Vertical Expansion of a 41-m High Geosynthetic Reinforced Soil Slope

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ABSTRACT: The Pampa Verde waste dump expansion is highly driven by the following constraints: (1) relatively limited options for laterally extending the existing buttress; (2) the need to satisfy both static and seismic stability; (3) the need to properly select high-strength reinforcement products; (4) the importance of conducting material-specific tests to characterize the soil-geosynthetic interaction; and (5) the complex geometry of the overall system, particularly its global stability. The existing facility is stabilized by a geosynthetic-reinforced toe buttress that includes a compacted earth fill, with a slope of 2.1H:1V, overlying an mechanically stabilized earth wall (MSE or reinforced soil). The MSE structure involves upper and lower MSE wall sections with a horizontal step in between. The upper section is a geogrid-reinforced wall, while the lower section is a Terramesh system involving gabions reinforced with geogrids. The overall design approach involves the design of a geosynthetic-reinforced soil slope (RSS) that would use geogrids of high tensile capacity. The design guidelines for the proposed RSS are those outlined by the US Federal Highway Administration (FHWA-NHI-10-024). A key aspect of the proposed design is the proper evaluation of the soil vs. geosynthetic interface; a total of five geogrid products were considered for possible use in the construction of the geosynthetic-reinforced slope at the Pampa Verde project.

1 INTRODUCTION

1.1 *Project Location*

The La Zanja mining project, owned by Minera La Zanja S.R.L. (Minera La Zanja), is located in the district of Pulán, province of Santa Cruz de Succhabamba, in the department of Cajamarca, Peru, at an altitude that varies between 2,800 and 3,800 meters above sea level (masl).

The Pampa Verde waste dump is located south-southwest of the Pampa Verde pit and comprises a total area of approximately 188,525 m^2, after vertical expansion (effective area that does not include perimeter access, diversion channels and cut and fill slopes).

1.2 *Background*

The Pampa Verde waste dump was designed by Knight Piésold Consultores S.A. (Knight Piésold), considering that inside it would be encapsulated an unsuitable material stockpile.

In December 2013, the construction of the containment dike of the Pampa Verde wasted dump was finalized, consisting of a 29-m high compacted earth fill, with a slope of 2.1H:1V, overlying a mechanically stabilized earth wall (MSE or reinforced soil). The MSE structure involves upper and lower MSE wall sections with a horizontal step in between; the upper section is a geogrid-reinforced wall, while the lower section is a Terramesh system involving gabions reinforced with geogrids. Figure 1 shows an aerial overview of current conditions at the waste dump and Figure 2 shows an overview of the existing MSE structure and detail of the existing reinforced toe buttress.

Figure 1. Caption of the overview of the Pampa Verde Waste Dump.

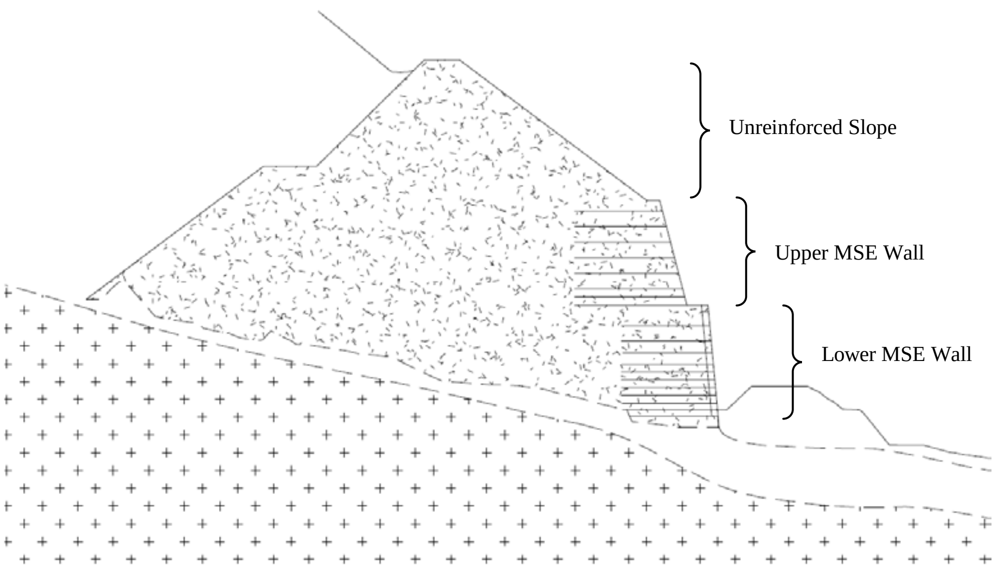


Figure 2. Cross section of the existing geogrid-reinforced toe buttress.

During the operation of the waste dump and the unsuitable material stockpile, several changes occurred, mainly due to the properties of the stored materials. Initially the Pampa Verde waste dump was designed to store siliceous rock, but in practice, up to four different types of materials were stored: argillic, advanced argillic, massive silica and moderate silica.

Because of the above, Minera La Zanja requested Knight Piésold to redesign the Pampa Verde waste dump in order to implement the necessary measures to ensure its physical stability, which made necessary the determination of the materials properties through a geotechnical investigations. As a result of the redesign of the Pampa Verde waste dump, Knight Piésold developed a loading plan to conform the four types of the materials identified.

In order to increase the storage capacity, Minera La Zanja requested the design of a vertical expansion of the waste dump to raise its elevation approximately 15 to 20 meters. The overall approach for stabilization of the vertical expansion involves the design and construction of a reinforced soil slope in front of the existing reinforced toe buttress. The raising of the Pampa Verde waste dump will allow for an additional storage capacity of 0.94 million cubic meters, compared to its initial configuration.

2 AREA CHARACTERIZATION

2.1 *Local Geology*

At the local level, in the area of the La Zanja project outcrops mainly pyroclastic volcanic rocks and spills of the Lama formations of the Lower Tertiary and volcanic rocks of the Porculla formation of the Middle Tertiary. The rocks of the Huambo formation of the Upper Tertiary, appear to the northwest, outside the limits of the project area.

