4+10 m and between the No. 11 and No. 12+15 m sections, deposits from a creep type talus are distributed. Especially the slope between the No. 11 and No. 12+15 m sections is considered to be the former sites of collapse 2-4 m thick.

4.2.2. Disaster mechanism and scale of disaster

The upper and the lower slopes are geologically different. In this block, therefore, it is presumed that disasters different in mechanism and scale will take place when a large-scale earthquake occurs.

4.2.2.1. Lower slope

The lower slope is composed largely of stable porphyritic andesite. It is expected that collapse or rockfalls from the heavily weathered part of the surface layer will take place. Presumably, a small-scale collapse about 1 m in depth will occur.

4.2.2.2. Upper slope

The layer of 300-400 m/s in elastic wave velocity continues to the depth of about 8 m as seen from the results of the seismic prospecting. This layer is described below as the "weathered rock layer." The weathered

rock layer is equivalent to soil. On the other hand, the collapse face is assumed to have been 2-3 m deep, from the former site of collapse in the vicinity of the No. 12 section.

In light of the former site of collapse in the vicinity of the No. 12+10 m section, it is assumed that the 1st velocity layer will scarcely collapse as a whole at a time. On the other hand, it is likely that a shear plane may be formed in the vicinity of the interface between the 1st and 2nd velocity layers when a large-scale earthquake occurs.

In this upper slope, the area between the No. 5 and No. 12 sections is considered to involve a high risk of collapse. In the middle part of the slope, a rockfall protection fence (5.0 m. high) is set up. Part of the foundation of this fence is not extended to the rock layer. Therefore, it is presumed that the framework of the fence will become unstable when an earthquake occurs.

4.2.3. Stability analysis of the existing section

Subject to stability analysis are two sections, upper and lower. The No. 4 section is selected as the lower section. In this section, the thickness of deposit soil in the slope is greater than those in other unprotected slopes. The No. 6 section is selected as the upper section.

4.2.3.1. Lower slope

In the lower slope, the depth of collapse is estimated at about 1 m. Collapsed soil is thick between the No. 1+9 m and No. 5+15 m sections. In other parts (between No. 5+15 m and No. 12+16 m sections), surface soil is very thin. The type of collapse is surface collapse. Most of the rockfalls are from the exposed surface of the heavily weathered rock layer. Cracks and exfoliations are conspicuous on the existing slope protections. However, the existing slope protections are expected to have a certain prevention effect, so the necessity to take countermeasures for this slope is inferior in priority order to that for other slopes.

The procedure to carry out a countermeasure is as follows. That is, first remove the deteriorated parts of the existing slope protection, then remove the remainder of surface soil and finally provide the same type of protection as the former one.

4.2.3.2. Upper slope

The analysis is conducted by the slip circle method, taking the three following layers as sliding surfaces. Here, it is assumed that a horizontal force ($w \times kHz$) will act on slip clods when an earthquake occurs:

- ① Surface layer (loam layer)
- ② Surface layer (loam layer) + 2 m (described below as the "intermediate layer").
- ③ Weathered rock layer

As for seismic intensity, the following values are adopted according to the Explanation of the Aseismic

Design Guidelines (draft) ⁽²⁾:

Horizontal seismic coefficient (k_h) = 0.20

Vertical seismic coefficient (k_v) = 0

As for soil constant, the values of **Table 4** are adopted, based on the soil test results and the Structural Design Standards ⁽³⁾

The safety factor not under earthquake is described as F_s . The safety factor under earthquake is described as F_{se} .

The results are given in **Table 5**.

From the slip circle analysis, the collapse face is assumed to be an intermediate layer about 2 m deep.

Table 4 Soil Constant

	γ_d		C		ϕ
	(t/m ³)	(KN/m ³)	(t/m ³)	(KN/m ³)	(°)
Kanto loam layer	1.8	17.66	1.0	9.81	30.0
Collapse soil	1.8	17.66	1.0	9.81	30.0
Weathered rock layer	2.0	19.62	2.0	9.81	40.0

Table 5 Analysis by the slip circle method

Case	Kanto loam layer				Intermediate layer				Weathered rock layer			
Sketch												
Unit	(t/m3)	(KN/m3)			(t/m3)	(KN/m3)	(t/m3)	(KN/m3)	(t/m3)	(KN/m3)	(t/m3)	(KN/m3)
γ_d	① 1.8	17.6	-		① 1.8	17.6	② 1.8	17.6	① 1.8	17.6	② 1.8	17.6
C	1.0	9.8	-		1.0	9.8	1.0	9.8	1.0	9.8	1.0	9.8
ϕ	30.0		-		30.0		30.0		30.0		30.0	
F_s	1.20				1.00				1.30			
F_{se}	0.91				0.75				1.00			

