

Geotechnical characterization of a heterogeneous unsuitable stockpile

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ABSTRACT: Typically, stockpiled unsuitable soils from construction activities are classified as saturated soft fine soils. Nevertheless, because of some projects particularities, unsuitable materials could also present coarse soils resulting in a heterogeneous stockpile. This paper presents the geotechnical characterization of a heterogeneous stockpile in Peru in order to verify the physical stability of the disposal pile and to optimize the closure configuration. This characterization was based on a geotechnical investigation campaign that included boreholes, test pits, CPTU, MASW, field and laboratory tests. Based on the analysis of that information, strength parameters were determined for slope stability analysis. These parameters were represented by undrained shear strength lineal profile variable in depth for fine soils, and constant values of friction angle and cohesion for coarse soils. The stockpile stratigraphy was defined through the compatibility of geotechnical investigations results such as N_{SPT} blow counts, shear wave velocity (V_s), soil behavior type index (I_c), laboratory results, etc.

1 INTRODUCTION

During excavation activities for mining and civil structures construction, part of the remaining material is unsuitable, which means inappropriate for construction works and must be placed on stockpiles. Often, this material is supposed to be saturated fine soils. Nevertheless, due to the construction of different structures in diverse areas, various types of soils are found, which then conform the stockpile.

The unsuitable material stockpile studied is located in La Zanja mine, which is in the north of Peru at elevations between 2800 and 3600 m.a.s.l. La Zanja project is operated by Minera La Zanja S.R.L (MLZ) and consists on the exploitation of two open pits (San Pedro Sur and Pampa Verde), where silver and gold ore is mined and processed by heap leaching.

The need to re-assess the stability of this structure was mainly due to two reasons: in the original design, it was considered that the stored material would be saturated soft fine soils; so, the configuration of the stockpile included flat overall slopes (5H:1V). However, after the construction of the stockpile was completed, the final overall slopes were steeper in some sectors. The second reason was that MLZ wanted to evaluate the possibility to optimize the configuration of the stockpile for the closure stage.

The unsuitable material stockpile has 30 meters height and 6.9 Ha of area. The plan view of the stock-

pile is shown in Figure 1, while cross section A is shown in Figure 2.