2.2 *Geotechnical Investigation*

2.2.1 *Background*

The first geotechnical investigation was carried out between August 16 and October 11, 2010, in order to determine the geotechnical characteristics of the foundation surfaces where the Pampa Verde waste dump and associated structures would be constructed. The field works consisted of 6 drilling holes, 46 test pits, tests with dynamic penetrometer of conical tip (DPL) and geological-geotechnical mapping. The second geotechnical investigation was carried out in June 2013 and consisted of 6 drilling holes, 9 test pits and 4 in-situ density tests were carried out by the water replacement method. The third geotechnical investigation was carried out between June 5 and 25, 2014, in order to characterize the materials that were being stored in the deposit. 2 drilling holes, 12 test pits (sampling for large-scale grainsize tests) and 8 in-situ density tests were carried out by the water replacement method, in the advanced argillic and argillic materials.

In November 2014, an additional geotechnical investigation was carried out, in order to characterize the advanced argillic and argillic materials that had been conformed. 2 test pits, 2 large scale grainsize tests and 7 in-situ density tests were developed by the water replacement method. Between April and June 2017, a geotechnical investigation was carried out with the specific objective of designing the raising of the Pampa Verde waste dump, having characterized the stored materials and the foundation at the site of the projected reinforced soil slope.

2.2.2 *Fieldworks*

The field geotechnical investigation consisted of the execution of 7 drilling holes and 23 test pits. The vertical geotechnical drilling reached variable depths between 15.0 m and 90.0 m, in which Standard Penetration Tests (SPT) and Large Penetration Test (LPT) were performed as well as in-situ permeability tests, Lefranc (in soils) and Lugeon (in rocks). In the test pits, which reached variable depths between 0.8 m and 5.6 m, 6 large scale grainsize tests and 6 in-situ density tests were carried out by the water replacement method; in addition, detailed records were taken of the stratigraphy of the materials found; in-situ tests and sampling of disturbed and undisturbed samples were carried out for the laboratory tests.

Georys Ingenieros S.A.C. (Georys) conducted geophysical prospecting tests consisting of 14 measures of surface waves in multichannel arrays by the Multichannel Analysis of Surface Waves (MASW) method, 9 readings by the Microtremor Array Measurement (MAM) method and 3 lines MASW 2D. In order to monitor the water level in the unsuitable material stockpile and the Pampa Verde waste dump, 6 Casagrande piezometers were installed.

2.2.3 *Laboratory Tests*

Laboratory tests were developed in order to determine the materials properties, including the existing rock in the foundation. In order to evaluate the potential for generating acid drainage, geochemical tests were carried out using the Sobek Modified Method (ABAM), in the laboratory of ALS Environmental Chemex (Peru). There were also carried out laboratory tests of the geogrid and geogrid vs. soil interface tests (ASTM D 5321) in the TRI Environmental Inc. (TRI), in Texas, USA, whose results are presented in Section 4.

2.3 *Design Earthquake*

There are three seismic hazard studies developed for the specific location of the La Zanja project, the last one of June 2017 prepared by ZER Geosystem Peru S.A.C. (ZER Geosystem), which included the characterization of the seismogenic sources near the site of study, the elaboration of the seismic model based on the Ground Motion Prediction Equation (GMPE), the evaluation of the seismic hazard through the probabilistic and deterministic methodologies, seismic disaggregation analysis and the generation of five synthetic accelerograms adjusted to the Uniform Hazard Spectrum of the site.

The results of the probabilistic seismic hazard are presented in Table 1 for a soil type 'B', accordingly to the International Building Code (IBC). To evaluate the physical stability of structures for the storage of mining waste, it is recommended to use as a design earthquake that corresponds to a return period of 1 in 100 years, during the operating period, a criterion that is accepted worldwide for the design of this type of structures. Accordingly to the "Environmental Guide for the Slope Stability of Solid Mine Waste Deposits" of the Ministry of Energy and Mines of Peru (MINEM), the seismic coefficient can vary from 1/2 to 2/3 of the peak horizontal acceleration of the soil, that is, from 0.07 to 0.10. For the purposes of the seismic design (pseudo-static analysis) of the Pampa Verde waste dump rising, 0.12 was used with conservative criteria, for operating conditions.

Table 1. Seismic accelerations for different return periods (Seismic Hazard Study 2017).

Return period (Years)	Maximum seismic ground acceleration [(1)] a (g)
100	0.143
250	0.205
500	0.262
1,000	0.331
2,500	0.435
5,000	0.530
10,000	0.637

1. Peak Ground Acceleration (PGA), considering rock terrain conditions for an average cut-off wave velocity equivalent to 760 m/s.

3 PAMPA VERDE WASTE DUMP VERTICAL EXTENSION

3.1 *Design Criteria*

The design criteria used have been proposed by Knight Piésold accordingly to international standards and national requirements for this type of structure, which were accepted by Minera La Zanja, as presented in Table 2.

Table 2. Design criteria.

Description	Value
Additional storage capacity	0.94 Mm^3 (maximum possible capacity)
Lift height	10 m
Slope lift	1.4H:1V
General slope of the waste dump pile	2.5H:1V
Peak ground acceleration (return period of 100 years)	0.23 g
Magnitude of the seismic event (return period of 100 years)	8.0
Seismic coefficient	0.12

Description		Value
Minimum safety factor - Waste dump (operating period)	Static condition	1.3
	Seismic condition	1.0
Minimum safety factor - Reinforced soil slope Global Analysis - External	Static condition	1.3
	Seismic condition	1.1
Minimum safety factor - Reinforced soil slope Slip analysis - Internal	Static condition	1.3 – 1.5
	Seismic condition	1.125

3.2 *Project constraints*

The project constraints are as follows:

1. Limited options for laterally extending the existing buttress to avoid encroaching on existing mining facilities;
2. The need to satisfy both static and seismic stability;
3. The need to properly select high-strength reinforcement products;
4. The importance of conducting material-specific tests to characterize the soil-geogrid interaction; and
5. The complex geometry of the overall system, particularly its global stability.