4.2.4. Countermeasures

4.2.4.1. Lower slope

Surface collapse from the lower slope is difficult to prevent by the side of the permanent way for the following reasons:

- ① As the margin by the side of the permanent way is limited, the topography here is a steep slope rising directly from there.
- ② Obstacles are too many to provide a countermeasure.
- ③ Providing a countermeasure results in forming a large-scale structure.

Therefore, the following are proposed to prevent surface collapse from the lower slope:

- ① Measures to restrain unstable parts
- ② Measures to remove unstable parts

Based on these measures, the following countermeasures are planned for slope protection:

Plan ① : Grid frame + ground anchor

Install grid frames on the entire slope and tension the grid frames and ground by ground anchors.

Plan ② : Cut + concrete spraying

Remove unstable parts and perform concrete spraying.

Plan ③ : Gravity type grid frame

Plan 1 prevents collapse by means of ground anchors, while this plan prevents collapse by the dead weight of grid frames.

A comparison of the countermeasures for the lower slope of Block II is shown in Table 6.

4.2.4.2. Upper slope

Table 7 shows the selected countermeasure for the upper slope.

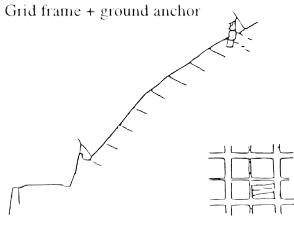
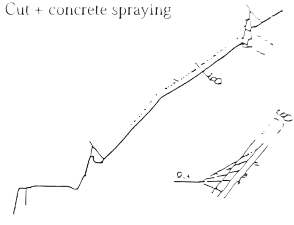
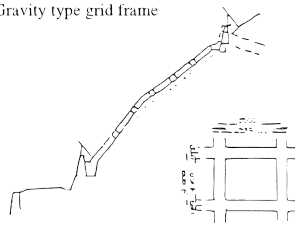
4.3. Block III (Between No. 12+16 m and No. 17 sections)

Fig.11 III block representative section

4.3.1. Site conditions

In this block, a rockfall prevention fence is set up by the side of the permanent way as in the case of Block II. Between the No. 12+16.0 m and No. 13+15 m sections, gravity type grid frames 1.0 m in girder width

Table 6 II block lower part slope counter measure comparative table. (Unit: 1,000 yen)

	1st plan	2nd plan	3rd plan
Sketch	Grid frame + ground anchor 	Cut + concrete spraying 	Gravity type grid frame 
Execution nature	Because there is not a margin in the permanent way side, the ground anchor execution of the lower berth is difficult.	Slope cutting is executed by a human power.	Establishment of the form is difficult due to cast in place concrete.
Cost efficiency	Grid frame 74,575 Anchor 33,600 Scaffold 14,965 Total. 123,140	Concrete spraying 29,150 Cut 16,328 Total 45,478	Concrete 40,200 Form 30,807 Reinforcing rod 804 Cut 1,376 Scaffold 2,788 Total 75,975
Structure	It becomes the structure of high stability. However, it seems too excessive for the disaster scale.	It becomes stable to remove the resource of disaster occurrence.	There is a sense of security. The foundations need to reach to the rock.
Maintenance control	Maintenance control is easy and we can plant trees inside the grid frame.	As for maintenance control, there is not a problem especially. However, it lacks the fine sight, as trees can not be planted.	Maintenance control is easy and we can plant trees inside the grid frame.
Evaluation	△	◎	○

are installed as far as the middle part of the slope. A leaning type retaining wall or a concrete-sprayed surface is provided on other slopes. However, these structures are considerably deteriorated, exhibiting cracks and exfoliations.

The topography here is a steep slope rising directly from beside the permanent way. Between the No. 12+16.0 m and No. 16 sections, the slope is the steepest with a gradient exceeding 50° . (The gradient in the vicinity of the No. 15 section is about 55° .) Between the No. 12+16.0 m and No. 14+10 m sections, there are gently sloping mandarin fields above the elevation of about 62 m.

Between the No. 14+10 m and No. 16 sections, the ground above the elevation of 55 m shows a concave topography with gently sloping Japanese cedar fields.