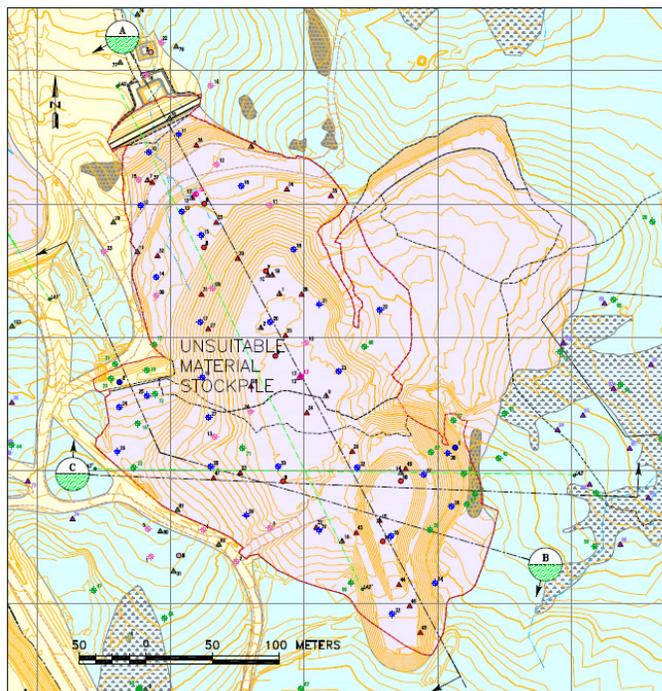


Figure 1. Plan view

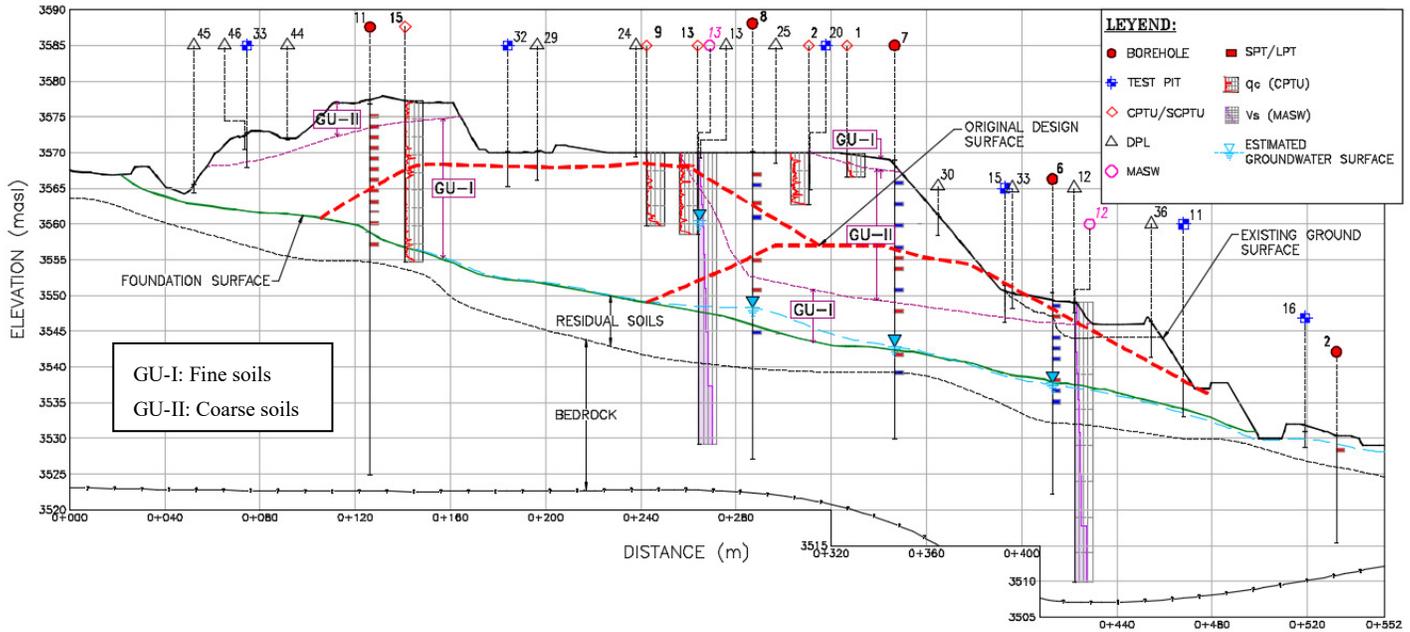


Figure 2. Cross section A

2. SUBJECT SITE

2.1 Geological characteristics

The unsuitable stockpile is compounded by anthropogenic deposits derived from the cut and fill materials from the earthworks. In the perimeter of the structure, quaternary deposits of the hydromorphic/colluvial/alluvial type were also found. In the same way, the stockpile foundation consisted on quaternary residual deposits or extremely to highly weathered bedrock. Beneath this layer, the highly resistant bedrock composed of porphyritic tuff of the Llama formation was located.

2.2 Configuration of the stockpile

As it has been previously mentioned, in the original design the overall slope to conform the stockpile was supposed to be flat (5H:1V). Nonetheless, the cut material from access roads and specific areas of the heap leach pad foundation came from residual soils or weak rock. Hence, it contained a proportion of gravels and cobbles, which increased the material's strength. For that reason, the overall slope of the unsuitable stockpile was steeper than the originally recommended.

In Table 1, the overall slope of the studied stockpile in different sectors is presented.

