Earth Reinforcement Practice, Ochiai, Hayashi & Otani (eds) © 1992 Balkema, Rotterdam. ISBN 90 5410 093 1

Application of soil nailing method to protection measures against failures of massive cut-slopes

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ABSTRACT : This paper presents a case study of steep-slope failure that occurred as a result of heavy rain during cutting of slope for a road rehabilitation project. From comparative studies the soil nailing method is found efficient and suitable as a slope protection measure. The case study includes design considerations on protection measures and construction supervision by instrumented measurements of reinforced slope's behaviors. The drainage boring is also conducted as an auxiliary work. It is confirmed that the instrumented slope-behavior measurement and feed-back of the data to construction is quite essential for the success of the soil nailing method.

1 INTRODUCTION

During the performance of cutting works on the steep slope of 40°～45°, a part of the road rehabilitation project, north of Hiroshima, Japan, the massive failure of slope (i.e. approximate dimension : length 80m, width 70m and maximum depth 20m) occurred as a result of the season's continuous heavy rain. The principal causes of slope failure (hereinafter slope collapse) are found to be the existence of a big fault along the slope and the weakness of the slope itself. Upon conducting comparative studies on slope protection measures the decision is reached to adopt the soil nailing method that is reinforcing of the slope with steel bars. In this method the counter-measure against the collapse is the removal of the unstable soil. And the natural earth slope adjacent to the massive cut area is protected by the soil nailing method. The scale of protection is 7200m^2 in area and 3～7m in depth. The drainage boring is also conducted as an auxiliary work to this method.

During the supervision of protection works the deformation of reinforced slope takes place unexpectedly once more and the slope stability is restored by employing additional reinforcements. The project is completed after 4 years' rigorous works.

This paper presents case study of the project which includes design consideration on protection measures and construction supervision by instrumented measurements of reinforced-slope's behaviors.

2 OUTLINE OF SLOPE-COLLAPSE AREA

The outline of slope-collapse area is as follows.

Terrain : The collapse takes place at the slope forming the right side of the river Gonogawa that flows winding through the deep ridge of rolling mountains, average altitude 400m ～500m. The collapse area forms a steep slope of 40°～45° and it is 200m ～ 300m high with respect to the river bed.

Geology : The area is composed of tuff and rhyolite lava that correspond to the rhyolite series of Cretaceous period, Mesozoic era. These rocks are intruded by andesite dykes from place to place. The river and terrace alluvial deposits that are estimated to be of pliocene to pleistocene form the overburden.

Fault : The fault zone that is directly related with the collapse is seen as outcrops of lineament running in the north-east to south-west direction and its existence can be traced for about 20 km.

At the collapse site the fault zone covers an area of width 100m and the middle portion of approximate width 50m consists of recessive layers of sandy soil. The surrounding area is made up of soft rock to medium hard rock. The fractured condition of upper layer of fault zone is quite significant. The results from soil test of the fault-zone clay are as shown in Table 1.

Table 1 Result from soil test of fault-zone clay

Test Item			S − 1	S − 2
Specific gravity			2.711	2.758
Water content ratio (%)			24.08	20.98
Grain size distribution (%)		Gravel	8	30
		Sand	27	28
		Silt	29	24
		Clay	36	18
Consistency		Liquid limit (%)	44.98	43.60
		Plasticity limit (%)	17.14	21.19
		Plasticity index	27.84	22.41
Classification		Name of soil	Clay	Clayey sand
		J. U. S. C. ※	(CL)	(S C)
Shear box test (CU)	Undisturbed	C (kgf/cm²)	0.23	0.67
		ϕ (°)	28.4	32.8
	Disturbed	C (kgf/cm²)	0.16	0.48
		ϕ (°)	26.6	32.0

※ Japanese Unified Soil Classification system

Collapse : During the temporary road construction required for the excavation of a slope from above, a part of the road rehabilitation project, the slope collapse takes place at the middle of the fault zone with the dimension; height 80m, approximate width 70m and maximum depth 20m. Refer to Fig.1. The collapse of slope proceeds apace and the falling debris spreads up to the bed of the river Gonogawa destroying the protection fence of the temporary works. The slope collapse occurrs in the rainy season. The rainfall recorded for the month before the collapse is 342mm which is comparatively high for the area in question.

