Difficulties encountered during construction of the Fujinosato Tunnel

T. Ikenoya*, A. Ishida*, F. Yamada**, M. Nomura**, H. Yamachi**, Shunsuke Sakurai***

Summary

Extraordinary deformation were exsperienced during construction of the Fujinosato Tunnel. These were caused by the weakness of the rock mass and inefficient growth of the ground arch. And moreover, the crushing phenomena due to slaking were thought as another reason for the deformations taking place around the tunnel.

1. Introduction

Geologically, Japan is one of the relatively complex regions of the world. Japan islands are located on a complicated subduction zone where the Pacific plate and the Philippine sea plate are subducted from east or southeast under the North-American plate and the Eurasian plate. These resulted in complex rock combinations in its plate tectonic crust, with intense fracturing, high rock stresses and seismic intensity, and the occurrence of frequent disasters. These conditions result in considerable difficulties in tunnelling. An example is the Fujinosato tunnel. The tunnel excavation faced difficulties with large deformations at almost any portion of the whole length. In this paper, various difficulties during excavation, as well as methods to overcome these difficulties, are discussed.

2. Aspects of Fujinosato Tunnel

Fujinosato Tunnel is a 242 meters long two-lane road tunnel that penatrates across the saddle of a mountain ridge. The cross section is up to 80m^2 as shown in Fig. 1. It was adopted as a horse shoe shaped tunnel with a short bench by use of partial cutting machine.

Ordinary tunnel support design is also shown in Fig. 1. As a standard tunnel support, a H-shaped steel rib (125mm in width), 17 rock bolts with 4.0m length and sprayed concrete(150mm in thickness) are installed for every 1.0m excavation.

In the section of portal, width of H-shaped steel rib is up to 200mm and 19 grouted fore-polings with 3m in length are additionally installed, along a 50m tunnel length, preparing for landslide.

* Shida Haibara Agriculture and Forest Office of Shizuoka Pref., Fujiea-city, Japan.
** Mitusi Construction Co., Ltd., Ohtemachi, Tokyo, Japan
***Kobe University, Rokko-dai, Kobe city, Japan

3. Geological considerations

Host rock mass of the Fujinosato Tunnel construction site consists of alternation of mudstone and sandstone that accumulated on shallow seabed in Miocene age. The tunnel has to pass through a landslide zone at portal section where the rock mass is extremely disturbed and weakened due to the landslide movement in the recent days.

Fig. 2 shows the tunnel axis line with relation between construction methods and geological characteristics of Fujinosato Tunnel for the sake of general understanding. From supplemental geological investigations through tunnel construction, well fissured mudstone was mainly observed, and generally the geological conditions on the right side in tunnel cross section were much poorer than on the left side.

4. Tunnel progress and difficulties

4.1. West portal section

The west portal section with 50m in length was filled with soil-cement preparing for the landslide.

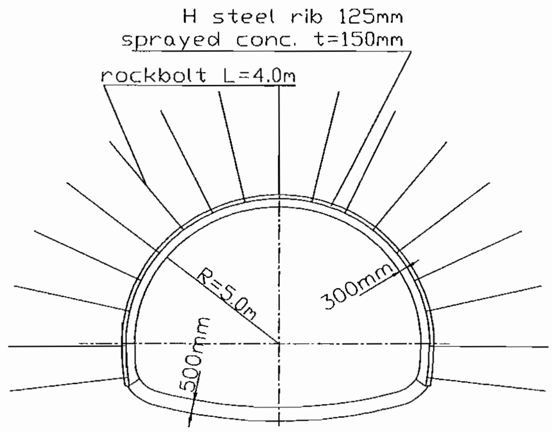


Fig. 1 – Cross section of the Fujinosato Tunnel.
Fig. 1 – Sezione tipo del Tunnel di Fujinosato.

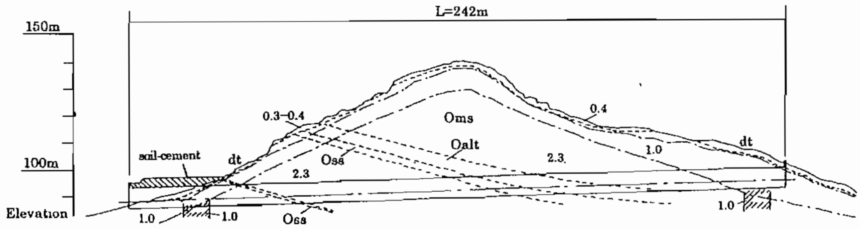
No major difficulties were encountered till 30m where collapse at tunnel crown took place, and moreover, floor heaves grew heavily .

