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Long term stability of soft sedimentary rock slopes

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ABSTRACT: The mechanism of long term instability of soft sedimentary rock of mudstone and tuff was discussed in this paper for cases of a cut slope excavation in a road construction and of high embankment construction in a land development. Severe sliding and settlement were observed in both sites in a few years after construction, so that case studies were conducted through laboratory element tests and some stability analysis. It was inferred that such damages are caused by reduction in strength due to stress release and expansion by cutting and to slaking and weathering of fill materials. Appropriate countermeasures were then proposed to apply confining pressure for cut slope and to control grain size distribution of fill materials in compaction.

1. INTRODUCTION

Mudstone and tuff dealt with in this paper belong to soft sedimentary rocks, which have the unconfined compressive strength less than about 10MPa as an intact rock. It is supposed that this kind of mudstone and tuff have been hardened under the action of various types of external forces applied through the process of sedimentation of clay minerals and stratification in the water, and that of successive elevation by crust movement and drying in the air. The period of sedimentation of these rocks is considered to be after Tertiary in the geological time scale, and their covering area is vast and worldwide. Engineers thus have encountered frequently a lot of problems in construction works associated with cutting and filling of soft sedimentary rocks; e.g., reduction in strength due to expansion and swelling in cut slope accompanying sliding failure, and slaking and weathering of mudstone material for embankment construction leading to a large settlement and lateral deformation and resultant slope failure.

This paper concerns the mechanism of long term instability of tuff slope due to excavation in a road construction and that of large settlement of high embankments constructed of mudstone in a land development. Because severe sliding and settlement were observed in both sites a few years later the completion of construction, case studies were conducted through laboratory element tests and some stability analysis. It was inferred under this investigation that such damages are caused by reduction in strength due to stress release and expansion by cutting in the former case and to slaking and weathering of fill materials in the latter. Appropriate countermeasures were then proposed to apply a confining pressure for cut slope to prevent sliding and to control grain size distribution of fill materials in compaction.

2. FAILURE OF TUFF CUT SLOPE

2.1 *Slope Failure and Preliminary Investigation*

In a road construction, a cut slope was made by excavating tuff layer as much as about 3m at maximum, as shown in Figure 1. The value of the unconfined compressive strength (q_u) of intact rock of this tuff was around 2MPa, so that the cut slope might have a very high safety against sliding just after excavation. The first time slide took place at about two years later the excavation, which successively developed upwards as the 2nd,

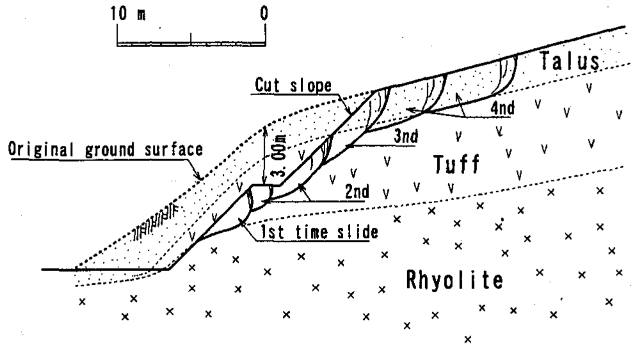

Figure 1. Failure of Tuff Cut Slope

3rd and 4th slide and finally resulted in a very large massive slide as illustrated in the figure.

Constant volume direct shear tests were carried out on undisturbed fairly softened samples taken from slip zone of the tuff layer, and shear strength parameters obtained under consolidated and drained condition resulted in $c' \fallingdotseq 0$ and $\phi' \fallingdotseq 43°$. This signifies that intact rock of the tuff with $q_u \fallingdotseq$ 2MPa just after excavation showed a strength reduction to the state of normally consolidation. In order to understand the mechanism of this long term strength reduction, some consolidation and shear strength tests were conducted on undisturbed samples as stated below.

Incidentally, X-ray analysis was additionally done to investigate mineral composition, which is considered to be one of influential factors for such strength reduction. The result showed that the tuff is composed principally of quartz and a little clay minerals of feldspar, kaolinite and mica, and contains little expansive montmorillonite minerals.

2.2 *Compression and Expansion Characteristics*

According to oedometer tests on some undisturbed samples, the value of the consolidation yield stress (p_y) ranges in 200 ~ 500kPa, mostly $p_y \fallingdotseq$ 500kPa, which might be considered to be the amount of stress release due to excavation.