The landslide topography in the vicinity of the No. 15 section is described in **Paragraph. 3.2.1.1.**

The geology here is an alternation of strata of porphyritic andesite and tuff breccia. However, somewhat fragile tuff breccia (including andesitic lava) is distributed on the lower slope, and hard andesite is exposed on the upper slope.

The gravity type grid frames in the vicinity of the No. 13+10 m section are assumed to have been reinforced after the surface collapse of fragile tuff breccia. On the upper, gently sloping part in the vicinity of the

No. 15 section, hard andesite is distributed in a very thin layer. Colluvial deposits by landslide are distributed on the lower layer of tuff breccia.

4.3.2. Disaster mechanism and scale of disaster

As in the case of Block II, the rock layer corresponding to tuff breccia is expected to collapse due to weathering. In this block, the lower stratum is a little compacter than tuff breccia in Block II. The velocity value obtained by velocity logging is as high as 670 m/s, compared with the value of 480 m/s in Block I. Between the No. 14 and No. 16 sections, however, the layer of andesitic lava in the upper part is very thin. There is the possibility that the rock layer will collapse due to weathering, with the layer of 300-400 m/s velocity by velocity logging as the collapse face. When the rock layer collapses, it will be unavoidable for the permanent way to be directly hit by falling rocks.

Spring water is constantly flowing out to the two gullies in the former site of landslide in the upper part. The gullies are deeply hollowed out. Collapsed soil may become unstable due to an abnormal outflow of spring water when a local torrential rain or an earthquake takes place. Moreover, the surface layer of the lower slope is likely to collapse as in the case of Block II.

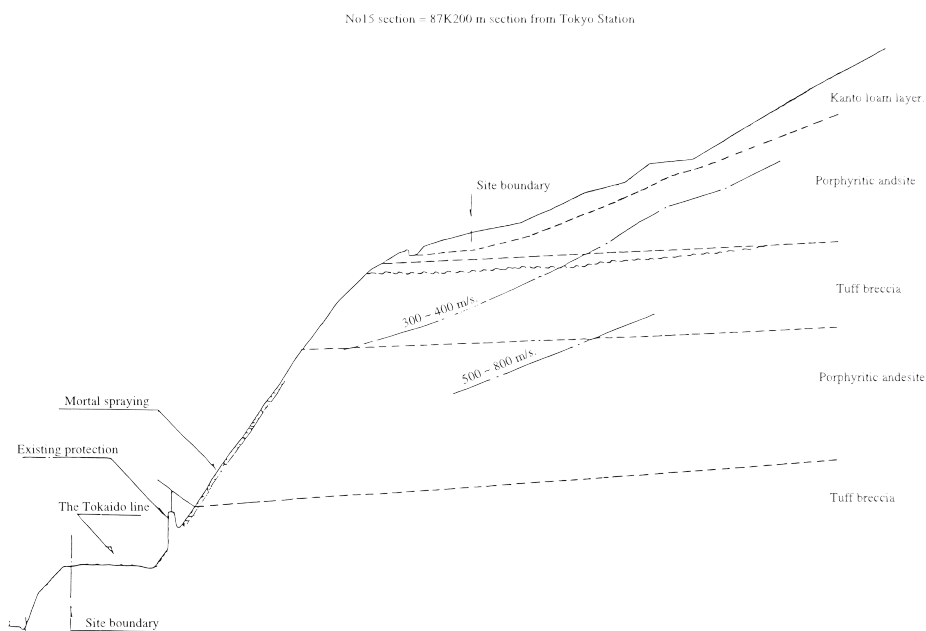


Fig.11 III block representative section

4.3.3. Stability analysis of the existing section

The analysis is conducted by the slip circle method, using the No. 15 section as a typical section for analysis and taking the unstable layer of tuff breccia as the sliding surface. Here, it is assumed that a horizontal force ($w \times kh$) will act on collapsed clods when an earthquake occurs.

As for seismic intensity and soil constant, the values shown in **Paragraph. 4.2.3.2** are adopted. The results are given in **Table 8**.

4.3.4. Countermeasures

Possible main factors leading to disaster in Block III are as follows:

- ① Surface collapse of the slope
- ② Abnormal outflow of ground water from the concave slope

As the margin by the side of the permanent way is limited, surface collapse of the slope is difficult to prevent there as in the case of other blocks.

Lowering the level of ground water is an appropriate selection for preventing abnormal outflow of spring water.