Considering the situation, additional rockbolts with special grouting were installed into the tunnel crown and floor. The requirements for the grouting materials were as follows: controllability of the gel-time, high strength in solidity, and also high cohesive strength. Several grout materials were compared and it was decided successfully to choose the special silicate resin material as shown in Tab. I.

The final grout volume that exceeded the normal quantities gave a fair effect for ground improvement (see Fig. 3). The total grouting materials amount to some 60 ton for 68 meter in tunnel length, and that means 0.88 ton per meter.

4.2. *Large deformed section*

Following section No. 47+7, as shown in Fig. 2, a standard NATM design with a short bench was adopted. However, extraordinary deformation occurred, and moreover, surface settlement also took place. The supplemental surface investigations indicated the possibility of a small landslide and/or ground movement toward the tunnel. This settlement caused a serious influence to environment



No.	44 – 45+15	45+15 – 47+7	47+7 – 54+7	54+7 – 56+17	
Pattern		DIII	DI	DIII	
Support		200mm H steel rib 4.0m rock bolts x8 3.0m grouted for-poling x19 /1.0m excavation 250mm sprayed concrete	125mm H steel rib 4.0m rock bolts x17 /1.0m excavation 150mm sprayed concrete	200mm H steel rib 4.0m rock bolts x8 3.0m grouted for-poling x19 /1.0m excavation 250mm sprayed concrete	
Length		52m	140m	50m	

Fig. 2 – Relation between construction methods and geological characteristics.
Fig. 2 – Relazione tra i metodi di scavo e le caratteristiche geologiche.

Tab. I – Characteristics of silicate resin material.
Tab. I – Caratteristiche del materiale a base di resine e silicati.

Product	Solution A	Solution B
Composition	Sodium silicate solution, Special modifier, Catalyst	Special polymer, Viscosity reduction agent
General properties Appearance Viscosity (cp/25° C) Specific gravity (20/4° C)	 Slightly white turbid liquid 110±30 1.43±0.03	 Brown liquid 160±30 1.23±0.03
Proportion	A/B 1/0.9±1.1	
Rise time	45±15 sec (20° C)	
Compression strength (kg/cm²)	40±3	
Bending strength (kg/cm²)	35±3	
Density (g/cm³)	0.45±0.05	
Expansion ratio	6	

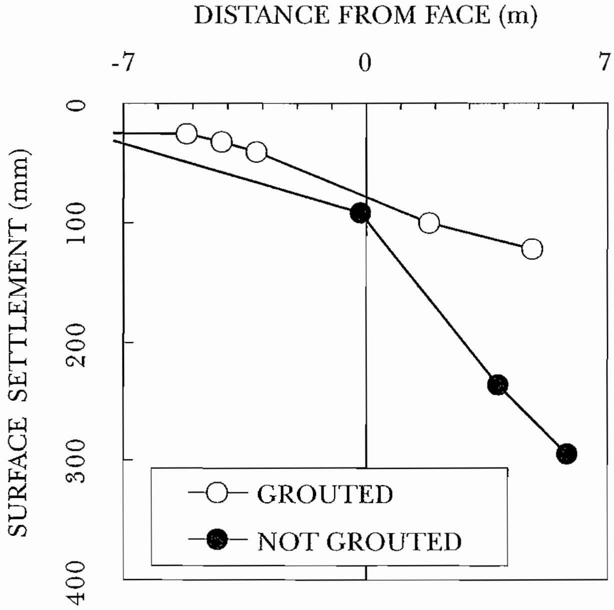


Fig. 3 – Ground improvement by grouting.
Fig. 3 – Miglioramento del terreno mediante consolidamento con iniezioni.

and/or residents of Fujinosato region, then the construction had to stop.

Fig. 4 shows the settlement of ground surface measured in section No. 46+15.; ground surface settlement attained a maximum value of 530 mm, with 230 mm occourring before reaching the face. Fig. 5 also shows the absolute displacement of tunnel wall following the top heading excavation(in section No. 47+0); the amount of right leg settlement exceeded 400mm within 6 days. It was four times greater than that of left leg. Such unequal settlement could induce instability of the tunnel, with a likely collapse.