Additional consolidation tests were carried out to measure the amount of expansion and the confining pressure to prevent expansion, through the process of ① compression by loading to the value of p_y, ② unloading to 5kPa and drying as it is, ③ supplying water to saturate the sample. The value of p_y was then obtained by reloading after the process of expansion test, which in the 1st time resulted in $p_y \fallingdotseq$ 250kPa, half of the initial value of p_y=500kPa. By repeating this process of loading, unloading (drying) and saturation, the value of p_y gradually decreases and, after 4 ~ 5 times repetition of the cycle, most of pre-compression effect (p_y) of the tuff disappears and rock particle is softened to be the state of normally consolidation.

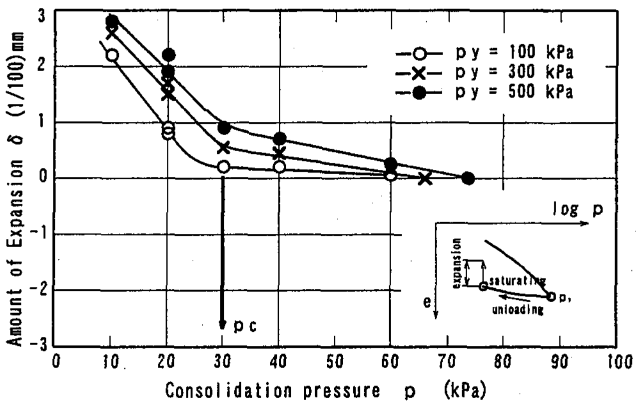

Figure 2. Expansion Characteristics of Tuff

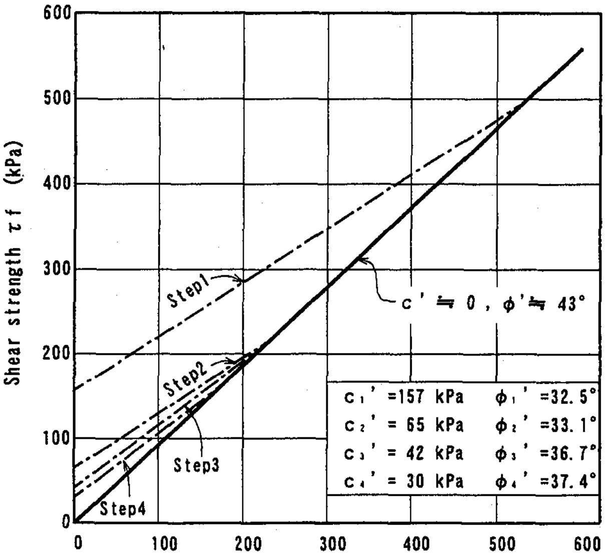

Figure 3. Shear Strength Characteristics of Tuff

Figure 2 shows relation curves of the amount of expansion (δ) and the consolidation pressure (p) for three representative samples having different value of p_y. It is seen that the value of δ becomes large as the value of p_y decreases and it increases abruptly under the confining pressure below around $p(p_c)$=30kPa.

It is thus recognized that the strength of tuff cut slope gradually decreases by the cyclic action of drying and saturation under the state of stress release due to excavation, and considerable strength reduction is anticipated under the confining pressure p<30kPa.

2.3 *Shear Strength Characteristics*

Figure 3 shows typical results of constant volume direct shear tests (CD-condition) on unloaded and saturated samples taken from the last process ③ of the expansion test mentioned above: e.g., the sample of Step.1 is first loaded by $p_1=p_y$=500kPa and unloaded to p_2=5kPa and saturated for swelling, and in Step.2, Step.3 and Step.4, loaded up to p_1=250kPa, 220kPa and 180kPa, respectively, and unloaded to p_2=5kPa and saturated.

The relationship between the values of shear strength parameters (c', ϕ') and that of p_y is plotted in Figure 4. It is seen that the value of ϕ' shows a constant increase as p_y increases in contrast to a gradual decrease in the value of c', approaching to the state of normally consolidation of $c' \fallingdotseq$ 0 and $\phi' \fallingdotseq 43°$. This signifies that the strength reduction of tuff cut slope due to excavation takes place by the cyclic action of drying and saturation from the surface part with low confining pressure, and it gradually develops deeply to extend progressive large sliding failure as observed in the field.

