Slope Stability Rehabilitation By Geomembranes In Greece

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Keywords: Case study, Erosion control, Geomembranes, Landslides, Rehabilitation

ABSTRACT: In the course for rehabilitation of a steep slope in Central Greece where the natural gas pipeline had already been constructed and landslide phenomena due to excessive rain water infiltration and run-off erosion were developed, the design and use of a geomembrane cover combined with deep and surface drains and slope reshape offered a major cost - effective solution facing radically the instability problems.

1 INTRODUCTION

The use of geomembranes in association with other geosynthetics has already been a long tradition in geotechnical engineering applications dealing mainly on soil and aquifers protection against pollution and general environmental applications. On the contrary, their indirect contribution in slope stability and rehabilitation, although very simple (preventing water infiltration and therefore increase of pore-water pressure) and practically needing minor design requirements is considered as equally important and provides often satisfactory and cost-effective solutions in stability problems under special conditions. Within this context, a slope rehabilitation design using geomembranes took place in Central Greece, concerning a mountainous area where the Natural Gaz Pipeline was placed along a steep slope with major instability occurencies.In the area of interest, the pipeline had been placed along a slope with 160 m difference of elevations with upper profile inclination of 30° - 35° and lower profile inclination of 25° - 30°, where surface and relatively deeper failures and slope movements have been reactivated upon the end of construction period for the pipeline, prior to the inflow of pressurised gaz. An extensive geological and geotechnical investigation program was then performed, including monitoring and site experimental partial applications of proposed rehabilitation method extended over 4 years of work (1995 - 1999). The final contract and rehabilitation stage was concluded by 11/99 and monitoring is still active.

2 SITE CONDITIONS - MODE OF FAILURE - STABILITY ANALYSIS

Morphologically, the broader area was characterized by the presence of steep slopes, shaped ravines and valleys, with generally dense vegetation and during winter time, snowfalls, intense rainfalls and the presence of frost are very common.

The geological bedrock along the slope was characterized by the presence of a thick intercalating schist - chert system, with intensive fracturation and erosion, and a relatively compact system of ophiolites, as lavas or ultrabasic serpentine. At the foot of the slope where the morphology was less steep and more levelled, the system of schist - cherts with marly limestones generally appeared less fractured and eroded near the surface. Those formations were covered by the weathering mantle and the volcanic sedimentary formations with variable thickness. At the upper part of the slope, the surface formations had a maximum depth of 15 m decreasing to the foot of the slope, where marly limestones appeared.

The weathering formations of the schists - chert system and the fractured cherts as well as the excavated deposits after the construction works for the pipeline placement were generally permeable formations because of the weathering conditions and the low percentage of clayey content measured in the laboratory tests (between 0 ÷ 10%). The underlying bedrock was generally an impermeable formation.

During the construction works and after the excavation works along the route in addition to the vegetation removal, plenty of soil fractures and faults extended further to the excavated area to the natural slope were revealed. The main fracture had a petal form with a width equal to 70.0 m and a length equal to 90.0 m. At the top of the fracture the faulting step was about 30 cm with an opening of up to 20 cm.

At the East side of the slope and further to the excavation zone the faulting step was 50 cm with an oppening of up to 60 cm. At the west side of the slope and further to the excavation zone, the highest step about 2.0 m high had an opening of 10 - 20 cm.

Deep points monitoring assured by inclinometers presented a sliding zone in a depth varying from 2 to 6 m with a sliding direction in an angle of about 19° in relation to the pipeline axis.

Surface points monitored by topographical methods presented a maximum displacement of 41 cm for the first year approximately, when rains were rather intense.

The magnitude of the displacements was low during the dry period of each year. Those displacements accelerated with the first rainfalls of the year at the same direction and angle with the pipeline axis and caused additional surface fractures during saturated conditions of the loose formations.

The determined failure model obeyed to the following procedure, which mainly guided the design proposals to involve the use of geomembranes :

a. The slope stability under dry conditions was marginal. Because of the excavation works a certain area around the pipeline route had been essentially disturbed. The removal of the vegetation, the accumulation of the excavated material without control on the slope and generally the disturbance of the already weak surface stratum created the presumptions for landslides and creep movements.
b. The surface zone around the pipeline had been seriously subjected to external loadings during construction works. Also the soil fractures and movements, which already upset the stability conditions, had seriously affected the slope by decreasing its mechanical behaviour to the significantly lower residual strength of the formations.
c. The slope stability under "pore pressure" conditions was critical. As long as the infiltrating underground water was subjected to level variations during the year depending to the rainfalls, the sliding zones and the interface between the surface permeable strata and the impermeable bedrock reached a very low bearing capacity.
d. The water content in the slope mass (rainfalls and hanging aquifer) created the appearance of several springs along the slope. According to the geological conditions the impermeable bedrock appeared from the middle of the slope to its foot almost on the surface or with a very thin weathering mantle. So, the appearance of the springs resulted to the creation of a plastic zone along the foot of the slope, proved also by the presence of several fractures around the spring.
e. The instability phenomena were more obvious and effective when the water content and free run - off created saturation conditions along the weathered mantle.
f. The residual strength of the weak formations was a result of failures activated by the water presence and saturated conditions in the slope and not a result of any external seismic acceleration, still needing to be taken into account (a = 0.58 g).