Table 1. Unsuitable stockpile slope angle in different sectors

Section	Slope from design (H:V)	Left of section		Right of section	
		Depth (m)	Slope (H:V) (°)	Depth (m)	Slope (H:V) (°)
A	5:1	11	2.5:1 22	27	4.5:1 14
B	5:1	21	2:1 30	11	5.5:1 11
C	5:1	23	2:1 30	12	7.5:1 8

3 SITE- SPECIFIC INVESTIGATIONS

3.1 Field investigations

A summary of the field investigation program conducted is presented in Table 2.

Table 2. Field investigations summary

Description	Quantity		In situ density	
	SPT/LPT	Quantity	Quantity	gr/cm ³
Boreholes	7	67	29	-
Test pits	29	-	7	1.32 – 1.68
DPL	24	-	-	-
CPTU	1	-	-	-
SCPTU	9	-	-	-
MASW	2	-	-	-
Piezometers	5	-	-	-

With the aim of obtaining referential values of the in-situ density of the different materials studied, tests were performed to obtain the in-situ density by water replacement method in some test pits.

On the other hand, 24 DPL were executed in the stockpile. The depth at which 30 blow counts were reached varied between 0.6 and 5.9 m, indicating the material compactness is not uniform.

3.1.1 CPTU and SCPTU soundings

Piezocone (CPTU) and seismic piezocone (SCPTU) tests were carried out at 10 positions within the subject stockpile, with the purpose of characterizing the fine soils and then establish its strength parameters. The depth of these tests varied from 3 to 27 meters. Dissipation tests of the pore pressure were also included to verify the static pore pressure conditions.

The SCPTU tests registered shear wave velocity (V_s) data at different depth intervals in order to measure soil stiffness.

Three principal parameters were measured:

- Cone resistance q_c , (MPa).
- Sleeve friction f_s , (MPa).
- Pore pressure u_2 , (MPa o m).

Figure 3 shows these three parameters registered in one SCPTU test:

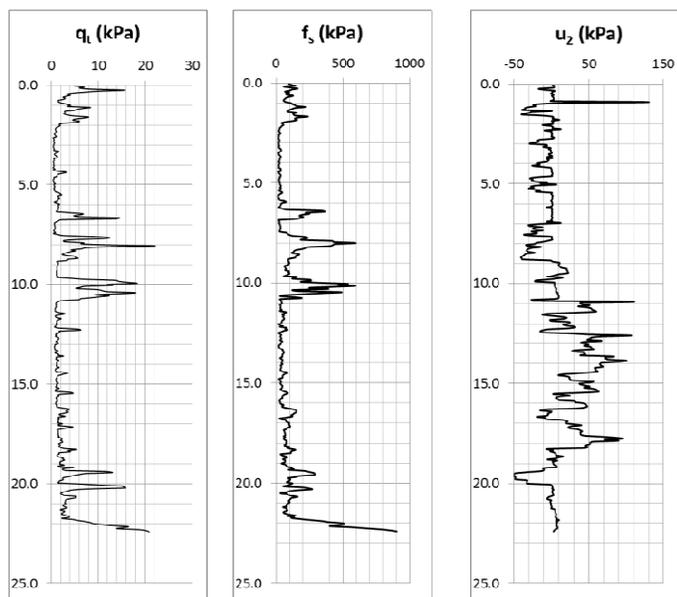


Figure 3. SCPTU profile

3.1.2 MASW soundings

Besides CPTU/SCPTU tests, a geophysical testing program was implemented and combined two MASW soundings with two geophysical lines of 440 and 360 m within the stockpile's footprint.

Figure 4 presents shear wave velocity (V_s) profiles of the two soundings. Moreover, V_s obtained from SCPTU tests and SPT blow counts from a near bore-hole are shown to compare the compactness/consistency of the material registered by each of the different tests performed. The results of shear

wave velocity obtained from SCPTU and MASW adjusted to the same level of compactness/consistency according to the International Building Code (IBC 2015).