The highly fractured rock of the fault zone and the steep slope of low strength that results from the prolonged erosion of soil by the rapid flow of the river Gonogawa are considered to be the causes of slope collapse. The secondary causes of the collapse are the rise of ground water table due to continuous heavy rain and the excavation of lower portion of the slope.

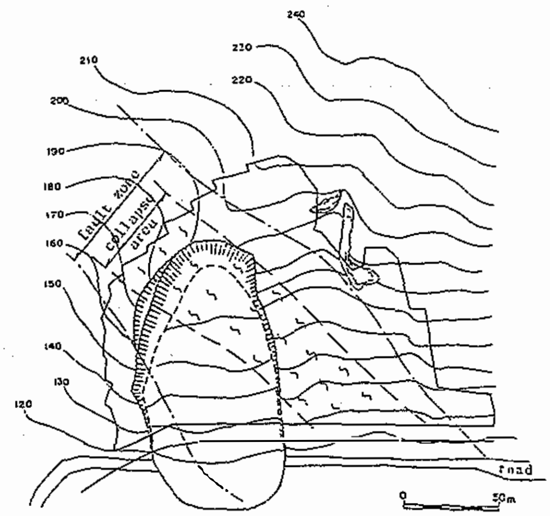

Fig.1 Plan showing slope-collapse area

3 DESIGN FOR COUNTER-MEASURES

As a result of comparative studies the practice of soil removal from the collapse area and the steel reinforcing of steep slope are adopted as counter-measures against the collapse. Furthermore the drainage boring is also conducted as an auxiliary work to this method.

The scale of slope cutting is as follows. The height of slope surface : 100m, width : approximately 150m, total number of berms : 14 berms (hereinafter steps), excavated earth volume : about 110,000 m^3. The slope's gradient differs from place to place as it is to be hung up with the steep natural slope. And the geological condition of outcrops at the slope is also differnet according to the extent of fracturing of the fault. As a result of slope stability analysis by slice method it is clarified that the slope of 1:1.2 (about 39.8°) at the middle portion of the fractured zone has a stability factor of 1.20 and hence the zoning of blocks as shown in Fig.2 (their explanations in Table 2) is done and the counter-measures or slope protection works as stated in Table 3 are performed.

The design for the steel reinforcement of slope is carried out by using the ultimate equilibrium method which is based on static equations.

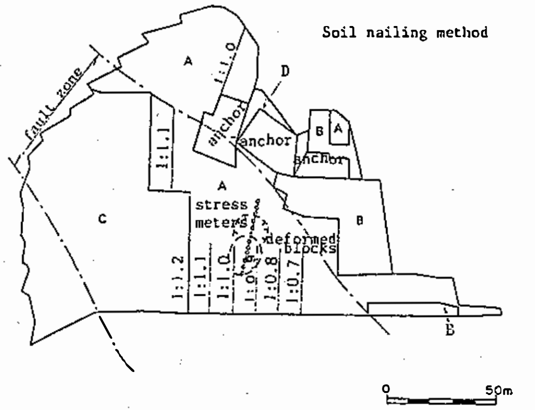

Fig.2 Zoning of slope protection blocks

Table 2 Zoning of slope protection works

Geological condition			Gradient of slope		
			1:0.7~1:1.0	1:1.0~1.2	Less than 1:1.2
			θ=55°~45°	θ=45°~40°	θ<40°
Talus deposit		Sandy soil	A	A	C
Gravel layer		Sandy soil	A	A	C
Rhyolite · Andesite	Inner portion of fault		A	A	C
	Around fault zone	Sandy soil	A	B	C
		Soft rock I	A	B	C
		Soft rock II	B	B	D
		Medium hard rock	D	D	D

Consequently the stability is considered as the equilibrium condition between the driving or sliding forces on the sliding plane and the resisting shear forces of the slope soil plus reinforcement on the other side. The reinforcement effect of reinforcing members is evaluated by the component of the reinforcement's tension in the sliding direction.