For the exact evaluation of the geotechnical parameters for each one of the geological formations encountered along the slope, the results of the site investigations by trial pits and boreholes, the geological conditions, the surface fractures, the underground water level variation and the inclinometric deep movements were taken into consideration. Under those conditions, the slip surface and the intense loose zone of the moving subsoil were determined. Along this loose surface, generally activated by the underground water circulation, the soil mass movement occurred with a circular most probable failure mode, refering to the surface formations overlying the impermeable bedrock.

The fact that during the observation period the highest soil displacements and fractures corresponded to moving masses of mostly saturated soil allowed for the exact evaluation of the residual strength of the surface soil formations. The safety factor corresponding to circular failure of the saturated top soil stratum was supposed $F = 1.00$ and back analysis technique was applied.

In addition to stability analysis, special stress analysis was also performed along the pipeline axis in order to determine stress - displacement relations under the maximum design possible earthquake (a = 0.58 g). According to the results, tensile stresses appeared along its upper part larger in absolute value and compressive stresses were concentrated at its lower part especially in the area where the pipeline crosses out of a soft surrounding soil into a more stiff soil embedment. The pipe under no internal pressure withstands the landslide scenario elastically up to a total imposed displacement of a little more than 2.00 m.

The main aim of the finally proposed stability measures for slope rehabilitation was to allow a development of dewatering conditions along the loose geological formations, the decrease of the pore pressures along the sliding zones and the interface between the surface permeable strata and the impermeable bedrock, as well as the creation of adequate benches, acting as small dams against the superficial type of mudflow. The main purpose was to keep the critical zone under dry conditions increasing thus the residual stability and shear resistance of the loose formations.

In this way, an important delay of the surface soil mass movements could be expected.

3 SLOPE REHABILITATION DESIGN AND CONSTRUCTION

3.1 *Proposals*

According to the principles issued by the stability and stress - displacement analysis performed, the proposed construction rehabilitation measures were the following :

a) Earth moving works and slope reshaping in the neighbouring zone and the larger area of interest. No excavated soil masses were allowed to be stockpiled on the slope, even temporarily.
b) Construction of soil benches (artificial), transverse to the pipeline route.
c) Construction of a collecting draining triangular trench along the pipeline route (Fig. 1).

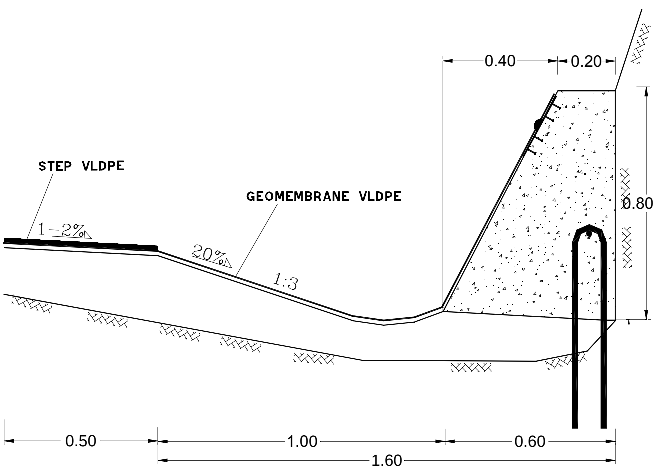

Figure 1. Detail of triancular trench in between bench zone.

d) Placement of surface drains over the zone of appeared water springs and soil deformations.
e) Sealing of the slope by an adequate geomembrane system against surface water filtration erosion and rapid saturation, of the slope. The geomembrane was to be placed over the prepared surface following the relief created by the benches, for a length of 2 m, also acting as its stabilisation system. The geomembrane sheets were placed along the slope from the end of the top bench (not covered) using an overlap of approximately 1.00 m, adequately welded. The geomembrane covered also the longitudinal trench eastwards and was anchored within special concrete abutments. At the areas where the emembrane met each transversal bench of the slope, a granular material cover was foreseen, reaching the total bench height (Fig. 2).
f) Construction of surface channels and plantation of the neighbour area.
g) Continuing measurements of the monitoring network with interim inclinometers and piezometers in the slope and strain gages on the gas pipe.

