Slope Consolidation of the Banks of the Monguelfo Reservoir, Italy

La consolidation des talus du réservoir de Monguelfo (Italie du Nord)

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Summary

Slope stability of the banks of the Monguelfo Reservoir Italy, which is part of a new hydroelectric scheme in is partially responsible for the stability of an adjoining railway embankment.

General site conditions were examined in advance by means of both geological and geoseismical investigations.

Geotechnical features of the sandy-gravelly soil with some content of silt and clay forming the banks of the river, were analized by means of shallow and deep soundings, permeability field tests, and laboratory soil tests.

The dangers of slope failure as well as of superficial erosion arising from reservoir operations were analysed, taking account also of the actual behaviour of the banks of another reservoir in the same valley.

Slope consolidation was obtained, where necessary, by addition of a twin zoned shell : the outer zone being of dumped and sluiced rockfill (slope 2/1) and the inner one of transition material. Berms 2·50 m wide were provided at minimum water level and on top of the shell.

Stability analysis was performed using the modified Swedish Method.

The whole work involved dumping or rolling of about 160 000 cu. m of rock and soil on an embankment 1 200 m long, corresponding to 25 per cent of the total.

Tests of filling and drawing down were successfully performed before the reservoir was put into service.

Sommaire

La stabilité des talus du réservoir de Monguelfo, qui fait partie d'un nouvel aménagement hydroélectrique en Italie du Nord est partiellement responsable de la stabilité d'un remblai voisin appartenant aux Chemins de Fer Italiens.

Les conditions générales de la zone intéressée ont été reconnues par des études géologiques et géosismiques.

Les caractéristiques du sol sablo-graveleux à faible teneur en limon et argile, qui forme les rives du fleuve ont été analysées au moyen de sondages superficiels et profonds, par des essais de perméabilité effectués sur place, et par des essais de laboratoire.

On a étudié le danger de rupture et d'érosion superficielle du talus comme conséquence des changements de niveau du réservoir, et en tenant compte du comportement des rives d'un autre bassin situé dans la même vallée.

La consolidation a été réalisée au moyen d'un revêtement en deux couches. La couche extérieure est en enrochements ayant une pente de 2/1, tandis que la couche intérieure est composée de matériaux de granulométrie intermédiaire.

De même ont été réalisées deux risbermes de 2,50 m de largeur, l'une à la hauteur du niveau minimum de la retenue, et l'autre à un mètre au-dessus du niveau maximum. Les calculs de stabilité ont été faits avec la méthode suédoise modifiée.

L'ouvrage comportait la mise en place d'environ 160 000 m^3 d'enrochements et de terres sur une longueur des rives d'environ 1,2 km, correspondant aux 25 pour cent de la longueur totale.

On a effectué avec succès des essais de remplissage et de vidange avant de mettre en service la retenue.

1. Monguelfo Reservoir is a part of the Brunico Hydro electric Scheme, built by Società Idroelettrica Atesina (Montecatini Group) in northern Italy close to the Austrian border.

This reservoir, with a storage capacity of 5 million cubic metres, was created by building a 50 m high, dome-shaped arch dam across the River Rienza at Monguelfo.

The authors describe the method followed to consolidate the reservoir slopes, the stability of which is partly responsible for that of the Fortezza-San Candido railway embankment (on the left side) and of the Val Pusteria State Highway (on the right side).

From the geotechnical standpoint, the problem was analysed by the usual methods of Soil Mechanics.

2. The preliminary geological investigations showed that the rocks of the reservoir area belong to archaezoic quartzous phillides, covered, both on the valley sides and bottom, by a thick stratum of alluvial deposits and scree.

Geophysical methods were employed to determine the rock formation. The seismic refraction method was used and it showed that, upstream of the left dam abutment, the bedrock surface dips downward to form a side valley constituting the old river bed, about 100 m below the present stream bed. This depression has been filled with alluvial deposits containing boulders of large size, embedded in finer sand deposits with impervious clay and silt layers interposed at different depths.

The river beds join about 2 km downstream of the dam.

Deep test boreholes revealed the presence of ground water pressure in the pervious alluvium filling the ancient valley. The results of these investigations were considered positive in respect of the water tightness of the reservoir. A large part of the bottom surface (150 000 sq.m) was covered with an impervious blanket of rolled silt about 30 cm.thick ; suitable material was borrowed from a local pit.