Therefore, countermeasures are as follows:

Table 7 Counter measure for 2 block upper slope

Dimension of grid frame	Girder section	400mm × 400mm
	Girder span	4.500 m
Slope length		185 m
Anchor		420sets
Total anchor length		2,100 m
Construction cost		303,980 × 1,000 yen

4.3.4.1. Measures against collapse

Plan ① : Concrete spraying + cut of upper part + (grid frame + ground anchor)

For the lower slope, remove the surface layer and perform concrete spraying (200 mm thick). For the upper slope, cut soil by 2 m and tension the grid frames and ground by ground anchors.

Plan ② : Grid frame + ground anchor

In place of cut in ① above, install girders of the same type as that for the upper part from beside the permanent way.

Plan ③ : Gravity type grid frame

Plans ① and ② prevent collapse by means of ground anchors, while this plan prevents collapse by the dead weight of grid frames.

A comparison of the countermeasures for Block III is shown in **Table 9**.

4.3.4.2. Measures against spring water

Horizontal boring is planned to lower the level of ground water as described in the above "Countermeasures."

Table 8 Stability Analysis of Section in Block III

Fs	1.29
Fse	1.00

A Study on Slope protection by Grand Anchor for the Toukai Earthquake

Table 9 III block slope counter measure comparative table. (Unit: 1,000 yen.)

	1st plan	2nd plan	3rd plan
Sketch	Concrete spraying + cut of upper part + (grid frame + ground anchor) 	Grid frame + ground anchor 	Gravity type grid frame
Execution nature	Cut of about 700m. by the human power is necessary. the period of the execution is a little bit longer than the w plan.	Execution nature is more excellent than the q plan or e plan, because there is not a slope cutting.	Establishment of the form is difficult due to the cast in place concrete.
Cost efficiency	Grid frame 12,325 Anchor 12,600 Scaffold 6,003 Concrete spraying 13,130 Cut 5,432 Total 49,490	Grid frame 40,587 Anchor 12,600 Scaffold 6,003 Total 59,190	Concrete 74,460 Form 20,997 Reinforcement 14,880 Cut 8,528 Scaffold 1,700 Total 120,565
Structure	It becomes the structure of high stability. However, it seems too excessive for the disaster scale.	It becomes stable structure to remove the resource of disaster occurrence.	There is a sense of security. Foundations need to reach to the rock.
Maintenance control	Maintenance control is easy. However, it lacks the fine sight.	Maintenance control is easy. However, it lacks the fine sight.	Maintenance control is easy and we can plant trees inside the grid frame.
Evaluation	◎	○	△