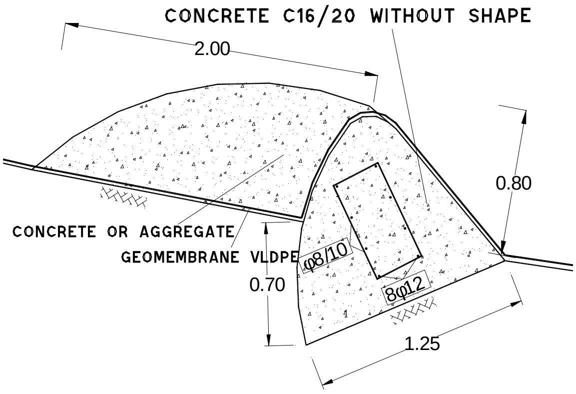

Figure 2. Detail of typical bench

3.2 *Geomembrane*

The proposed use of a water - proofing liner covering the totality of the rehabilitated slope was a result of a rather simple design procedure but a more complicated structural application, in terms of constructional plans, with details necessary for site realisation.

The geomembrane simple design refered mainly to the determination of necessary thickness, able to withstand the maximum differential displacement of 200 cm, as calculated for the maximum design earthquake of a = 0.58 g, checking also the possibility to resist wind uplift during placement.

Design of the adequate function thickness t_{all} of the geomembrane involves the assumption of a cost - effective safety factor F expressed as the ratio between the minimum necessary design thickness $t_{\min}$ to the allowable function thickness t_{all} as :

$$F = \frac{t_{\min}}{t_{all}} \tag{1}$$

This safety factor (ranging between F = 2 to 5 depending on special construction conditions and the polymer type of the liner) was conservatively assumed to be F = 4, since strict rules for the welding procedure and seams of the liner parts were insignificant for the specific type of application.

The simple design model, as proposed by R. Koerner is presented at next figure 3, where deformation is induced as differential settlement ΔH of two parts of the liner.

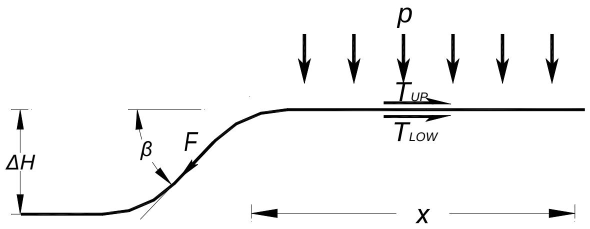

Figure 3. Design model used to calculate geomembrane thickness

By checking the sum of forces in the X direction and setting it to zero to assume perfect equilibrium state, the following equations are valid :

$$\Sigma F_x = 0 \tag{2}$$

$$F \times \cos\beta = T_{UP} + T_{LOW}$$

The unfavorable case of water presence above the liner was considered, therefore $T_{UP} = 0$. Then:

$$(t_{all} \,.\, \sigma_{all}) \times \cos\beta = p \times \tan\delta_L \times x$$

$$t_{all} = \frac{p}{\cos\beta} \times \frac{x}{\sigma_{all}} \times \tan\delta_L \tag{3}$$

and inducing the safety factor $F = 4$ we obtain

$$t_{\min} = F \times \frac{p}{\cos\beta} \times \frac{x}{\sigma_{all}} \times \tan\delta_L \tag{4}$$

where
ΔH = settlement mobilizing the stresses (ΔH = 200 cm),
F = force mobilized in the liner (= $\sigma_{allow} \times t$),
σ_{allow} = liner allowable stresses,
t = liner thickness,
T_{UP} = shear force on top of liner (zero force)
T_{LOW} = shear force below liner,
$T = p \tan\delta$,
p = applied pressure from run-off water and snow
δ = angle of shearing resistance between liner and the adjacent soil material, and
x = distance of mobilized liner deformation.

Since no special laboratory tests to determine direct shear resistance were available (e.g. large shear box tests), the corresponding input data were assumed mostly based on previous experience (water containment ponds) to be :

$\beta = 45°$, $\delta_u = 0$, $\delta_L = 30°$, $x = 5$ cm, $p = 20$ kPa.
$\sigma_{all} \approx 2400$ kPa (conservative).
The required thickness was calculated :

$$t_{\min} = 4 \times \frac{20}{\cos 45°} \times \frac{0.05}{2400} \times \tan 30° = 1.36 \times 10^{-3}\, m$$

The final thickness selected for the geomembrane was 1.50 mm.