Fig. 6 illustrates the displacement at tunnel crown and at both sidewalls (left and right), where as Fig. 7 depicts the corrisponding ground movements at the surface. The displacement at the right sidewall is shown to be much greater than observed at the left sidewall in the same tunnel section. These dispacements were shown to occur concurrently with tunnel convergence and ground surface settlement, as top heading excavation was taking place.

Based on the conditions observed during tunnel excavation as described above, two fundamental issues were posed as a need to change the tunnel excavation method:

1. how to reduce the deformations taking place around the tunnel concurrent with the excavation sequence;
2. what is the allowable strain for the rock mass under observation.

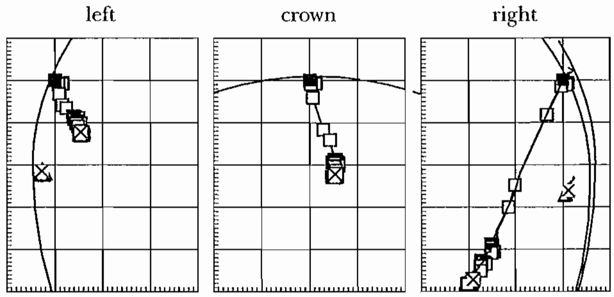


Fig. 6 – Displacements at tunnel crown and sidewalls (left and right).
Fig. 6 – Spostamenti in corona ed in parete (sinistra e destra)

Note: One block indicates 100mm displacement. Arc lines show tunnel wall (one block indicates 1.0 m). Black rectangular marks indicate the initial place of measurement point and white ones indicate dayly displacement. X marks show the place when a bench excavation reached this section.
Nota: La scala in ascissa indica 100 mm per ciascun lato del reticolo. I tratti ad arco rappresentano il perimetro di scavo (un tratto del reticolo è pari a 1.0 m). I simboli neri indicano la posizione iniziale mentre i simboli bianchi danno lo spostamento giorno per giorno. I simboli X indicano la posizione dello stesso punto nel momento in cui la sezione di scavo di ribasso ha raggiunto la sezione di misura.

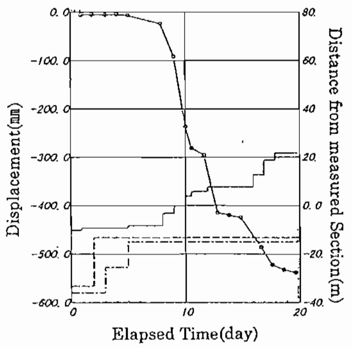


Fig. 4 – Ground settlement with elapsed time (Section No. 46+15 m).
Fig. 4 – Deformazioni del terreno in superficie in funzione del tempo (Sezione n. 46+15 m).

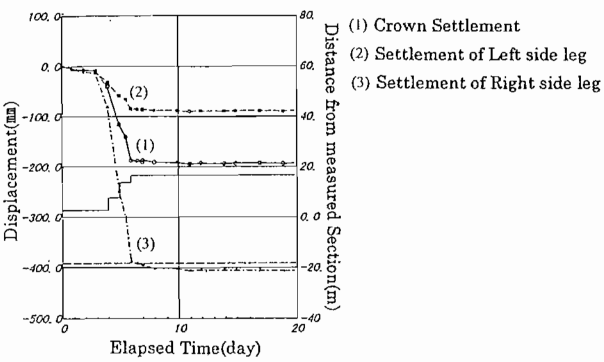


Fig. 5 – Tunnel wall movement with elapsed time following top heading excavation (at Section No. 47+0).
Fig. 5 – Spostamenti del contorno del cavo a seguito dello scavo di calotta, in funzione del tempo (Sezione n. 47+0).