3.3 *Reinforced Soil Slope Design*

3.3.1 *General*

In order to increase the storage capacity of the Pampa Verde waste dump, it was proposed to vertically extend the containment dike by a soil reinforced slope with uniaxial geogrids. The crest of the reinforced soil slope will have 10.0 m width, an upstream slope of 2H:1V and will be supported on the existing containment dike, while the downstream slope will be 1H:1V. The total height of the reinforced slope will reach 41 m. At the foot of the reinforced soil slope, it was proposed to conform a reinforcement embankment with a slope of 2H:1V. The waste material inside the deposit will be formed with a general slope of 2.5H:1V, in 10 m height lifts, with 1.4H:1V slope, having to maintain berms of 11 m width between the waste material lifts. The additional volume to be stored will be 941 900 m^3.

3.3.2 *Slopes Geometry*

The section for the analysis has been considered one that runs longitudinally through the Pampa Verde waste dump, considered the most critical because it covers the largest amount of waste material and the steepest slope downstream of the existing dike (where the haul road passes). Likewise, the general slope of the projected reinforced slope has been considered. The locations of the section that was analyzed is shown in Figure 3.

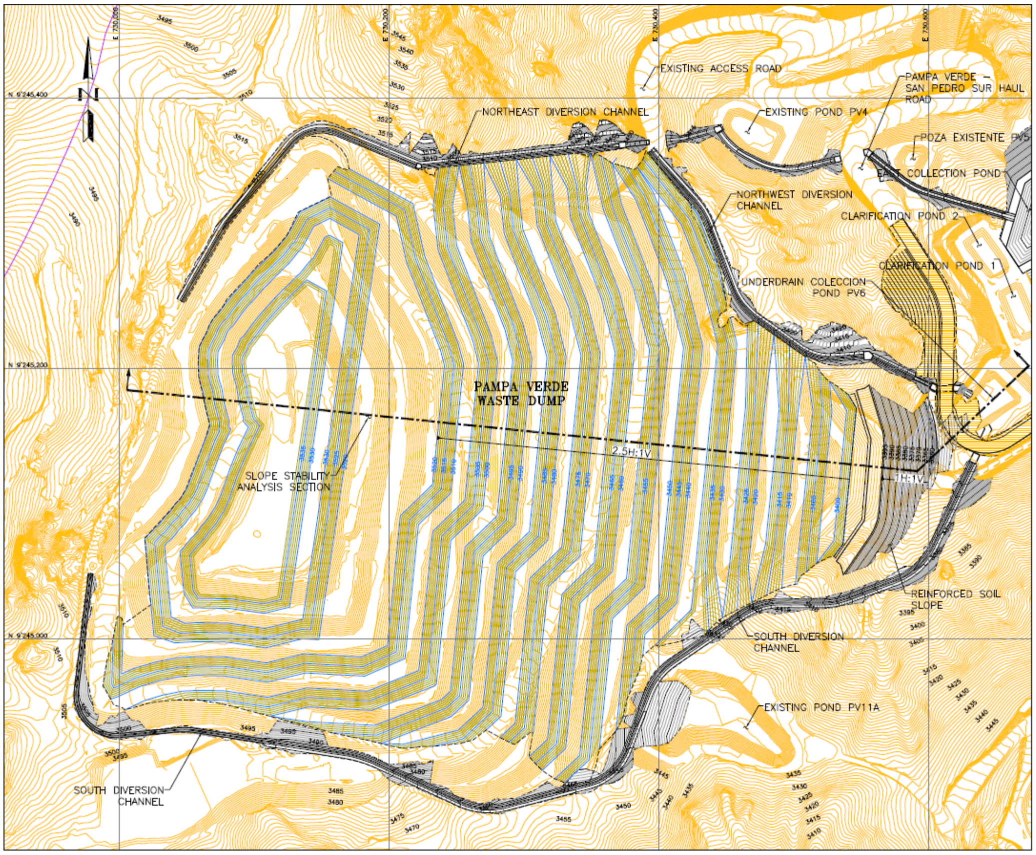


Figure 3. Section location for the slope stability analysis.

3.3.3 *Materials Properties*

For the geotechnical characterization of the materials involved in the slopes stability analysis, the results of the geotechnical investigation developed for the Pampa Verde mine waste design and also the results of the previous geotechnical investigations (2010, 2013 and 2014) were used. The properties of the different materials that intervene in the slopes stability analysis are presented in Table 3.

Table 3. Geotechnical properties of materials for the slope stability analysis.

Material type	Unit weight (kN/m^3)	Cohesion (kPa)	Friction angle (Degrees)	Undrained shear resistance (kPa)
Rocky basement (1)		Impenetrable material (bedrock)		
Residual soil (1)	21.0	50	29	-
Soil liner (1)	19.0	0	22	-
Anthropogenic fill (1)	19.0	0	33	-
Unsuitable material stockpile containment dike (1)	19.0	0	35	-
Unsuitable material – fine dense (2)	17.0	-	-	55
Unsuitable material stockpile containment dike without QA (1)	19.0	0	32	-
Unsuitable material – coarse medium dense (1)	18.0	0	30	-
Unsuitable material – coarse loose (mixed) (1)	18.0	0	28	-
Waste material – silica (1)	21.0	0	37	-

Material type	Unit weight (kN/m^3)	Cohesion (kPa)	Friction angle (Degrees)	Undrained shear resistance (kPa)
Waste material – argillic [(1)]	20.0	0	31	-
Waste material – advanced argillic [(1)]	20.0	0	30	-
Mixed waste material – silica and argillic [(1)]	19.0	0	35	-
Mixed waste material – advanced argillic, argillic and unsuitable [(1)]	20.0	0	30	-
Reinforced soil slope fill material [(1)]	20.0	0	36	-
Reinforced soil slope fill material (projected) [(1)]	20.5	0	35	-
Randomfill and transition material [(3)]	20.5	0	36	-
Gabions [(3)]	21.0	0	33	-
Mud [(3)]	15.0	-	-	9
Waste material (projected) [(1,3)]	21.0	0	33	-
Rock fill	21.0	0	40	-

1. Effective parameters obtained from field geotechnical investigation and laboratory tests, carried out in 2013, 2014 and 2017.
2. Effective parameters obtained from Standard Penetration Tests (SPT) and "back analysis".
3. Effective parameters based on the experience of Knight Piésold in similar materials.
4. The foundation of the waste consists mainly of residual soil and/or rock outcrops. The unsuitable materials stored in the deposit, which will be encapsulated by the waste material, have been considered as the weakest.