The effect of wind uplift ΔP to determine the necessary anchoring tube elements on the slope was estimated as proposed by M. Wayne - R. Koerner for a maximum wind velocity V_p = 50 miles/hour as follows:

$$\Delta_p = 0.0025 \times V_p^2 = 6.25 \text{ lb/ft}^2 \approx 300 \text{ N/m}^2$$

and for the total slope length of 200 m approximately the maximum possible force developed on the geomembranes was :

$$F = 200 \times 200 = 60.000 \text{ N/m} = 60 \text{ kN/m}$$

The anchoring system of the liner on the slope was designed with the use of abutments for liner stabilisation by constructing partial cells of 20 × 20 m surface each, offering a stabilizing force of 4 × 18 kN/m = 70 kN/m > 60 kN/m.

The geomembrane support - stabilisation elements consisted of C12/15 concrete, placed laterally to the grounds dip, while also over the ends. The anchorage of the geomembrane with the concrete sidewalls was of special significance. The anchoring elements were specific water stops of a nominal width of 240 mm in panels, which were covered by the cast concrete for the safe anchorage of the membrane.

The geomembranes panels had a nominal width of 6.0 m. A geomembrane of low density polyethylene (VLDPE) of a thickness of 1.50 mm was selected. The liner was placed on the slope starting from the upstream area, where they were temporarily kept in place with sand bags.

Over the prior constructed berms, following the installation of each geomembrane panel and its welding, a balast comprising lean concrete was placed. The precast berms consisted of C16/20 concrete and for construction purposes where joints due to interruption of works were anticipated, these were internally reinforced with lateral reinforcement 8Φ12, surrounded by a mantle for protection against shear equal to Φ8/10. Parallel to the membrane surface and mainly in the location of seams, 10 kgr weights were placed every 2 m, interconnected with a special chain, so as not to cause possible local uplift of the liner (Fig. 4).

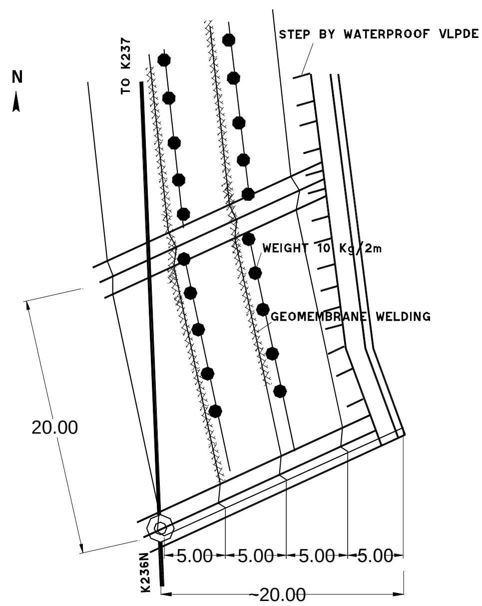

Figure 4. Detail of geomembrane support against uplift

Permeability tests of the seams were generally performed but in any case the purpose of welds and overlaps of the panels was to ensure that the water will run-off the surface of the membrane towards the side trench along the natural gas pipeline axis without infiltrating in the ground.

In the downstream longitudinal section of the liner, a shallow ditch was formed for the outflow of water. In its upper end, the ditch was followed by precast elements for avoiding weathering.

The rainwater over the area of the liner was designed to be discharged via a side trench comprising concrete elements, so as to avoid weathering. This side trench was constructed with special formation of the lateral dip of the ground. At the end of the side trench a surface collection ditch and diversion of water towards the surface elements and towards the downstream area was foreseen.

4 MONITORING

Permanent monitoring of the slopes behaviour and of the stress conditions applied to the pipeline by means of strain - gages were included in the design of the slope rehabilitation and protective measures. In the area of the pipeline, six inclinometers and four piezomenters covered by the liner were set in operation. Additional monitoring instrumentation for measurement of the stress conditions with strain gages were placed over six locations on the metallic pipe prior to its back filling.

At the locations of all instruments, an appropriate hole was opened in the membrane, from where the piezometer or inclinometer pipe or the ends of strain gages would protrude. A relative impermeability of that point was ensured by a special polyethelene collar welded on the membrane and fixed to a metallic tube that surrounded the inclinometer or piezometer.

Access to the locations of monitoring instruments against the steep slope was achieved with the construction of berms using water stops of VLDPE with elements of adequate width fixed over the membrane.

All monitoring was fully automated using a photovoltaic arc in a special structure, recording all results on site and providing the necessary data to a data-logger connected with the electrical source.

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