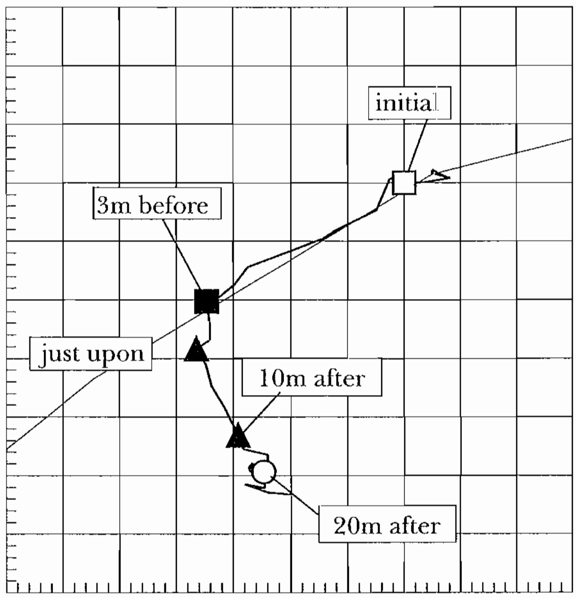


Fig. 7 – Ground settlement (Section No. 47+0).
Fig. 7 – Deformazioni del terreno in superficie (Sezione n. 47+0)

Note: One block indicates 20 mm displacement. In this figure distance between measurement point and tunnel face is shown.
Nota: Un lato del reticolo quadrato indica uno spostamento di 20 mm. È anche riportata la distanza tra il punto di misura ed il fronte di scavo.

Then, the grouting method was adopted for the remaining tunnel length, in consideration of the large plastic deformations which developed in the mudstone rock mass. The main objective was to induce the formation of a supporting arch around the tunnel, as soon as possible following excavation. Another objective was to be able to reinstall the supports including the invert arch in order to prevent further tunnel deformations. In this way the stability of the Fujinosato Tunnel could be guaranteed.

5. Expansive mechanism around the tunnel

The violent expansive deformations taking place in some tunnel sections were considered to be out of wisdom and needed to be brought under control. For this purpose a series of investigations were carried out.

A number of laboratory tests were performed in order to obtain the caracteristics of mudstone. The samples tested are of two kinds of tertiary mudstone for different degrees of weathering. The typical properties are shown in Tab.II.

Tab. II – Properties of Mudstone.
Tab. II – Proprietà dell'argillite.

Density of Grain	2.75g/m^3
Natural water content	6,30%
Fine grain content under 2μm	19,0%
Liquid limit	25,2%
Plastic Limit	13,3%
Plasticity Index	11,9%
CEC	21.2meq/100g

5.1. Slaking phenomena

The slaking phenomena were investigated by using the slake-durability test according to the ISRM recommendations. This test is intended to assess the resistance offered by a rock sample to weakening and disintegration when subjected to two standard cycles of drying and wetting. The slake-durability index Id_n was calculated as follows:

$$Id_n = \frac{m_n}{m_i} \times 100\,\% \tag{1}$$

where m_n is the initial weight of the mudrock sample and m_i is the weight of the mudrock sample after a given number of slaking cycles.

Fig. 8 shows the results of tests performed by showing the slaking durability versus the number of slaking cycles. It is clear that the slaking durability decreases as the number of slaking cycles increases. There is an obvious difference between sample A and B; sample A exhibits a relatively wide range of variation for the slaking durability from 46 to 5 per cent. The greater is the range of variation, the more significant is the possibility for the slaking phenomena to occur. Sample A was picked up from a cross section of the tunnel where large deformations took place. It is therefore obvious that such a mudstone exhibits a very high crushability due to the effect of drying and wetting.

5.2. Consistency and mineralogical composition

Based on empirical evidence in Japan, squeezing phenomena are thought to be closely related to consistency, fine grain content under 2μm, liquid limit, plastic limit, plastic index, CEC (Cation Exchange Capacity), etc. On the other hand, the mineralogical composition of rock is also believed to be playing an importan role in the squeezing phenomena. Rocks containing only anhydrite do not swell, whereas rocks containing 5 percent clay may develop a swelling pressure up to 1 MPa. It is therefore important to clear the influence of consistency and mineralogical composition of mudstone.

Tab.II shows the consistency of mudstone (sample A) and Fig. 9 compares the results obtained for the Fujinosato Tunnel with other tunnels in Japan, which are reported to have experienced serious squeezing deformations. As illustrated in Fig. 9, it is clear that the consistency of the mudstone for the Fujinosato Tunnel is much smaller than that of other tunnels. It is therefore hard to believe that squeezing would be the predominant behaviour for this tunnel.