3.3.4 *Piezometric Level Conditions*

The piezometric level was defined based on the records of the piezometers installed in the 7 drilling holes of the geotechnical investigation carried out in June 2017. Two piezometric levels were considered:

– Variable depth between 26.4 and 36.8 m with respect to the existing ground level. This piezometric level appears due to the moisture of the material discharged and to the leaks that have occurred inside the existing waste material deposit.
– Depth variable between 38.0 and 75.2 m with respect to the existing ground level. This level is close to the foundation level of the deposit.

3.3.5 *Geogrids Pullout Analysis*

The computer program RESSA version 3.0, which belongs to the set of programs of Adama Engineering Inc., has been used. The program allows to develop the stability analysis considering the type of translational failure through the interaction between the geogrid and the soil. It has been used the results of the laboratory tests of the interface soil vs. uniaxial geogrid and geogrid performance. The design allowable tension of the uniaxial geogrids was 230 kN/m. The geogrids pullout analyzes were performed under static and earthquake conditions (pseudo-static analysis). The results of the geogrids pullout analyzes show a minimum static factor of safety of 1.51 and a minimum pseudo–estatic factor of safety of 1.16.

3.3.6 *Waste Dump Facility Stability Analysis*

The slope stability analyses associated with the vertical extension of the Pampa Verde waste dump were developed using the computer program SLOPE/W® version 7.23, for static and earthquake conditions (pseudo-static analysis). The following cases were analyzed:

– Global failure of the downstream slope. Failures through the body of the reinforced soil slope and the current and projected waste dump.

– Local failure downstream of the toe of the waste dump. Failures through the soil reinforced slope with uniaxial geogrids.
– Global failure of the upstream slope. Failures in the slopes of the existing and projected waste material.

The geotechnical model is shown in Figure 4 and the results of the slope stability analyzes of the Pampa Verde waste dump are presented in Table 4.

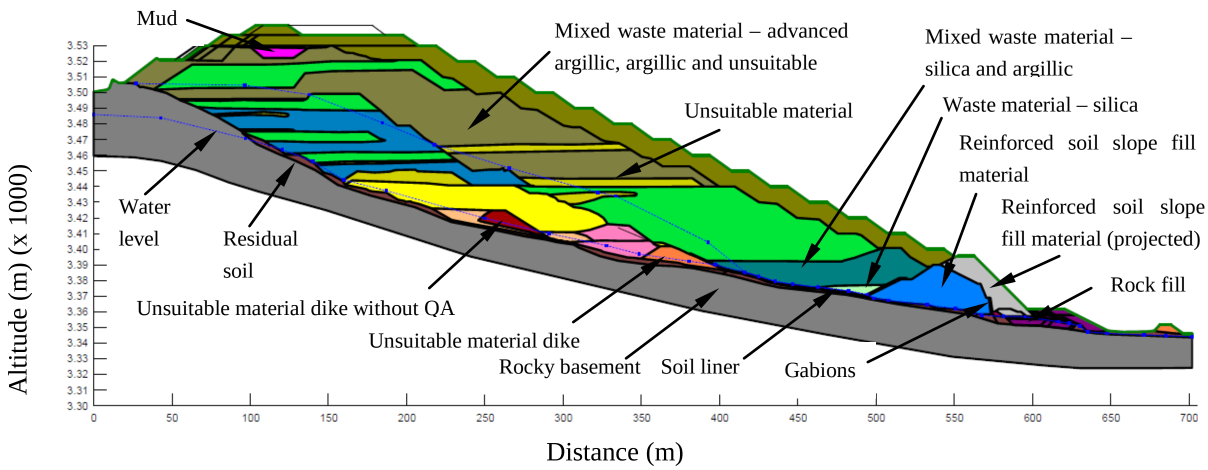


Figure 4. Section location for slope stability analysis.

Table 4. Results of the slope stability analyzes.

Failure type	Location	Static factor of safety	Pseudo–estatic factor of safety
Global	Downstream slope	1.34	1.00
	Upstream slope	1.46	1.07
Local	Downstream slope	1.39	1.12

4 UNIAXIAL GEOGRIDS EVALUATION

4.1 *Testing Methods and Criteria*

The obtained results would not be valid in the case there was not an appropriate interaction between the geogrids and the soil to be used in the construction of the geogrid-reinforced slope, so a proper selection of geogrids is particularly relevant in this project, mainly due to the following aspects:
– The structure is relatively high, which leads to the selection of geosynthetic products of high tensile strength.
– Direct shear is a relevant mode of failure for the configuration of this project. Consequently, interface shear strength between soil and geogrids should be properly characterized, not only for pullout evaluation, but also for wedge analyses.
– Because of the potential contact between the geogrid reinforcements and acidic fill materials, chemical degradation considerations are more relevant than for conventional retaining structures.

A proper evaluation of the soil vs. geogrid interface starts with the establishment of the testing conditions used for determination of the interface properties between uniaxial geogrids and the backfill material. The interface testing program was developed in the TRI Environmental Inc. (TRI) geotechnical and geosynthetics testing laboratory, located in Austin, Texas.