1) Checking of the pull-out resistance

The horizontal component of forces in time of collapse of upper layer of soil: H : it is the resistance by the bond of the natural slope (mountain) and the reinforcement.

- Horizontal component: H (tf)

$$H = (Q \cdot F_{sp} - W_r) \cdot \cos\alpha \quad (1)$$

Q: sliding force (tf)
F_{sp}: projected safety factor
W_r: resistance against sliding (tf)
α: angle between sliding line and horizontal (°)

- Required bond per reinforcement: P (tf)

$$P_o = H/n \quad (2)$$

$$P = P_o/\cos(\beta-\alpha) \quad (3)$$

P_o: horizontal component of bond per one reinforcing bar
n: number of reinforcements in one cross section (number)
β: angle between sliding plane and reinforcement (°)

- Required bonding length : L (m)

L is the maximum value out of the following L_1, L_2, L_3.

$$L_1 \geqq F_s \cdot P/\mathrm{Pull} \quad (4)$$

$$L_2 \geqq F_s \cdot P/(\pi \cdot D \cdot \tau_1) \quad (5)$$

$$L_3 \geqq P/(\pi \cdot d \cdot \tau_2) \quad (6)$$

where

L_1 : cohesion length as obtained from the in situ pull-out test (m),
L_2 : cohesion length that depends upon the bond between the grout and ground (m),
L_3 : cohesion length that depends upon the bond between steel and grout (m).
F_s: safety factor against pull-out forces of reinforcement
Pull : friction resistance of the surface around reinforcement as obtained from pull-out test (tf/m)
D: drill-hole diameter (m)
d: diameter of reinforcement (m)
τ_1 : friction resistance of the surface around reinforcement (tf/m^2)
τ_2 : allowable bond between reinforcement and grout (tf/m^2)

2) Checking of shear resistance

$$\tau \cdot A \geqq (Q \cdot F_{sp} - W_r)/n \quad (7)$$

τ : allowable shear strength of reinforcement (tf/m^2)
A : cross-sectional area of the reinforcement (m^2)

As a result of the above design computation the steel reinforced earth of A block has parameters of the reinforcements determined as shown in Table 3; they are the steel diameter 19mm ~29mm, bore-hole diameter 86mm and length 3~7mm respectively.

Table 3 Counter-measure for each block

Block	Counter measure	Specification
A	Concrete spray crib	Crib section 200×200mm
	Steel reinforcement	Crib span 2.0×1.5m
	Rock greenery	Steel laying D19~29 3-7m
B	Concrete spray crib	Crib section 200×200mm
	Rock greenery	Crib span 2.0×1.5m
C	Simple concrete spray crib	Crib width 200mm
	Rock greenery	Crib span 2.0×1.5m
D	Mortar spray	Thickness 5cm

4 SLOPE DEFORMATION DURING CONSTRUCTION

The measurement for slope's behavior observations as stated in Table 4 are conducted with a view to confirm the feasibility of the construction method and the safety management during construction. As a result of this observation the deformation is confirmed at the middle portion of the fault zone where the block (3rd~5th step from below, refer to Fig.3) is cut by the slope 1:0.9. Furthermore this block is the only place where the observation of steel stress is originally planned.

4.1 The condition of slope-deformation

When the reinforcement is completed up to the 7th step with the progress of the slope cutting on the 5th step, the slope on a part of the 5th to 6th step and the slope below it (not yet cut) are found fractured. Refer to Fig.3. The fracture is quite intense at around the toe of the 5th step's slope and the tensional fracture has a maximum width of 10cm whereas the 6th step's slope has only a few number of mm-thick fractures. As a result of measuring the deformation with the inplacement of stress meters in the fracture the deformation is found to take place at the rate of 1mm per day for five consecutive days after the occurance of fracture. This coincides with the time

Table 4 Measurement for slope behavior

Measuring instruments	Quantity
Inclinometer in bore-hole	10 No
Bore-hole for water table observation	5 holes
Movable tack	38 tacks
Stress meter for steel reinforcement	52 meters
Slope checking by visual observation	once/week