Mineralogical analyses were also performed by using the X-Ray diffraction method, so as to clear the chemical expanding properties. The results obtained are illustrated in Tab. III and Fig. 10. Based on these data it is quite evident that there is no mineralogical component of this mudstone which could contribute to the chemical expanding behaviour and swelling phenomena would not be predicted to occur.

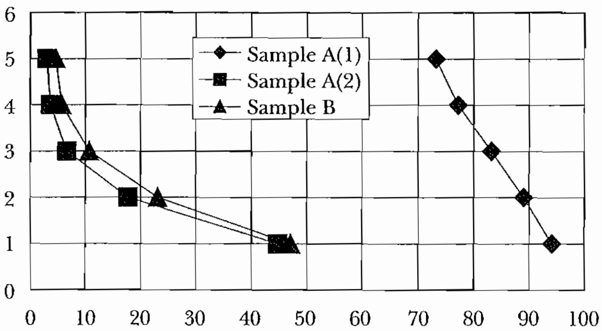


Fig. 8 – Relation between Id_n and iteration times.
Fig. 8 – Relazione tra il numero di cicli di imbibizione ed essiccamento e l'indice di durevolezza.

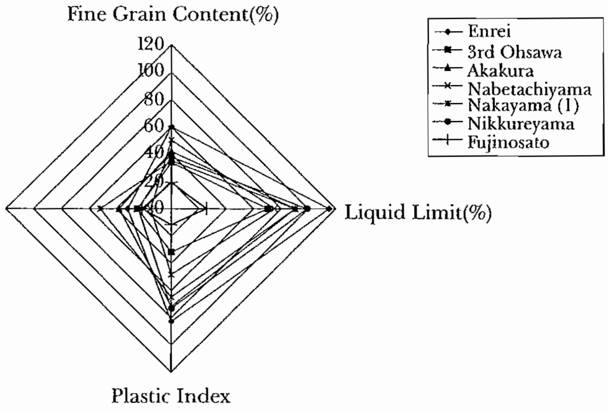


Fig. 9 – Comparison of consistency with other tunnels.
Fig. 9 – Confronto di consistenza con i dati di altre gallerie.

Based on the above factors, one can easily say that the extraordinary deformations experienced in the Fujinosato Tunnel would not depend on the effect of consistency or mineralogical components of rock. They would rather be caused by the slaking phenomena.

5.3. Multi-stage triaxial compression tests

Multi-stage triaxial compression tests were carried out by means of artificial test-pieces (40×40mm×80mm in rectangular solid shape) composed of mudstone in-situ. Test pieces were previously kept for a certain period in vacuum desiccator in order to assure natural water content. Multi-stage triaxial compression tests were conducted under different confining pressure for the same test piece. In this case, confining pressure was changed from 0.5 MPa to 2.0 MPa for every 0.5 MPa. For every confining pressure stage, loading was conducted twice, first loading called virgin loading and another called residual loading.

Fig. 11 shows the relationship between differ-

Tab. III – Results of X-ray diffraction method.
Tab. III – Risultati ottenuti mediante analisi diffrattometrica a raggi X.

Mineral	Intensity of X-ray	Rank
Quarts	>600cps	+++
K-feldspar	<20cps	
Plagioclase	100·300cps	+
Mika clay Mineral	<100cps	-
Kaolinite	<20cps	
Chlorite	<100cps	-
Smectite	<20cps	
Random mixed-layer mineral	<20cps	

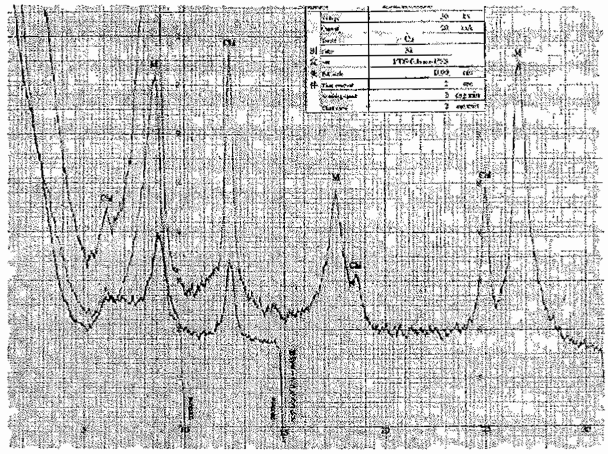


Fig. 10 – Results of X-ray diffraction method Sample B.
Fig. 10 – Risultati di analisi diffrattometrica a raggi X - Campione B.