The conditions and characteristics of the soil direct shear and soil vs. geogrid interface shear tests conducted at TRI considered the following conditions:

- The need to conduct soil direct shear tests (in addition to the soil vs. geogrid interface shear tests) to properly define the soil vs. geogrid coefficient of interactions. This determination is significant to account for possible differences between the tested soil and backfill soil ultimately used during construction.
- The need to conduct tests under submerged conditions and comparatively small displacement rates (0.1 mm/min) to minimize the possible generation of pore water pressures. This approach was adopted after evaluating the effect of the displacement rate in the test results.
- The need to adopt test approach "B", under ASTM D 6637, of the soil vs. geogrid interface testing method, which was found to be more consistent with the approach used for determination of soil shear strength and better account for corrections (e.g. area correction) in the interpretation of test results.

4.2 *Geogrids Considered in the Testing Program*

A total of five geogrid products were considered for possible use in the construction of the geogrid-reinforced slope at the Pampa Verde project; for the purposes of this paper, we are going to refer to the geogrids as "Geogrid 1" to "Geogrid 5". The polymers used in the manufacturing process and key results from wide-width tensile tests (ASTM D 6637, Method B) are summarized in Table 5.

Table 5. Geogrids properties.

Geogrid type	Fibers type	Ultimate tensile strength (kN/m)	Ultimate elongation (%)	Unit tension (kN/m)	Tensile strain (%)	Secant stiffness (kN/m)
Geogrid 1	Polyester (PET)	389	9.4	180	5	3,600
Geogrid 2	Polyester (PET)	360	11.7	122	5	2,440
Geogrid 3	Polyester (PET)	302	12.8	77.1	5	1,542
Geogrid 4	Polyvinyl Alcohol (PVA)	364	5.44	312	5	6,240
Geogrid 5	Polyester (PET)	457	11.7	176	5	3,520

The preliminary design considered an admissible tensile strength (design tensile strength) of 230 kN/m, which was the basis for the identification of the five geogrid products. It should be noted that the allowable tensile strength is defined as the ultimate tensile strength penalized by a series of reduction factors (construction damage, degradation, creep); the reduction factors for each geogrid are different and established by certified documentation provided by the manufacturers. Table 6 summarizes the ultimate tensile strength as reported in tests conducted at TRI, the reduction factors and the predicted allowable tensile strength.

Table 6. Geogrids tensile strengths.

Geogrid type	Ultimate tensile strength (kN/m)	Reduction factor	Allowable tensile strength (kN/m)
Geogrid 1	389	1.76	221
Geogrid 2	360	1.57	229
Geogrid 3	302	2.73	111
Geogrid 4	364	1.49	244
Geogrid 5	457	1.68	268

As shown in Table 6, Geogrid 1 and Geogrid 2 led to an admissible tensile strength that is slightly below the 230 kN/m originally considered in the preliminary design. Geogrid 3 resulted in an admissible tensile strength that is significantly below 230 kN/m. Finally, Geogrid 4 and Geogrid 5 satisfied the admissible tensile strength considered in the preliminary design.

Two other considerations are also important for the Pampa Verde dike raising:

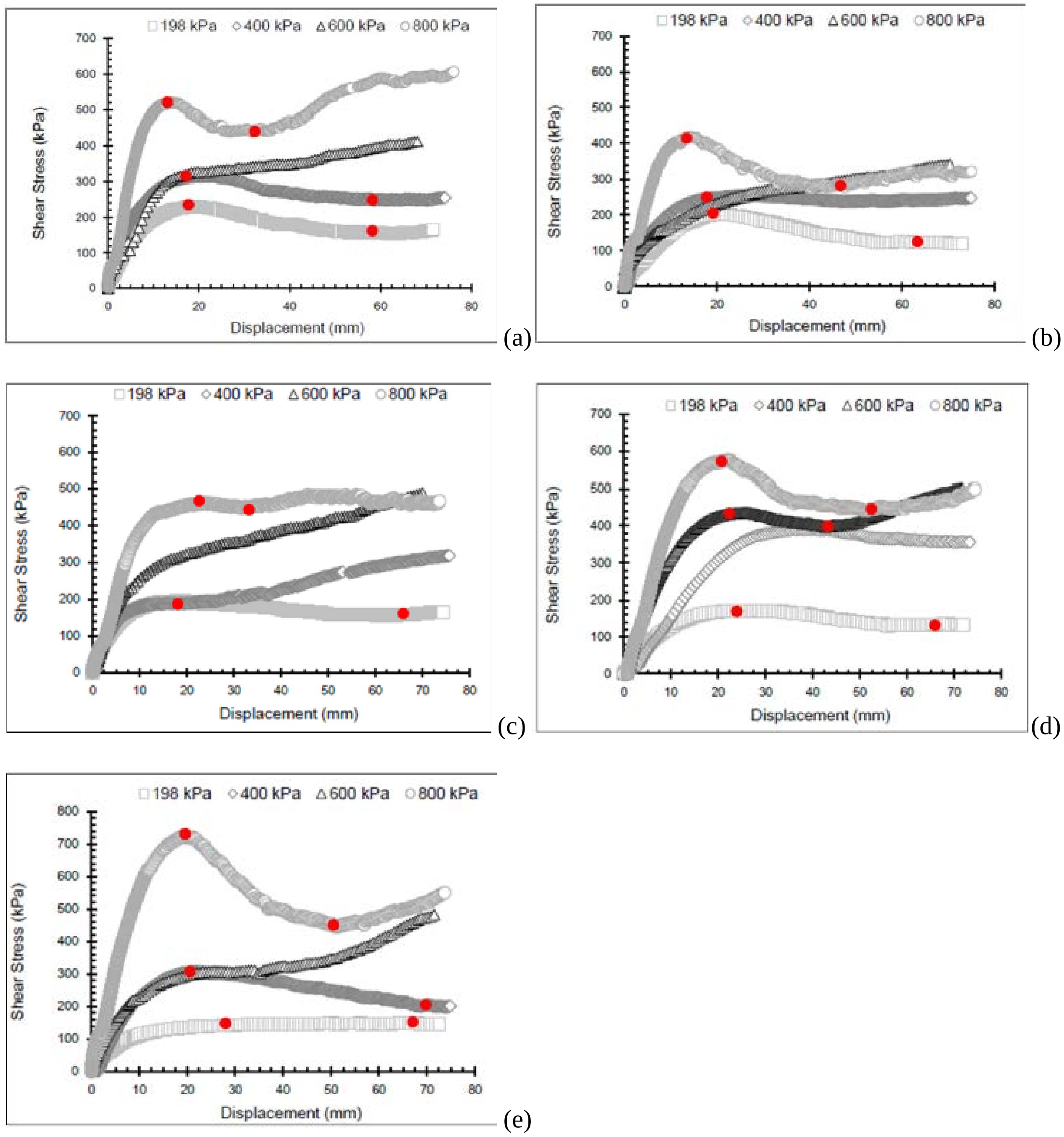
- Compatibility of soil and geogrid strains: As will be discussed in Section 4.4, the soil peak shear strength is found to occur at a shear strain of approximately 5%. Consequently, the unit tension at a strain of 5% will lead to enhanced performance. This is because although the tensile capacity of geogrids may continue to develop beyond a 5% strain, the soil shear strength would have already been achieved. Therefore, a relevant parameter to consider in the comparison of the different products is the secant stiffness at a tensile strain of 5%. With a secant stiffness of 6,240 kN/m, Geogrid 4 is the product that provides the best compatibility of displacements with the backfill soil. Geogrid 1 and Geogrid 5 provide a secant stiffness of about 3,500 kN/m. Finally, Geogrid 2 and Geogrid 3 provide a secant stiffness under 2,500 kN/m.
- Chemical resistance to acid soil environments: An important aspect to consider in the selection process, which is directly related to the raw polymeric material used in geogrid manufacturing, relates to the chemical resistance of the products. While polyester (PET) is susceptible to chemical degradation in basic environments (pH over 10) and acidic environments (pH below 2), polyvinyl alcohol (PVA) offers comparatively high chemical resistance in both highly basic and acid environments. While the actual borrow source of fill material may not be precisely defined, there is concern that the fill used in the geogrid-reinforced slope may possibly involve a comparatively acid environment. Consequently, polymeric materials such as PP, HDPE and PVA will provide better chemical resistance than PET. Among the geogrids considered in this project, the Geogrid 4 is the only product manufactured using a polymer that resists acid environments (PVA); all other products are manufactured using PET, as this material allows manufacturing of the high-strength geogrids required for this project. No PP or HDPE products have been identified that satisfy the tensile strength requirements for this project.