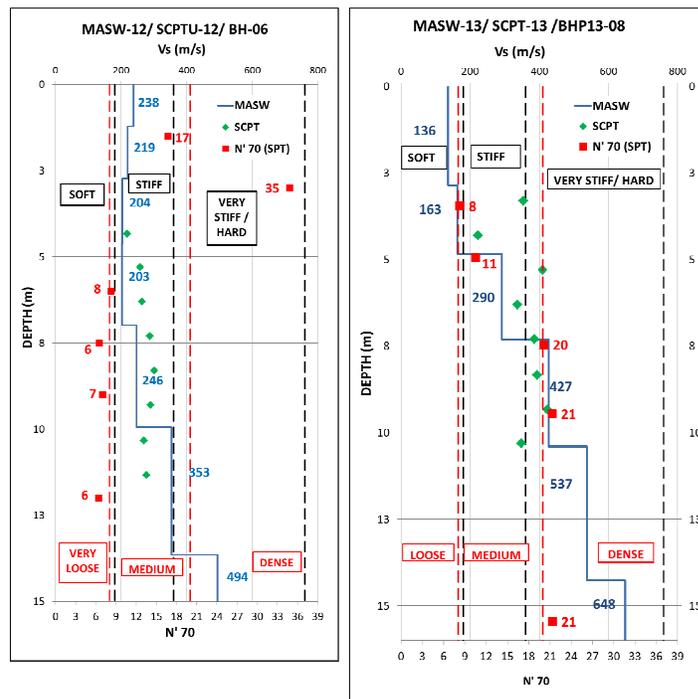


Figure 4. Shear wave velocity vs. N'_{70} (corrected SPT blow counts) profile

The intervals of compactness/consistency of soils for N'_{70} presented in Figure 4 were obtained from the empirical relations proposed by Bowles (Bowles 1996).

3.2 Laboratory testing

Concurrent with the geotechnical site investigations, the behavior of the different material types was also further evaluated by geotechnical laboratory testing. The results for fine and coarse materials are provided in Tables 3 and 4. In addition, grain size distribution curves of the most representative materials are illustrated in Figure 5.

Table 3. Laboratory results for coarse material

Coarse material	USCS	Minimum	Maximum	Average
Overall USCS classification	SM/GC/ SC/GM			
ω_0 (%)		3.1	46.7	15.7
LL		36	79	53
PI		16	36	25
Fines content (%)		15.5	49.3	35.5
γ_{nat} (g/cm ³)		1.73	2.10	1.96

Table 4. Laboratory results for fine material

Fine material	USCS	Minimum	Maximum	Average
Overall USCS classification	MH/CH/ ML/CL			
ω_0 (%)		6.8	55.4	31.3
LL		38	65	53
PI		16	35	24
Fines content (%)		51.9	85.5	64.3
γ_{nat} (g/cm ³)		1.61	2.10	1.88
UU triaxial:	MH			
Axial strain at failure (%)		10	Peak	
Friction angle (°)		0	5	
Cohesion (kPa)		13	40	

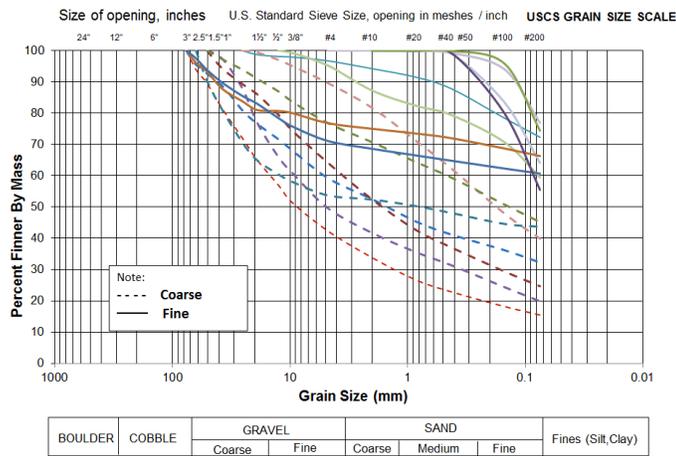


Figure 5. Grain size distribution of representative samples

Regarding the triaxial tests, UU tests were completed in undisturbed samples obtained from Shelby tubes and in remolded samples from test pits. In like manner, CU tests were completed in remolded samples obtained from test pits.