3. The extension of slopes, the stability of which was analysed in order to ensure the safety of the nearby railway and road, is about 5 km. Consolidation was carried out over an area 1·2 km long (Fig. 1). For the remaining areas it was envisaged that limited local landslides could be permitted without causing unnecessary danger.

The average properties of the overburden on the slopes (which do not differ appreciably from zone to zone) were tested in the Soil Mechanics Laboratory of the Milan Institute of Technology. This material may be classified as a "sandy-gravelly soil with some content of silt and clay". Grading was by sieve and hydrometer analyses ; the sand/gravel ratio is practically 1 : 1 and the percentage of particles smaller than 7 microns is approximately 3·5 (Fig. 2).

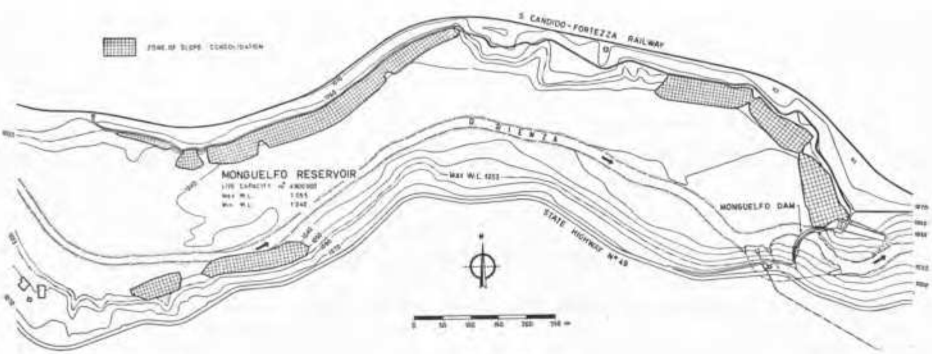


Fig. 1 Monguelfo reservoir. Plan of slope consolidation work.
Réservoir de Monguelfo. Plan des travaux de consolidation des talus.

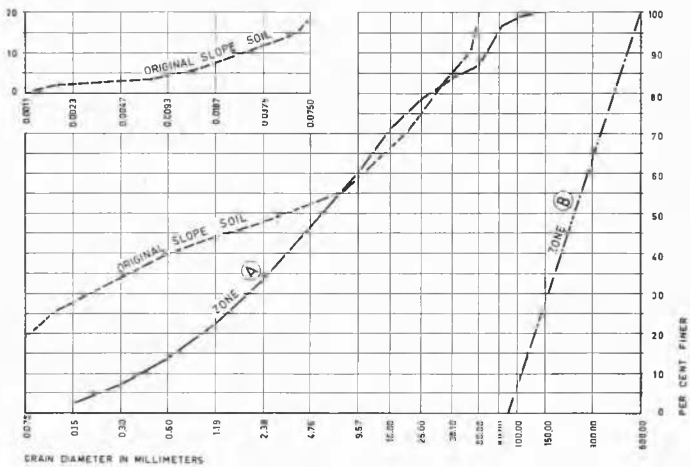


Fig. 2 Slope protection. Grading diagrams.
Protection des talus. Diagrammes granulométriques.

Direct shear tests on remoulded samples proved that an internal friction angle of 33° and a cohesion of 0·2 kg per sq. cm can be assumed in the calculations.

Permeability tests performed on remoulded samples in Hogentogler equipment gave a maximum permeability coefficient $k = 10^{-5}$ cm/s, the load on soil being 1-2-3 kg/sq. cm.

Direct measurements of ground permeability were performed on site by pumping tests in filtering wells and level observations in piezometric pipes. Average permeability coefficient k is about $5 \cdot 10^{-3}$ cm/s, the variation limits being 10^{-2} cm/s and 10^{-1} cm/s.

4. Slides in the reservoir may originate from the following causes :

(a) erosion on the surface due to the dynamic action of water ;

(b) the action of seepage in reducing the internal balance of the whole slope or of a part of it ;

(c) the action of frost and thaw, particularly effective in this area, where the winter temperature is extremely low and the lake surface is covered by a thick layer of ice during some months of the year. The reservoir was built for the purpose of daily regulation of the water flow to the power station and therefore wide slope areas are affected by the action of frost and thaw.