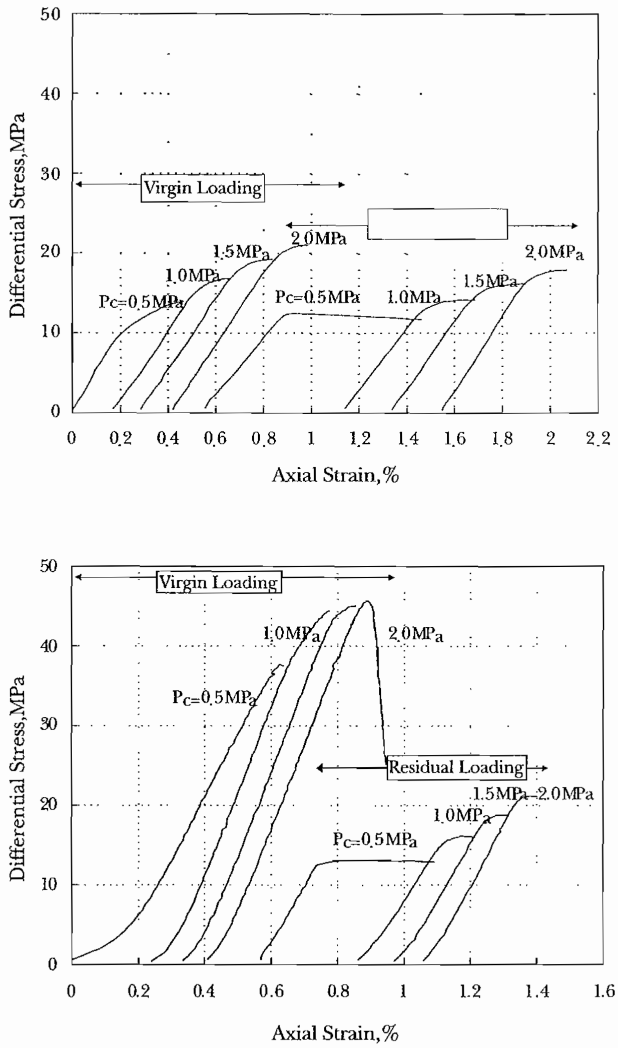


Fig. 11 – Relation between differential stress and axial strain in multi-stage triaxial compression tests.
Fig. 11 – Relazione tra tensione deviatorica e deformazione assiale in prove di compressione triassiale multifase.

ential stress and axial strain. For sample A, the loading behaviour is shown to be slightly different between virgin loading and residual loading. On the other hand the reduction of peak strengh at every confining pressure results to be quite evident. In particular, it is of interest to point out that the loading behaviour of sample A at virgin loading is very similar to that of sample B at residual loading. Then it would be thought that the effects of slaking with tunnel excavation converted mudstone of sample A into clastic mudstone of sample B.

5.4. The extraordinary large deformations due to slaking phenomena

In the case of tunnelling the following explanation can be provided. The rock mass surrounding the opening is stress relieved due to excavation, and the loosened zone increases its volume simultaneously. As a result of the expansion of this zone, the ground water looses its resistance due to weakening and disintegration (i.e. the slaking phenomenon), and therefore should become easy to deform.

6. Conclusion

The deformations observed during excavation of the Fujinosato Tunnel in the mudstone length are caused by the extreme weakness of the rock mass and are due to inefficient growth of the ground arch around the tunnel. In addition, the crushing phenomena due to slaking are believed to be another reason for the extraordinary expansive behaviour observed. The existence of a ground water flow would act to accelerate the crushing phenomena and to weaken the rock mass strength.

Difficoltà incontrate durante la costruzione del Fujinosato Tunnel

Sommario

Durante la costruzione del Fujinosato Tunnel si sono verificate deformazioni del cavo del tutto inusuali. Questi fenomeni sono stati causati dalle caratteristiche particolarmente scadenti dell'ammasso roccioso e da una formazione poco efficace dell'arco portante. Inoltre, una causa aggiuntiva delle deformazioni verificatesi è da farsi risalire ai fenomeni di rottura associati al decadimento delle proprietà meccaniche della roccia.