No18 section = 87K260 m section from Tokyo Station

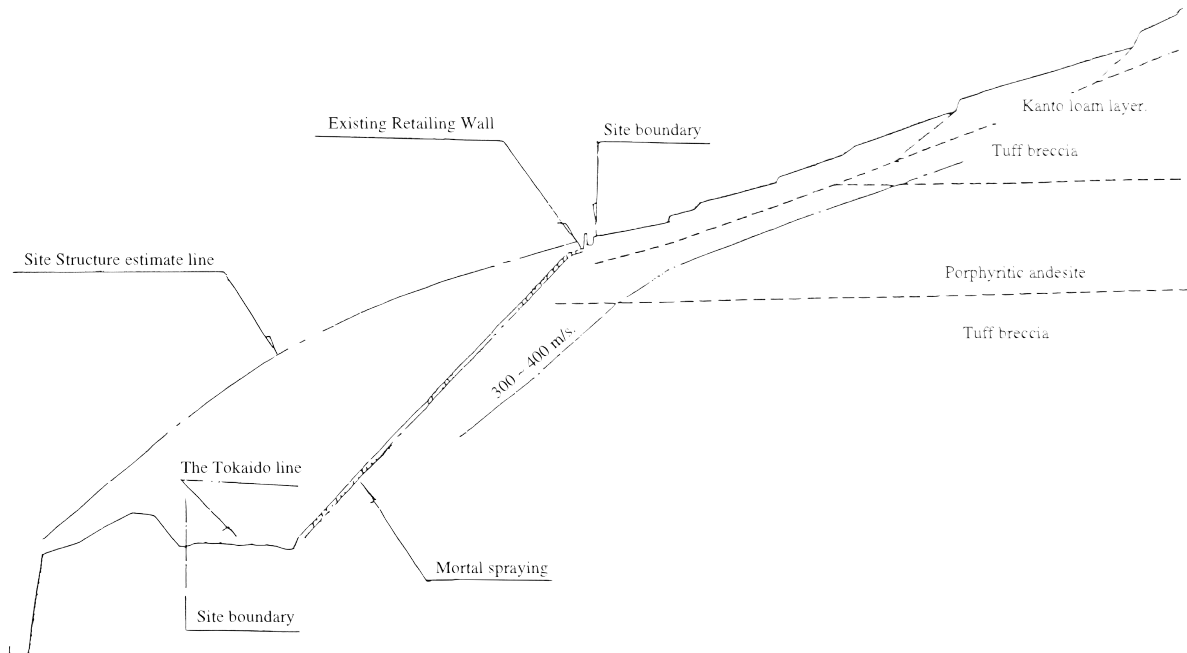


Fig.12 IV block representative section

4.4. Block IV (Between No. 17+12m and No. 19+2 m sections)

Fig.12

4.4.1. Site conditions

In Block IV, a leaning type retaining wall is provided over the entire slope from beside the permanent way as in the case of Block I. Drainage is provided at the top of the leaning type retaining wall.

The topography here is a steep slope rising from beside the permanent way as in the case of other blocks. The gradient of the slope is 45° on average. There are some areas with a gradient of 55-56°. In the area above the existing drainage, there are gently sloping mandarin fields. The slope between the No. 18 and No. 19 sections is somewhat concave.

The geology of the slope is for the most part composed of andesitic tuff breccia. From the topography of the surroundings, this block is assumed to have been a fairly large cut slope at the time of construction. The ground and the existing slope protections are fairly stable.

The slope protection nearer to Block III is thickly wooded with trees and other vegetation and exhibits cracks. On the other hand, the foundation of the slope protection nearer to Block IV is extended to base rock, so this structure is stable.

Porphyritic andesite is distributed over tuff breccia. An about 2 m thick layer of Kanto loam is deposited on the gently sloping mandarin fields.

The existing drainage exhibits cracks in the vicinity of the No. 18 section. The results of the observation made since several years ago has shown no abnormality. As the ground is stable, the cracks are assumed to have resulted from the deficiencies in construction.

4.4.2. Disaster mechanism and scale of disaster

In Block IV, the existing protections being stable, there is no risk of the slope collapsing as in the case of other blocks. In addition, the upper part above the existing protections is a gentle slope, so rockfalls are unlikely to take place.

However, different from other blocks, no protection is provided by the side of the permanent way. Therefore, should falling rocks etc. get over the existing drain-

age on the top of Block IV for any reason, they will directly hit the permanent way without fail.

4.4.3. Countermeasures

For the reasons mentioned in **Paragraph. 4.4.2**, no disaster is likely to occur in Block IV. In view of the site conditions, a temporary catwalk should be constructed across this block at the time of providing countermeasures for other blocks. In this case, it is necessary to set up a temporary fence to protect the permanent way. This protection fence must be designed as a structure that can be made use of even after the countermeasures for other blocks are provided. This structure should serve as a protection line by the side of the permanent way in Block IV.

5. Selection of Countermeasures

5.1. Countermeasures for each block

① Block I (between No. 0 and No. 1+9 m sections)

Set up a rockfall prevention fence on the top of the retaining wall.

② Lower part of the slope in Block II (between No. 1+9 m and No. 12+16 m sections)

Build a leaning type retaining wall to prevent rockfalls and collapse of base rock in the surface layer.

③ Upper part of the slope in Block II (between No. 1+9 m and No. 12+16 m sections).

Install grid frames with ground anchors from the retaining wall in the middle part of the slope to the elevation of about 80-85 m to prevent a somewhat deep weathered layer of base rock from collapsing. The length of anchorage is to be determined by a draw test.

④ Block III (between No. 13+15 m and No. 17 section)

Provide a leaning type retaining wall on the entire slope below the erosion front. As there is spring water between the No. 14+15 m and No. 15+15 m sections, install grid frames to prevent the weathered layer of base rock from collapsing. Investigate installing a culvert or the like in this area as a measure against spring water in the concave part.

⑤ Block IV (between No. 17+12 m and No. 19+2 m sections).

Investigate repairing or reinforcing the leaning type

retaining wall according to the progress of cracking on the top of the retaining wall.

In light of the foregoing, high priority should be given to the protection against disaster of the upper part of the slope in Block II. Therefore, it is necessary to undertake countermeasures for this block first of all.