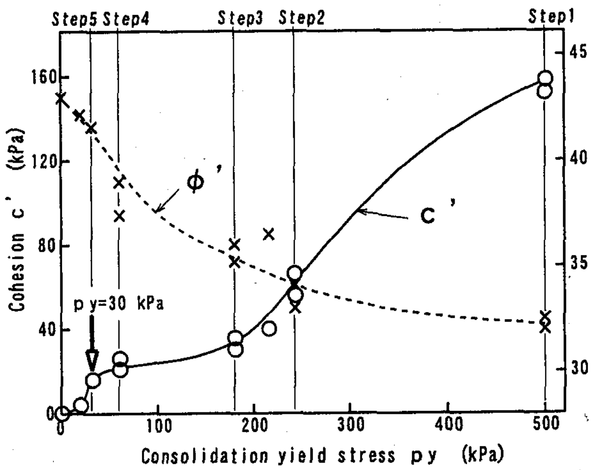

Figure 4. Shear Strength Parameters

2.4 *Countermeasures*

The confining pressure p_c=30kPa defined in Figure 2 becomes to be the most significant parameter to discuss stability of the tuff cut slope, because considerable strength reduction is anticipated in the low stress range $p<p_c$=30kPa where cohesion component diminishes to $c' \fallingdotseq$ 0 as shown in Figure 4. Basic concept of countermeasure to prevent sliding is therefore simple to be an application of some confining pressure of p_c>30kPa on the sloping surface. Though earth anchor method is appropriate and popular in this situation, earth retaining wall and counterweight fill of talus of about 1.5m high were adopted in the field.

3. SETTLEMENT OF MUDSTONE FILL

3.1 *Outline of Delayed Settlement*

About 5m high embankment was constructed in a project of land development by using mudstone of q_u=2 ~ 10MPa as fill materials, as illustrated in Figure 5. Materials were spread in a depth of 30cm and compacted by a vibration roller to be the degree of compaction $D=\rho_d/\rho_{dmax}$ greater than 90%. According to the data of field observation, most of compression of the fill had occurred until the end of construction and settlement was little after that. About 4.5 years later the completion of the fill, however, puddles arose intermittently during about two months on the surface of the land due to a long term rainfall. Settlement was then observed on the surface of the land by about 20cm at maximum. The mechanism of settlement which appeared following a long term delay is discussed below.

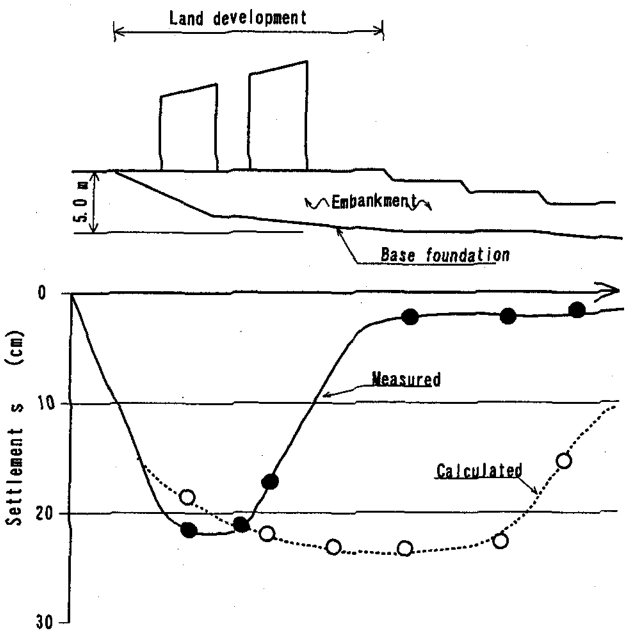

Figure 5. Delayed Settlement

3.2 *Gradation and Compaction of Fill Materials*

Gradation curves of mudstone materials just after ripper excavation and those after bulldozer spreading are compared in Figure 6. These curves in general can be approximated by the following Talbot equation;

$$P=\left(\frac{d}{D}\right)^n\times 100(\%) \qquad (1)$$

and the results showed that the maximum grain size of both materials is $D \fallingdotseq$ 250mm and the power number *(n)* of the former ranges n=0.5 ~ 0.7 and that of the latter n=0.4 ~ 0.6. Though rock particles were later crushed by compacting with a vibration roller, the change in n-value is considered a little because crushing is usually done only in the surface part of spread layer. The value of n after compaction is then assumed as $n \fallingdotseq$ 0.45.