4.3 *Soil Direct Shear Testing*

Soil direct shear tests (ASTM D3080) were conducted using samples sieved to a maximum particle size of ¾”. Tests were conducted at four different confining pressures (198, 400, 600, and 800 kPa). Tests were conducted under submerged conditions (with container flooded one hour prior to shearing initiation). Conditioning of the soil specimen involved application of the normal stress for a period of 15 minutes before shearing. The shearing displacement rate was 0.1 mm/min, which was deemed adequate to minimize the development of pore water pressures. The bottom half of the direct shear box had dimensions of 457 x 305 mm and was sheared against a smaller stationary container (top half with dimensions of 305 x 305 mm). Consequently, no area correction was considered in the interpretation of the results. This setup is consistent with ASTM D5321 used for interface shear testing. Shear testing typically took approximately 13 hours because of the comparatively small shear displacement rate. The test conducted at a normal stress of 800 kPa required use of a smaller box (203 x 203 mm) to achieve the target normal stress.

4.4 *Soil vs. Geogrid Interface Shear Testing*

Figure 5 (a) to (e) shows the shear stress versus displacement results obtained for the four soil vs. geogrid interface shear tests conducted using each one of the tested geogrid (Geogrid 1 to Geogrid 5, respectively).



Figures 5 (a) to (e): Interface shear stress vs. displacement response using Geogrid 1 to Geogrid 5, respectively.

Determination of the peak and residual shear strength values required careful interpretation. Accordingly, only the results indicated with "red dots" were considered in the determination of the shear strength parameters. For Geogrid 1, the peak interface shear strength was characterized by an interface friction angle of 26.0 degrees and an adhesion intercept of 134 kPa. The interface residual shear strength was characterized by an interface friction angle of 24.7 degrees and a cohesion intercept of 89 kPa. The coefficient of interaction for the peak interface shear strength was characterized by a frictional coefficient of 0.84 and an adhesion coefficient of 11.75. In addition, the coefficient of interaction for the residual interface shear strength was characterized by a frictional coefficient of 0.99 and an adhesion coefficient of 44.68.

For Geogrid 2, the peak interface shear strength was characterized by an interface friction angle of 20.4 degrees and an adhesion intercept of 115 kPa. The interface residual shear strength was characterized by an interface friction angle of 14.9 degrees and a cohesion intercept of 70 kPa. The coefficient of interaction for the peak interface shear strength was characterized by a frictional coefficient of 0.64 and an adhesion coefficient of 10.11. In addition, the coefficient of interaction for the residual interface shear strength was characterized by a frictional coefficient of 0.57 and an adhesion coefficient of 35.05.

For Geogrid 3, the peak interface shear strength was characterized by an interface friction angle of 24.2 degrees and an adhesion intercept of 100 kPa. The interface residual shear strength was characterized by an interface friction angle of 24.5 degrees and a cohesion intercept of 77 kPa. The coefficient of interaction for the peak interface shear strength was characterized by a frictional coefficient of 0.78 and an adhesion coefficient of 8.80. In addition, the coefficient of interaction for the residual interface shear strength was characterized by a frictional coefficient of 0.98 and an adhesion coefficient of 38.79.

For Geogrid 4, the peak interface shear strength was characterized by an interface friction angle of 33.8 degrees and an adhesion intercept of 37 kPa. The interface residual shear strength was characterized by an interface friction angle of 28.6 degrees and a cohesion intercept of 33 kPa. The coefficient of interaction for the peak interface shear strength was characterized by a frictional coefficient of 1.16 and an adhesion coefficient of 3.23. In addition, the coefficient of interaction for the residual interface shear strength was characterized by a frictional coefficient of 1.18 and an adhesion coefficient of 16.96.