Based on the tests results, the high heterogeneity of the unsuitable material was verified. In general, there was a mixture of fine and coarse material, predominantly sands and fines.

4 GEOTECHNICAL CHARACTERIZATION AND STRENGTH PROPERTIES

After the review, processing and analysis of all the data obtained, two geotechnical units were determined according to their USCS classification and strength parameters: fine unsuitable material (GU-I) and coarse unsuitable material (GU-II). The distribution of the geotechnical units can be seen in Figure 2 for cross section A.

4.1 Fine unsuitable material (GU-I)

It was determined that the predominant stockpiled material was fine grained and classified as ML, MH, CL and CH, soft to very stiff consistency and wet; according to test pits and boreholes descriptions, and laboratory results.

Then, to establish the stratigraphy of the stockpile and materials shear strength parameters, data from CPTU/SCPTU tests were analyzed, which are ideal for characterizing fine grained soils.

Again, results from the evaluation of the three measured parameters in CPTU tests (q_c , f_s , u_2) with the analytical functions developed for them (Robertson & Cabal 2012, Robertson 2010a, b, Robertson 2009); confirmed the composition of the unsuitable material consisted mainly of fine grained soils with a percentage of sands and gravels. This is also stated by the profiles of the normalized soil behavior type index (I_c). Some examples of these profiles are shown in Figure 6.

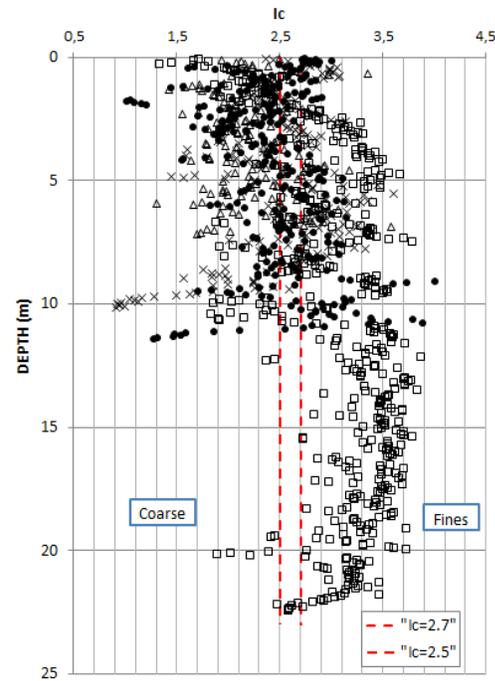


Figure 6. I_c profile

The stratigraphic profiles were established with the information from I_c profiles and grain size distribution tests. Then, considering the fact that perched water table conditions were identified in two CPTU tests and the stability analyses was going to be run for the short term; it was concluded that the undrained shear strength conditions resembled in a better way the actual conditions of the stockpile.

In agreement with the existing correlations in CPTU tests bibliography, the undrained shear strength (S_u) of fine materials was determined by using the following equation:

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}} \text{ (kPa)} \quad (1)$$

where: q_t = corrected cone resistance (kPa); σ_v = total vertical stress; and N_{kt} = cone factor ($N_{kt} = 10.5 + 7 \log(F_r)$). (Robertson 2013).

Whereas the cone factor N_{kt} in previous publications was defined as an empiric number in the range between 10 and 20, for this study the latest correlation developed by Robertson was used and the value was limited to a minimum of 14.