5.2. Countermeasures for the upper part of the slope in Block II

A description is made below of a method of slope protection designed to make $F_{se} \geq 1.2$ at the time of an earthquake.

Reinforced concrete girders are arranged in the form of a grid, and anchors are fastened to the girders. After that, prestress P is introduced in the slope to increase the direct stress on the sliding surface in order to increase the friction resistance as a binding force and thereby to enhance the stability of the slope.

5.2.1. Stability analysis

The results of the stability analysis conducted in Paragraph 4.2.3.2 are as follows:

The section is divided for slip circle analysis as

shown in Fig.13.

Table10 The analysis by the slip circle method

$$\sum L = 185 \text{ m.}$$

Safety factor not under earthquake.

$$F_s = \frac{\sum (W \cos \theta \tan \Phi + cL)}{\sum W \sin \theta} \quad (4)$$

$$= \frac{3,317.91 \times \tan 30^\circ + 9.81 \times 185.0}{3,728.45}$$

$$= \frac{3,730.44}{3,728.45}$$

$$= 1.001.$$

Safety factor under earthquake.

F_{se} is given with the next formula . The next formula is determined on the basis of the reference⁽⁵⁾.

$$F_{sc} = \frac{\sum [(W \cos \theta - KhW \sin \theta) \tan \theta + cL]}{\sum (W \sin \theta + KhW \cos \theta)}$$

$$= \frac{(3,317.91 - 0.2 \times 3,728.45) \times \tan 30^\circ + 9.81 \times 185.0}{3,728.45 + 0.2 \times 3,317.91}$$

$$= \frac{3,299.92}{4,392.03}$$

$$= 0.751$$

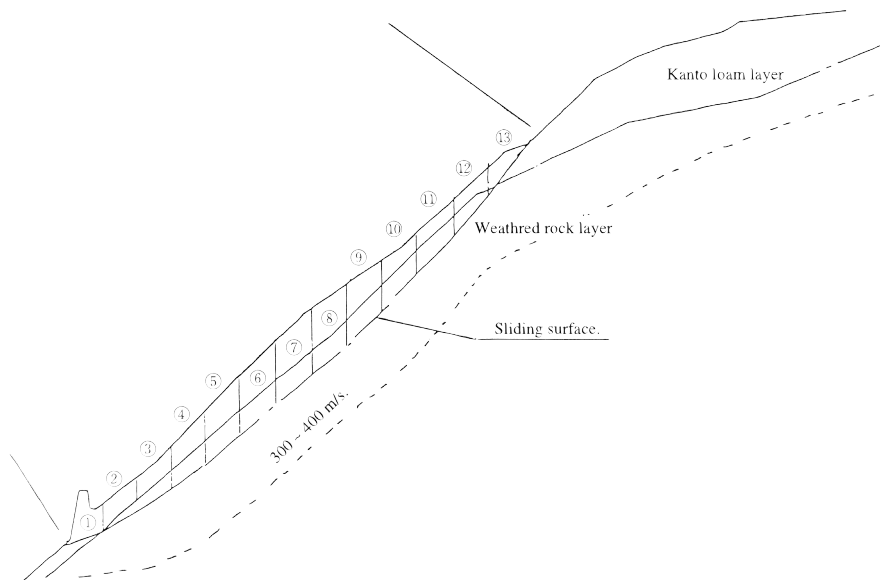


Fig.13 Section division figure

5.2.2. Prestressing of prestress force P

Safety factor F_{sep} at the time of the earthquake of case that does prestress force P prestressing is given with the next formula. The authors have determined the next formula on the basis of the references⁽⁵⁾ and ⁽⁶⁾.

$$F_{sep} = \frac{\sum[(W \cos \theta - khW \sin \theta) \tan \Phi + cL] + P(\cos \beta + \sin \beta \tan \Phi)}{\sum(W \sin \theta + khW \cos \theta)}$$

Prestress force P is calculated by the next formula from these value.

$$P = \frac{F_{se} \sum(W \sin \theta + khW \cos \theta) - \sum[(W \cos \theta - khW \sin \theta) \tan \Phi + cL]}{(\cos \beta + \sin \beta \tan \Phi)}$$