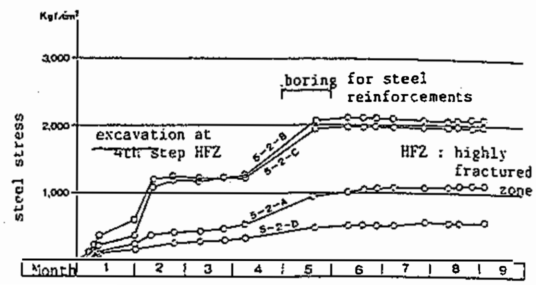

Fig.4 Changes in steel stresses with time

when the water level of the inclinometer laid in the slope of the 7th step rises to 1m~3m. This is considered to be due to the daily rainfall of 120mm recorded two days earlier and also the supply of water from the anchor bore-holes drilled for the counter-measure against the rock sliding along the andesite outcrops of the upper slope. The high water table is not observed after the completion of the emergency drainage boring.

As a result of the boring observations conducted within the area of deformation

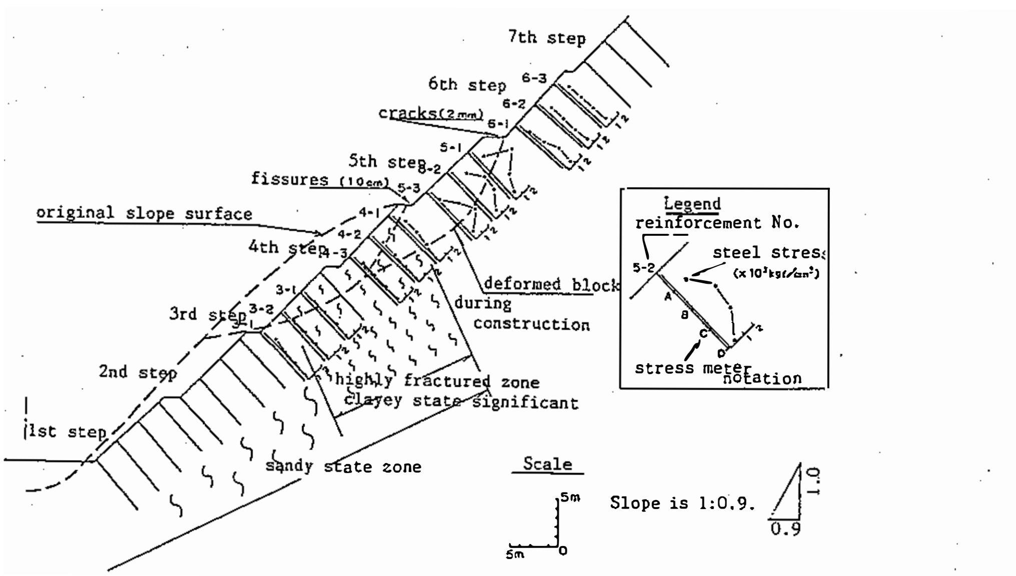

Fig.3 Cross-section showing steel-stress measuring points

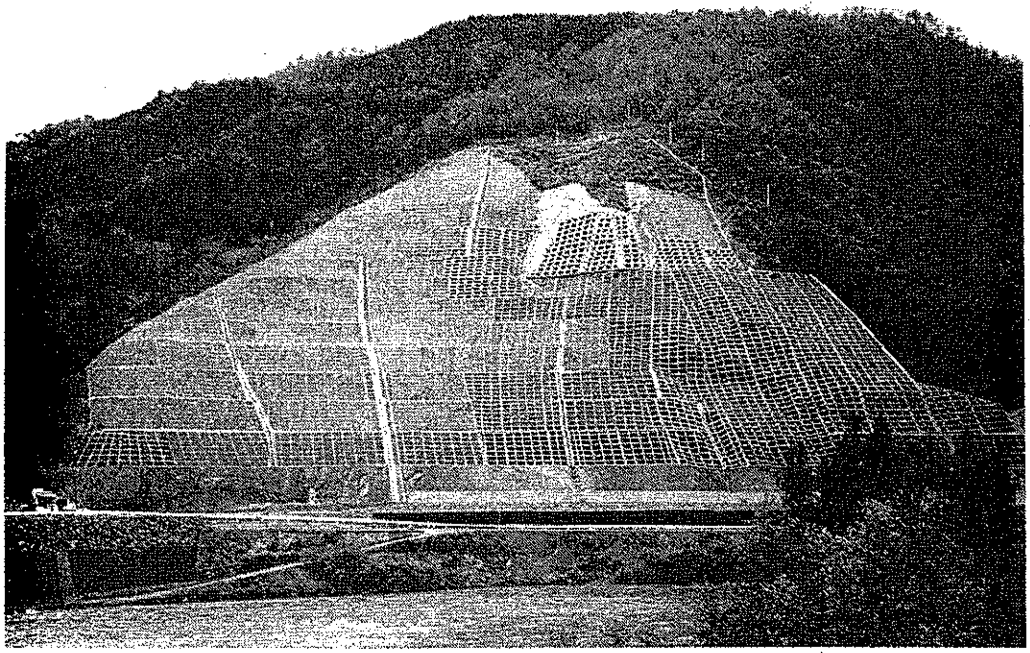

Fig.5 General view of the completed construction site

it is learnt that the rock has transformed into sandy soil mixed with clay layers due to excessive fractures. The observations by the additional inclinometers laid in the boreholes reveal that the deformation mostly occurs at the soil layer with N value 10 at around the depth of 7m below the surface.

4.2 Results from measurement of steel stresses

After the slope's deformation stops the steel bars are inserted into the slope of 5th to 6th step and the construction works on the lower slope are carried out.

According to the results from measurement of steel stress by stress meters attached to the steel bars (refer to Fig.3) an abnormal increase in the steel stresses of the 5th step is confirmed. The increase in the stresses occurs in stages as shown in the graph of changes in steel stresses with time (Fig.4). The time of stress increase just coinsides with the time of insertion of the steel bars after the excavation of highly fractured slope of 4th step. And the increase in stresses of the stress meters laid in the slope of 3rd and 4th step is small and it is less than 500 kgf/cm^2 at each point. Thus the abnomal increase in steel stresses of 5th step's slope is due to the fact that the steel reinforcement at the 5th step's slope is in a state of carrying a concentrated load as the original sliding soil mass moves its position under the pressure of slope cutting and bore-hole drilling. It is presumed that the measurement is not so much reliably trustworthy as the recorded steel stress at 5th step's slope exceeds 3000 kgf/cm^2. Hence the intensity of steel reinforcement is increased to two times the original designed reinforcement by installing additional reinforcing bars. The additional reinforcement has an increase in stress less than 200 kgf/cm^2 and this means that the stress increase is negligible.