For Geogrid 5, results of the test conducted at a normal stress of 800 kPa were observed to affect the estimated friction angle significantly. Nonetheless, they were considered in the interpretation of the results. The peak interface shear strength was characterized by an interface friction angle of 41.0 degrees and an adhesion intercept of 0.00 kPa. However, obtaining this comparatively high interface friction angle was highly influenced by the 800 kPa test. The interface residual shear strength was characterized by an interface friction angle of 28.1 degrees and a cohesion intercept of 15.4 kPa. The coefficient of interaction for the peak interface shear strength was characterized by a frictional coefficient of 1.50 and an adhesion coefficient of 0.00. In addition, the coefficient of interaction for the residual interface shear strength was characterized by a frictional coefficient of 1.16 and an adhesion coefficient of 7.76.

Figure 6 summarizes the peak interface shear strength results for all five soil vs. geogrid interfaces; the peak interface shear strength envelopes for the Geogrid 4 and Geodrid 5 (as well as the soil shear strength envelope) are represented by somewhat thicker lines. Figure 7 summarizes the residual interface shear strength results for all five soil vs. geogrid interfaces; the residual interface shear strength envelopes for the Geogrid 4 and Geogrid 5 (as well as the soil shear strength envelope) are represented by somewhat thicker lines in this figure as well.

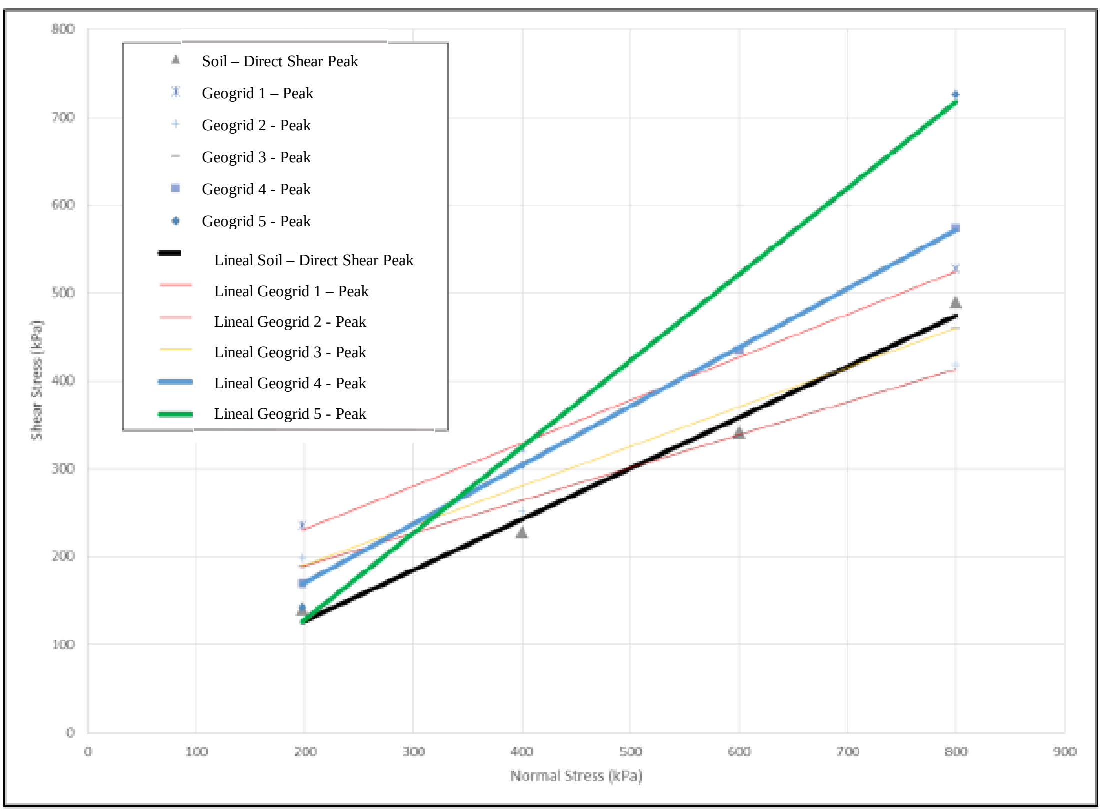


Figure 6. Summary of peak interface shear strength envelopes.