Figure 7 represents the undrained shear strength profiles (S_u) of fine materials estimated from CPTU and SCPTU tests:

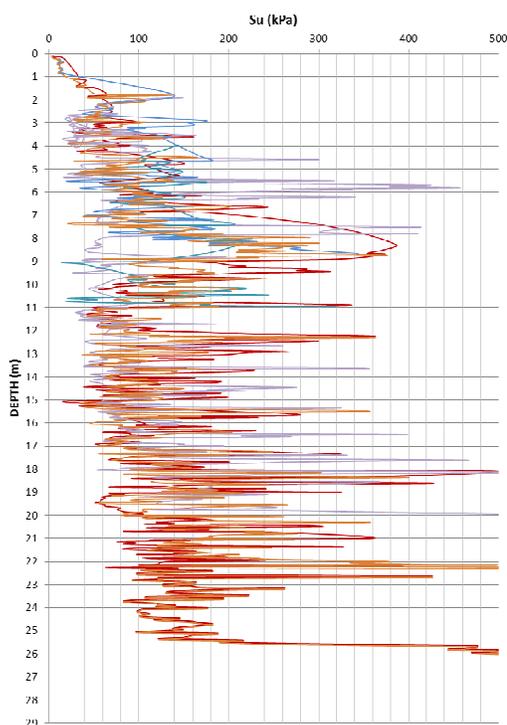


Figure 7. Undrained shear strength profiles

After obtaining the strength profile of each CPTU/SCPTU test, they were jointly processed and finally, it was decided to utilize the average values of S_u at different depths, without considering any possibly erratic or erroneous data.

Undrained shear strength estimated based on the average values of CPTU tests were more suitable for these fine grained material. Even though, these soils were predominantly cohesive, they also contained an important proportion of sands and less gravel.

Furthermore, average strength values were more representative because they complied with the back analyses done in the critical sections of the deposit, as there were not observable unstable conditions detected during neither the execution of the geotechnical investigation campaign nor latter visits to the site.

With the objective of characterizing the strength variation of fine material through the deposit, results from the CPTU data were fitted in several regressions, S_u vs. depth. Finally, based on the back analysis performed for the stockpile, the best fitting appeared as a linear regression, which is supplied in equation 2 and in Figure 8:

$$y = 4.81x + 21.3 \text{ (kPa)} \quad (2)$$

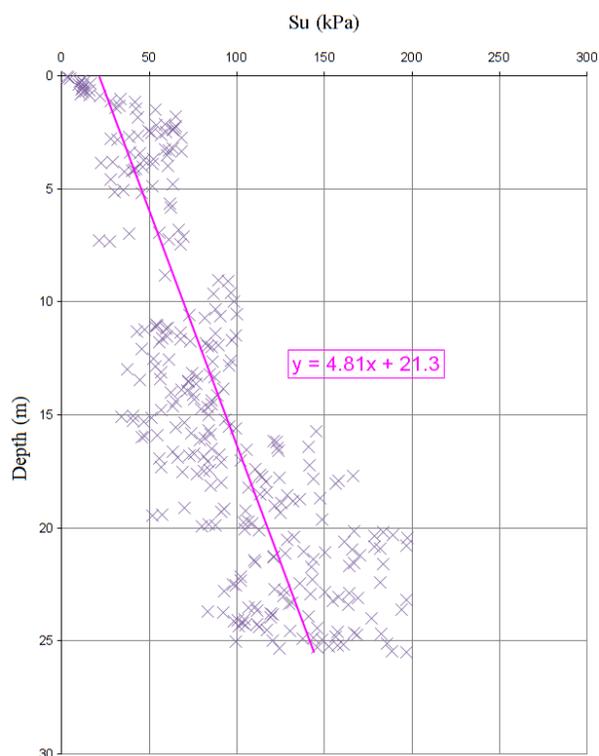


Figure 8. Final undrained shear strength profile for stability purpose

It can be observed that the lowest undrained shear strength value is 21 kPa. This value is within the range of the results of UU triaxial tests completed in this type of material.