The average grade of the slide face. 45° (7)
 The angle of the anchor. $\alpha = 23^\circ$
 $\beta = 68^\circ (= 45^\circ + \alpha)$
 $\Phi = 30^\circ$

$$\begin{aligned} \text{Numerator} &= 1.2 \times (3,728.45 + 0.2 \times 3,317.91) \\ &\quad - (3,317.91 - 0.2 \times 3,728.45) \\ &\quad \times \tan 30^\circ - 9.8 \times 185.0 \\ &= 1,970.52 \\ &= 1,971.0 \end{aligned}$$

$$\begin{aligned} \text{The denominator} &= \cos 68^\circ + \sin 68^\circ \times \tan 30^\circ \\ &= 0.9099 \end{aligned}$$

$$\begin{aligned} P &= \frac{1,971}{0.9099} \\ &= 2,165.62 \text{ t} \\ &= 2,166 \text{ t} \end{aligned}$$

Total tension $\sum P$ is as follows of to every 1 grid frame, when it makes the interval of the grid frame the pitch 4.5 m.

$$\sum P = 2,166 \text{ t} \times 4.5 = 9,747 \text{ t}$$

Table 10 The analysis by the slip circle method

$\gamma d = 1.8 \text{ t/m}^3$
 $\phi = 30^\circ$

Number	W	W1	W2	W cos θ	W sin θ	θ
1	88.29	88.29		78.67	40.08	27.0
2	176.58	123.61	52.97	149.75	93.57	32.0
3	317.84	211.90	105.95	260.36	182.31	35.0
4	406.13	264.87	141.26	324.35	244.42	37.0
5	423.79	282.53	141.26	314.94	283.57	42.0
6	494.42	282.53	211.90	349.61	349.61	45.0
7	476.77	264.87	211.90	325.15	348.68	47.0
8	512.08	264.87	247.21	329.16	392.28	50.0
9	512.08	264.87	247.21	315.27	403.53	52.0
10	512.08	264.87	247.21	293.72	419.47	55.0
11	459.11	247.21	211.90	250.05	385.04	57.0
12	388.48	211.90	176.58	194.24	336.43	60.0
13	282.53	105.95	176.58	132.64	249.46	62.0
Total	5,050.19			3,317.91	3,728.45	

Anchor power P_0 is as follows of to every 1 step when it arranges the P C steel bar to 42 steps ((185 / 4.5) +1)

$$\begin{aligned} \Sigma P/N &= 9,747 \text{ t} / 42 = 232.07 \text{ t} \\ &= 232.1 \text{ t} \end{aligned}$$

5.2.3. Design of the reinforced concrete girder

The grid girders are analyzed as a continuous beam of 9 equal spans that is shown in **Fig.14**.

The absolute value of the bending moment of the B point;

$$M_b = 0.1057 \times q (l)^2 \cong q (l)^2 / 9$$

Tension (P) of the ground anchor, balances to uniform load (q) like the **Fig.15**;

$$\begin{aligned} P &= 232.1 \text{KN}, L = 4.5 \text{ m are substituted;} \\ Q &= 51.58 \text{KN/m} \end{aligned}$$

$$\begin{aligned} M_b &= q (l)^2 / 9 = 51.58 \times (4.5)^2 / 9 \\ &= 116.06 \text{KNm} \end{aligned}$$

The section of Reinforced Concrete Girder is shown in **Fig.16**.

$$\begin{aligned} AS &= (M / \delta sa) \times (7/8) \times d \\ &= (116.06 \times 10^5 / 1.766 \times 10^5) \times (7/8) \times 4 \times 10 \\ &= 18.78 \text{cm}^2 < 20.26 \text{cm}^2 = 5.065 \times 4 \quad (\text{D-25} \times 4) \end{aligned}$$