It has been pointed out that slaking and the strength reduction due to weathering of compacted mudstone materials are affected by their grain size distribution (n-value), the amount of air content (v_a) and the applied confining pressure (p) (Ohne 1984). Laboratory tests were conducted here to investigate strength reduction due to weathering, in which cylindrical specimen of mudstone material first undergoes five cycles of wetting and drying weathering processes under a specified constant vertical confining pressure (p_v) in a similar way as adopted in the stabilization test for concrete aggregate. The specimen is then loaded vertically in a triaxial cell with a lateral confining pressure of 100 ~ 300kPa to obtain its shearing strength after weathering (Nakamura et al. 1998).

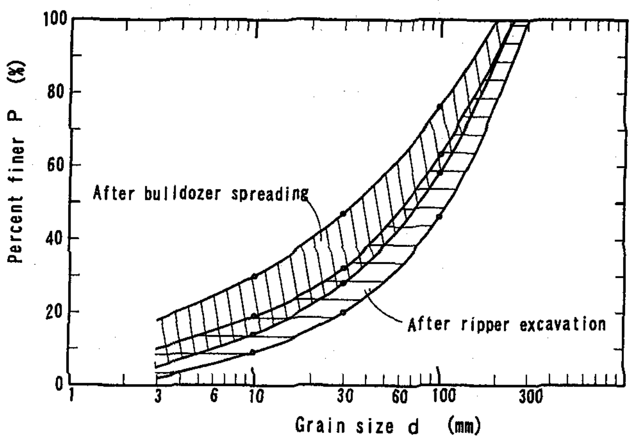

Figure 6. Gradation of Mudstone Fill Materials

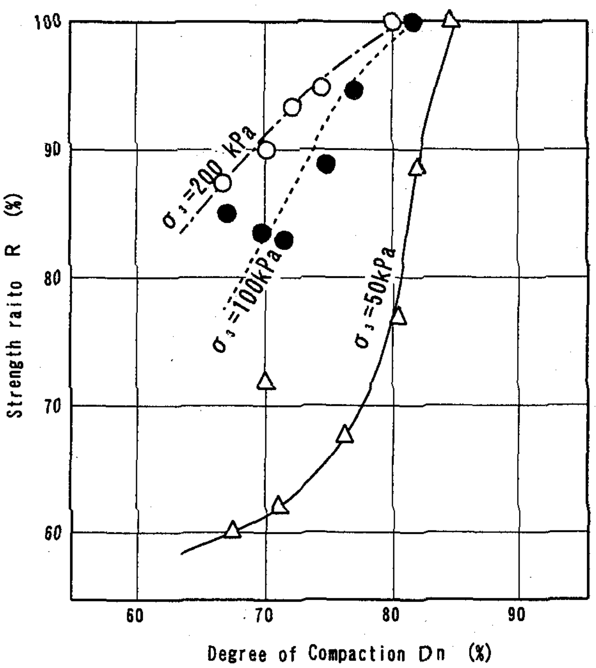

Figure 7. Strength Reduction due to Weathering

The results are presented in Figure 7 as a relationship between the strength ratio R (%) of the deviator stresses after to before weathering process and the degree of compaction (D_n) defined below by the ratio of a compacted dry density (ρ_d) to the dry density of the intact rock particle (ρ_f).

$$D_n=\frac{\rho_d(\text{compacted})}{\rho_f(\text{intact rock})}\times 100(\%) \qquad (2)$$

The reason why D_n-value was adopted here instead of usual D-value is due to the fact that the value of ρ_{dmax} to be a standard of the degree of compaction itself changes much in soft rock materials, together with their grain size distribution, in various processes of embankment construction.

It is noted in Figure 7 that strength reduction is avoidable in the range $D_n \geqq$ 80% in cases of σ_3=100, 200kPa and $D_n \geqq$ 84% in case of σ_3=50kPa. Considering that D_n-value of 80% and 84% corresponds to D-value of 90% and 94.5%, respectively, strength reduction of about 25% is anticipated for the fill because compaction was controlled to satisfy $D \geqq$ 90% and the average confining pressure in the fill is estimated to be around σ_3=50kPa.