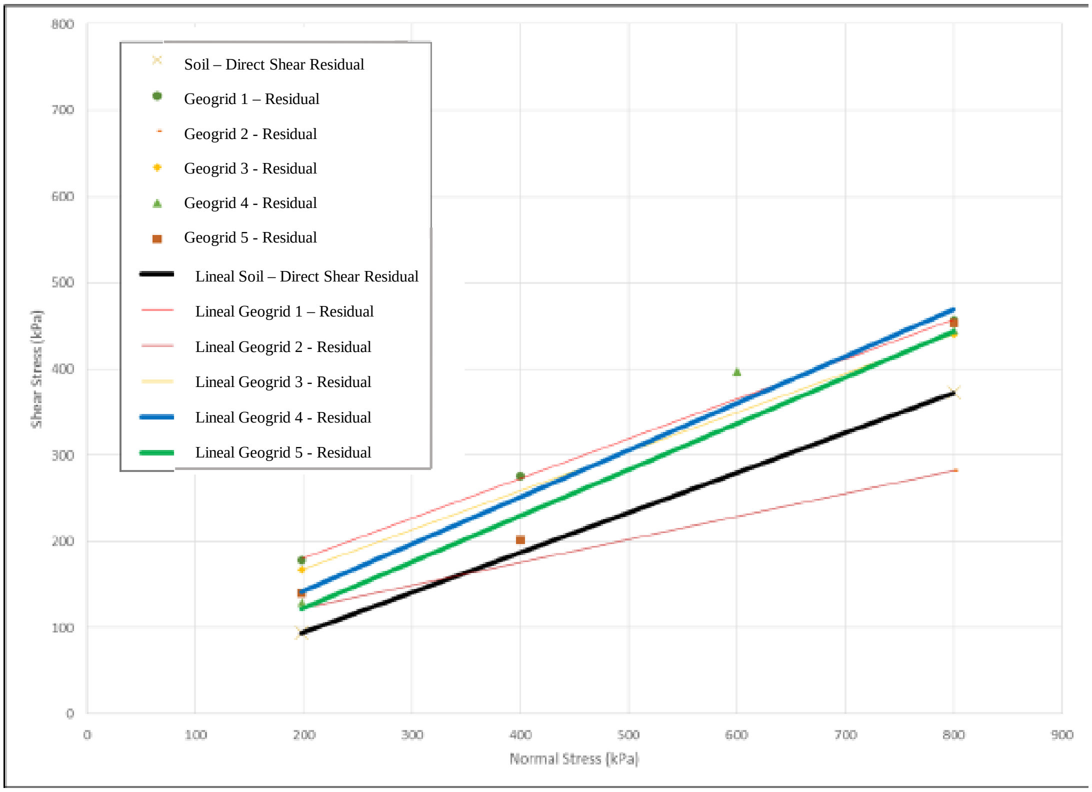


Figure 7. Summary of residual interface shear strength envelopes.

In the comparison of the different products, the most relevant parameters to assess, in relation to their interface shear strength characteristics, are the interface friction angle or the frictional coefficient of the interface shear strength. The highest interface coefficient for the peak interface shear strength was that of the Geogrid 5 (frictional coefficient of 1.50), followed by the interface coefficient for the Geogrid 4 (frictional coefficient of 1.16). In addition, the highest interface coefficient for the residual interface shear strength was that of the Geogrid 4 (frictional coefficient of 1.18), followed by the interface coefficient for the Geogrid 5 (frictional coefficient of 1.16). Overall, Geogrid 4 and Geogrid 5 were the products that provided the highest interface shear strength performance; Geogrid 1 was also observed to provide good interface shear response. In contrast, Geogrid 2 and Geogrid 3 produced comparatively low interface shear strength results.

4.5 *Selection of the Reinforcement Geogrid*

The proper selection of the geogrid to be used to mechanically stabilize the geogrid reinforced slope requires assessment of numerous factors. To objectively evaluate these various considerations, a value engineering approach, often used by the FHWA to assess the merits of different alternative retaining structures, was adopted. Specifically, the following factors were identified as relevant for the different geogrid products being considered as alternatives:
– Soil vs. geogrid interface shear strength properties.
– Deformation compatibility.
– Performance in acid environments.
– Documented manufacturing quality control and reduction factors.
– Anticipated quality of technical support during the final stages of design and installation.
– Tradition in the use of the geogrid product in geoenvironmental applications.

The tensile strength requirement was not adopted as a factor, as the allowable tensile strength was a minimum requirement for consideration of all five geogrid products. Additionally, cost was not considered among the factors for selection and consequently only technical considerations were weighed in this evaluation.

Because all factors do not carry the same relevance, weighted ratings (WR) ranging from 1 to 3 were assigned for each selection factor. Accordingly, a WR of 3 was assigned to interface properties, a WR of 2 was assigned to performance in acid environments and to manufacturing QC, and a WR of 1 was assigned to the remaining selection factors. The selection factors are shown in Table 8.

For each geogrid reinforcement considered, qualitative ratings (QR) ranging from 1 to 4 were subsequently assigned based on the merit of each geogrid for each selection factor. The selected QR values are also shown in Table 8. Finally, the weighted ratings were obtained by multiplying the WR by the QR, as summarized in Table 8. A final score for each geogrid alternative is also shown in Table 7.

Table 7. Geogrids evaluation matrix.

Evaluation criteria	Interface properties	Deformation compatibility	Performance in acid environments	QC Documentation	Technical support in project	Tradition in geoenvironmental applications	Weighted total score
Weighted level	3	1	2	2	1	1	
Geogrid 1	3	2	2	1	2	2	21
Geogrid 2	1	2	2	3	2	2	19
Geogrid 3	1	2	2	3	1	2	18
Geogrid 4	4	4	4	4	4	4	40
Geogrid 5	4	3	2	3	4	2	31

1. The individual score refers to the qualification of each supplier, where 1 means that the product is well below the evaluation criteria and 4 means that the product is adequate.
2. The weighting assigned to each criterion is based on the experience of Knight Piésold and the geotechnical design reviewer of the vertical expansion of the Pampa Verde waste dump.

As presented in this evaluation, Geogrid 4 was found to be the most appropriate geogrid alternative, with an aggregated score of 40. Geogrid 5 and Geogrid 1 were identified as somewhat distant second-rate alternatives, with aggregated scores of 31 and 21, respectively. Finally, Geogrid 2 and Geogrid 3 were identified as the least appropriate alternatives, with aggregated scores of 19 and 18, respectively.

It is recommended to select Geogrid 4 as geogrid reinforcement for the reinforced soil slope designed to stabilize the Pampa Verde waste dump. This selection is supported by the various considerations summarized in the value engineering approach documented in Table 7.

5 CONCLUSIONS AND RECOMMENDATIONS

The following are the conclusions and recommendations derived from the engineering of the vertical extension design of the Pampa Verde waste dump:

– The slope stability analyses indicate that the new configuration of the Pampa Verde waste dump will remain stable for static and earthquake conditions.
– Five different types of geogrids were evaluated. The results of the laboratory tests indicated that, accordingly to their mechanical properties (resistance and deformation), it was recommended to use the Geogrid 4, whose raw material is polyvinyl alcohol (PVA).
– The Senior geotechnical design reviewer of the vertical expansion of the Pampa Verde waste dump considered particularly robust the efforts involved in the geotechnical characterization and engineering evaluations conducted by Knight Piésold. Complementing such efforts with the selection of an appropriate geogrid product is expected to lead to a safe and well performing vertical expansion of the Pampa Verde waste dump.

– Perform the laboratory tests of the geogrid that will be used in the construction of the reinforced soil slope, in the event that a geogrid different from the recommended one is used, in order to review the slope stability analyzes and verify the design of the reinforcement.

6 REFERENCES

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