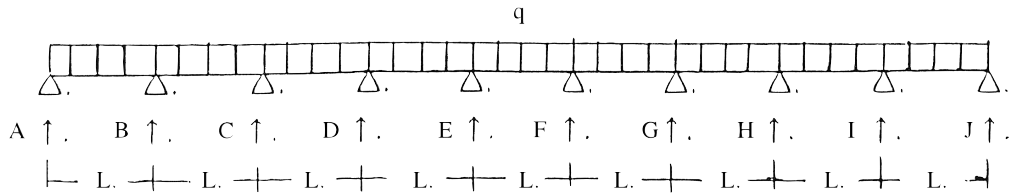


Fig.14 Continuous beam of 9 Equal Spans

$$Q = 2P / 2L$$

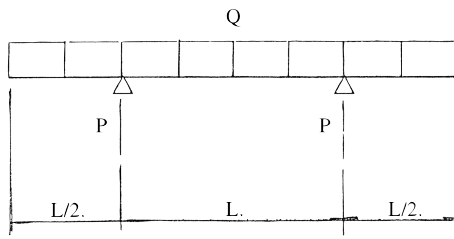


Fig.15 Overhanging beam

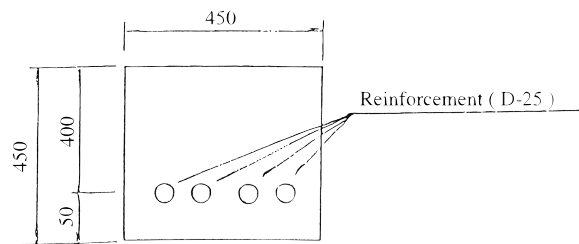


Fig.16 Section of Reinforced Concrete Girder

6. Conclusion

The authors have carried out this case in 1982. In this place, in the case that the authors adopt soil reinforcing that is paid attention recently, main problem are as follows.

- ① The soil reinforcing is method⁽⁸⁾ that intends the unification with the ground by the circumference face frictional resistance between the reinforcement material and ground.
- ② It is expected that cracking etc. exists in the ground of the surroundings of the reinforcement material and the weathering of the ground is going on. This is showing that is the condition which the ground is loose and we can not expect sufficient frictional force of the reinforcement material circumference face.
- ③ Pulling the material that reinforces sufficient from the above reason the power is not able to expect. Therefore the direct stress of the sliding surface does not increase and the friction resistance as the binding force is not able to expect. Accordingly the improvement of the aseismicity is not able to expect.

From over, the authors judge that the soil reinforcing is insufficient as an earthquake countermeasure in this ground. On the other hand, the improvement of the aseismicity is able to expect in the case of the ground anchor method. The main reason is as follows. When we introduce the prestress force the direct stress of the sliding surface increases. Therefore the friction resistance as the binding force increases. It assumed that

the aseismicity improves from this case.

The effect of the slope protection can be enlarged, if we adopt the ground anchor method that introduced the prestress as the earthquake countermeasure from the condition of the ground, even when it is present accordingly.

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東海地震のためのグラウンドアンカーによる斜面防護の研究

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概 要

従来, 鉄道の斜面では特別な補強が行なわれてこなかった。そのため, 鉄道では地震により各種の被害を受けてきた。近年, 盛土斜面が新設される場合, 斜面補強工法として補強土工法が採用されるようになってきている。又, この補強土工法の耐震性についての報告がなされている。

一方, 切土斜面では, 斜面補強工法としての補強土工法の事例が殆ど報告されていない。この主な理由は次のとおりである。我々は盛土斜面では周面摩擦力を容易に管理できる。一方, 切土斜面では周面摩擦力を管理することが困難である。

筆者達は大規模地震対策特別措置法による東海地震に係る地震防災対策強化区域内の切土斜面について補強土工法を採用せずにグラウンドアンカー工法を採用した。グラウンドアンカー工法は周面摩擦力に依存しない工法である。又, 補強土工法の内容は施工時に確定してなかったことがその理由である。

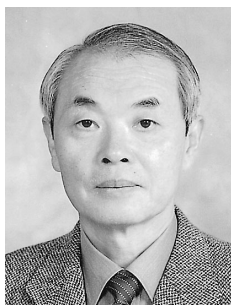
グラウンドアンカー工法を地震対策工法として切土斜面に施工すれば, 斜面防護の効果を大きくすることができることを筆者達は解明した。

Biographical Sketches of the Authors



Takakimi Ohki was born in Miyazaki, Japan on October 22, 1946. He received Bachelor of Civil Engineering from Nihon University in 1969. He received the Professional Engineer in 1991. He joined the Japanese National Railway in 1969, Japan Freight Railway Company in 1987 and JR Freight Real Estate Development Company Limited in 2003. He has engaged to the maintenance control, especially disaster prevention, of the railroad structure.

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He was appointed to an associate professorship at the Department of Production Engineering, Nihon University, in 1975, serving in that capacity up to this date. He has been conducting studies mainly on the technological properties of Kanto loam, mechanism of compaction by rolling volcanic cohesive soil, mechanism of liquefaction of sandy ground by earthquake, and so forth.

He served on the Science and Technology Agency's Examination Committee for Professional Engineers as a member in 1988, secretary in 1989 and section chief in 1990.

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