5 CONCLUSION

This report presents an introduction to the case history of slope collapse originating from the fault zone during road rehabilitation works and the results of observation of the ground's dynamic behavior during construction. No deformation is recorded at the slope 1:1.20 and it is also considered stable by the stability analysis of slice method. But the deformation is noted at the slope 1:0.9. This indicates that the stability analysis can reliably be performed by the slice method. In the present case the concentrated loads are applied to the reinforcing steel and the management by instrumented measurement is required in case the soil nailing method is adopted.

As a result of pursuit of high land-utility factor the land development projects are likely to increase and it is presumed that similar cases may arise in future. Hence establishing an efficient and precise method of large-scale geological investigation is required to determine rock quality, geological structure, dis-continuity of rock, and so on. And it is considered essential to establish an automatic measuring system for slope's dynamic behaviors and speedy processing of the precise instrument-data with a view to enhance the development of instrumented measurement. And again it will be necessary to establish an analytical method to solving simultaneously the evaluation of feasible slope protection method, slope's safe excavation height and slope's safety factor simulating the behaviors of ground at every stage of construction.

The completed construction site is as shown in Fig.5.

ACKNOWLEDGEMENT

The authors are deeply grateful to the engineers of Miyoshi Civil Works Office and Fukken Co., Ltd. (Consulting Engineers) for their help and support in executing the design and construction works. And the authors would like to express their sincere thanks particularly to Dr. N. Fukuda for his valuable advices and comments on the report, and to Mr. Aung Swe for his active collaboration in translating the report into English.

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