The most rapid drawdown during normal operation is effected when the hydroelectric plant uses the maximum possible water flow and simultaneously the natural runoff arriving at the reservoir is a minimum ; this may happen in Winter. Under this condition the maximum drawdown rate is 0·27 m per hour.

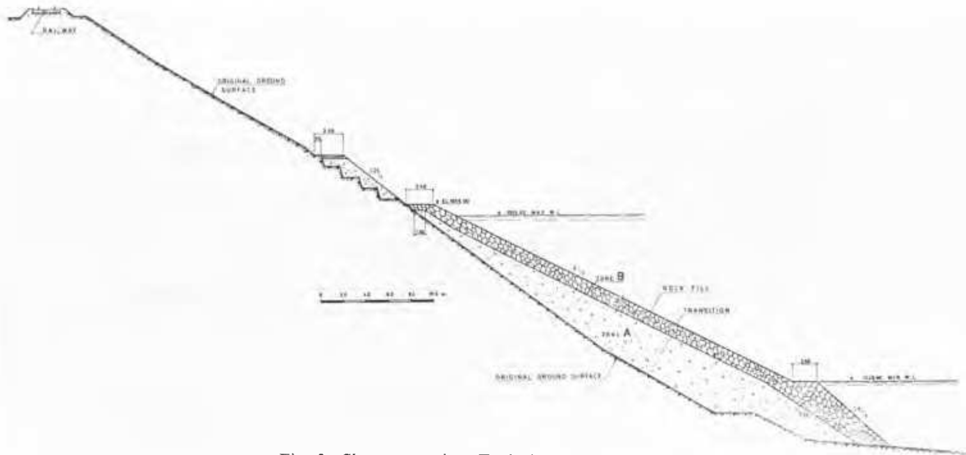


Fig. 3 Slope protection. Typical cross section.
Protection des talus. Coupe type transversale.

5. The various alternatives examined for slope consolidation were as follows :

(a) slope protection by means of masonry and concrete structures (slabs, columns, arches, supporting walls, etc.);

(b) reduction of the natural bank slope by the addition of suitable material.

The former alternative represents a complete solution of the problem of both stability and surface erosion, but requires a firm foundation base, such as solid bedrock.

Since in the present case this was not available, the foundation had to be taken down to a depth sufficient to avoid any danger arising from reservoir operation. Studies and cost analyses showed that this alternative was economically prohibitive.

On the other hand, the experience gained by the authors in several similar cases was in favour of the latter alternative.

A significant example is protection of the State Highway running along the large S. Valentino Reservoir (110 000 000 cub. m) located in the same region. The slope protection was obtained by a layer of rockfill with a slope of 1 : 1·6. No trouble was experienced during the first ten years of operation of this reservoir.

The bank features, the slopes of which are based on alluvial ground, reveal that this problem is similar to that of the upstream slope stability of an earth dam on a porous foundation.

From these considerations the idea was derived of integrating the existing slopes in such a manner as to achieve a safe slope, using something similar to the rock rip-rap of an earth dam resting on a transition zone which acts as a filter.

The behaviour of the embankment areas left without protection can be easily foreseen. Experience was gained at the Rio of Pusteria Reservoir, located in the same valley, the slope of which consists of material closely similar to that at Monguelfo. Slope slides recorded in about twenty years of operation took place only on the steepest zones and the slope stabilized itself on a 45-degree slope.

6. In conclusion, after having examined several types of slope protection, also within the selected alternative, a twin-zoned protective shell having the following features was adopted (Figs. 3 and 4) :

Fig. 4 View of a consolidated stretch of slope.
Vue d'une zone à talus consolidés.

(a) Slope protection starts 1 m above the highest water level (1 055 m above sea level). Thickness at the top is 2·5 m. Slope inclination is 1 : 2 and remains unchanged down to the minimum water level, 15 m below. The shell consists of a rockfill layer (material *B*), 1 m thick, overlying a transition layer of finer grading (material *A*), acting as a filter.