4.2 Coarse unsuitable material (GU-II)

With regard to the second geotechnical unit or coarse materials, existing correlations between the SPT or LPT tests and friction angle in the soils mechanics literature were used. The analytical functions applied were from: Shioi & Fukui (1982), Kulwaha & Mayne (1990) and Hatanaka & Uchida (1996). Likewise, CPTU own formulations were used to estimate the ef-

fective friction angle for coarse materials by Jefferies & Been (2006) and Robertson (2009).

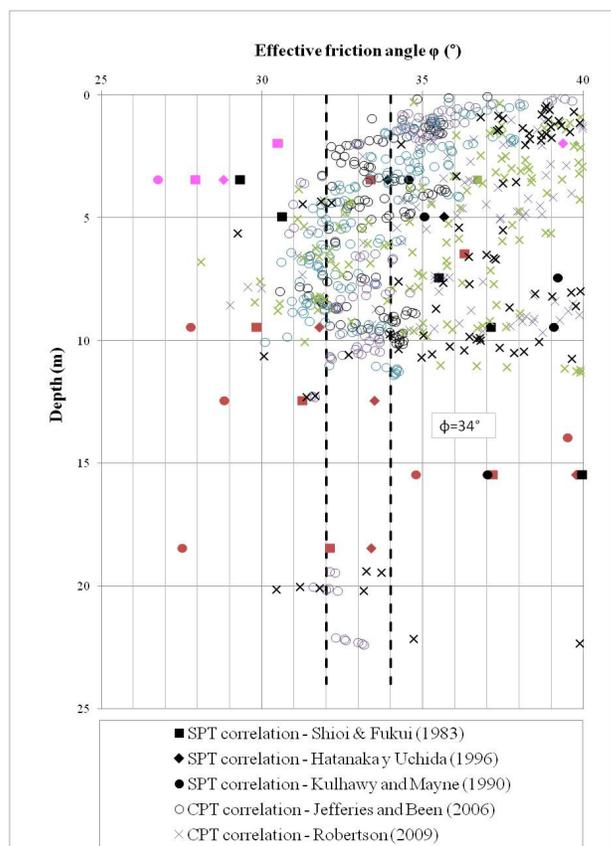


Figure 9. Effective friction angle for coarse material

In order to determine the range of values for the friction angle, back analysis of the existing conditions of the stockpile were performed. The range obtained, between 32° and 34°, was confirmed with the results of correlations with CPTU and SPT tests data.

At the end, these values were validated because they were similar to the steepest angle of repose slope found in the unsuitable stockpile.

Later, in the stability analysis it was determined that almost all the configuration of the stockpile was stable and a new configuration was proposed in the unstable sector. Also, for final closure stage, it was determined that not much earthwork was needed, influencing in the economics aspects of the project.

5 CONCLUSIONS

The case study presented has the distinction of having allowed correlating different kinds of geotechnical investigations. First, in spite of the heterogeneity of the stockpiled material, CPTU tests were successful in defining more accurate stratigraphy profiles and obtaining strength parameters for fine and coarse soils. If

coarse soils are predominately sands, CPTU tests can be used in the project.

Moreover, one important parameter was I_c , which showed the heterogeneity of materials placed in the stockpile and was confirmed by the boreholes performed.

On the other hand, CPTU tests are useful to find perched water table conditions unlike the piezometers. As a result, drained and undrained behavior of different strata can be identified.

Regarding shear wave velocity results from MASW and SCPTU tests, it was observed that the values obtained from both tests adjusted to the same level of compactness/consistency. In addition, the level of compactness or consistency found with SPT blow counts was lower than the one obtained with shear wave velocity values.

Finally, because of the amount of data provided by the CPTU tests, statistical analysis could be run to get even more precise strength parameters for the different strata.

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