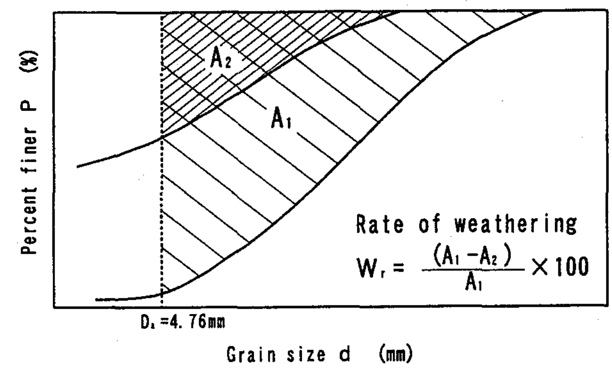

Figure 8. Rate of Weathering (W_r)

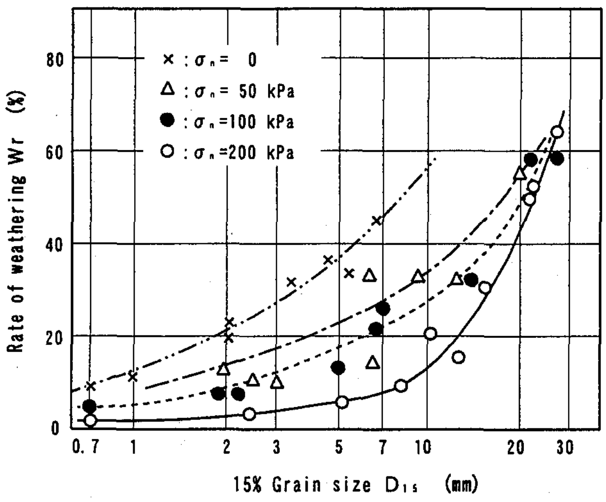

Figure 9. Grain Size (D_{15}) ~ W_r Relationship

3.3 *Relation of Rate of Weathering and Settlement*

It has been noted that the size of pore composed by rock materials and the amount of finer materials filling the pore are to be influential factors on the strength reduction due to slaking and weathering. The rate of weathering *(W_r)* is now defined for materials of grain size under 4.76mm as presented in Figure 8, and the relationship between the 15% grain size *(D_{15})* and W_r are plotted in Figure 9 by taking the confining pressure (σ_n) as a parameter. It is inferred from the figure that slaking and strength reduction is avoidable in the range $D_{15} \leqq$ 0.5mm for confining pressure $\sigma_n \geqq$ 50kPa. This limit value of D_{15}=0.5mm corresponds to the Talbot n-value ≒ 0.31 for the maximum grain size D ≒ 250mm.

In this project of land development, fill materials were controlled to have D ≒ 250mm and n ≒ 0.45 as stated before, which results in D_{15}=15mm for σ_n=50 ~ 100kPa and the rate of weathering becomes to be W_r ≒ 27%. Settlement of the fill can then be estimated by using the relationship between the values of W_r and compressive strain (ε_v) as presented in Figure 10. This gives ε_v ≒ 4.2% for W_r ≒ 27% and the value of settlement to be s=0.042 × 5m=21cm for the fill, which is equivalent to the settlement of about 20cm observed in the field (Figure 5). Delayed settlement can thus be interpreted well by using the concept of slaking and weathering of mudstone materials.

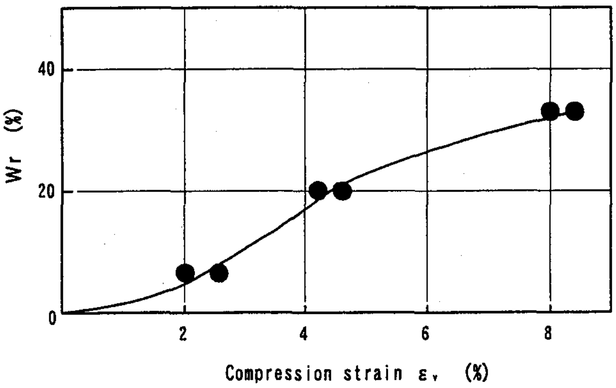

Figure 10. Settlement (ε_v) ~ W_r Relationship

4. CONCLUSIONS

The mechanism of long term instability of soft sedimentary rocks such as tuff and mudstone was discussed through case studies of a cut slope and an embankment construction, mainly from view points of strength reduction due to swelling and weathering. Concluding remarks drawn from the present study are summarized as follows.

1) Strength reduction of soft sedimentary rock due to excavation takes place by the cyclic action of drying and saturation, that is, contraction and expansion under low confining pressure, which gradually develops deeply to be a sliding failure.

2) The confining pressure to be applied on the sloping surface to prevent sliding is in the order of p_c>30kPa.

3) In case of using soft sedimentary rock as fill materials, the degree of compaction of over 80% of the dry density of its intact rock is required to avoid strength reduction.

4) Strength reduction of fill can be avoided by compacting the material to control the Talbot n-value to be n ≒ 0.3 and the 15% grain size *(D_{15})* to be under 0.5mm.

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