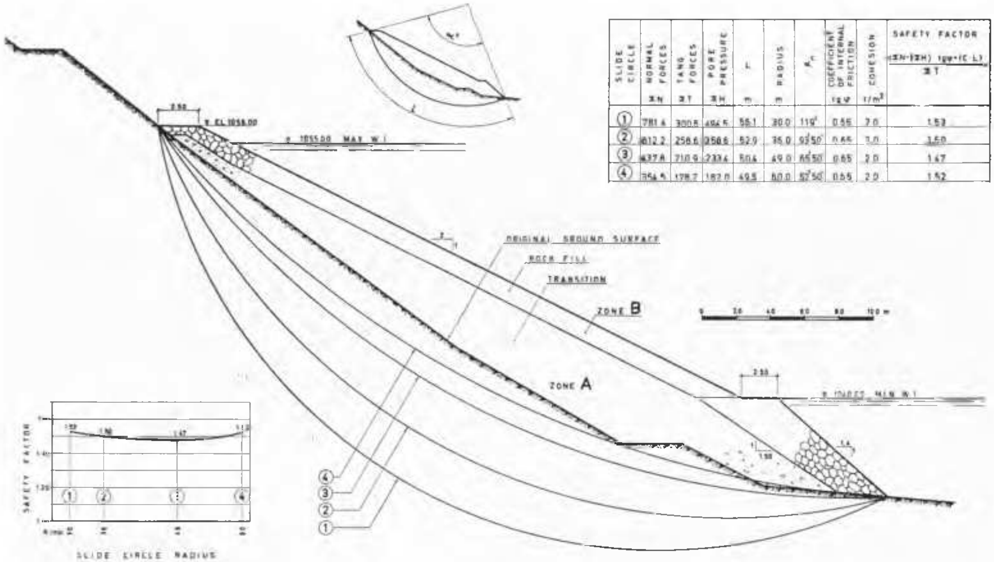
(b) A 2·5 m berm was provided at the minimum water level (1 040 m above sea level). Below this elevation the outside slope is equal to the natural angle of repose of dumped rockfill (about 1/1·4).

Before placing, the surface was thoroughly stripped and all humus, roots and residual organicmaterials were removed.

The transition material (*B*) was placed in layers and rolled. Its grading was selected so as to ensure an effective filtering action. According to Terzaghi's recommendations, the following relationship was used :

$$5B_{85} > F_{15} > 5B_{15}$$

F_{15} being the diameter which corresponds to 15 per cent in the grain size diagram of material *A*;



SLIDE CIRCLE	NORMAL FORCES ΣN	TANG FORCES ΣT	PORE PRESSURE ΣH	L m	RADIUS m	α	COEFFICIENT OF INTERNAL FRICTION tg φ	COHESION t/m²	SAFETY FACTOR $\frac{(\Sigma N-\Sigma H)\,tg\varphi+(C\cdot L)}{\Sigma T}$
①	781.4	300.8	494.5	56.1	30.0	119°	0.55	2.0	1.53
②	812.2	258.6	258.6	52.9	35.0	93°50'	0.65	2.0	1.50
③	437.8	710.9	233.6	50.6	49.0	65°50'	0.65	2.0	1.47
④	354.5	178.7	182.0	49.5	60.0	57°50'	0.55	2.0	1.52

Fig. 5 Stability analysis of a typical cross section.
Calcul de la stabilité d'une coupe transversale type.

B_{15} and B_{85} the diameters which correspond to 15 pour cent and 85 pour cent respectively in the grain size diagram of the original slope material (Fig. 4).

The rockfill is formed by elements having a minimum size of 100 mm ; 40 per cent there of has sizes larger than 300 mm (Fig. 2).

Compaction of rockfill was obtained by dumping and sluicing with water under pressure.

Both rockfill and transition material were hauled in lorries from pits located at some distance from the site.

7. Stability analyses were performed using the slip circle method (May modification) for the cross sections which were considered to be relatively weak.

The data assumed for calculations are those resulting from the soil tests.

Analysis was done on the assumption of rapid draw-down. A residual pore pressure was assumed on the slip circle equal to the depth of each point in respect of the slope surface; the total resulting uplift was multiplied by a reduction coefficient equal to 0·5, taking into account the soil properties and the fact that a rapid draw-down is practically impossible.

The minimum safety factor resulting was approximately 1·5 and therefore regarded as completely satisfactory.

8. The work involved dumping or rolling about 160 000 cub. m of material over a length of banks of 1 200 m of embankments, corresponding to 25 per cent of the total slope length. Ancillary works, such as access roads, were also carried out.

Several successful filling and drawdown tests were carried out before the reservoir was put into service.