Bearing capacity of a mine waste dump more than 100 m thick

Capacité portante d'un remblai de mine 100 m haut

Cañizal, J., Sagaseta, C. and Da Costa, A.
Universidad de Cantabria. Santander, Spain

ABSTRACT

A new industrial park is being developed on an area previously used as a mine dump associated to the Reocín mine in Cantabria, Spain, which has been recently closed. Total area is about 450000 m2. The fill is up to 140 m thick, and was placed during the last 25 years, only compacted by circulation of construction equipment and self-weight.

The suitability of the area for building includes a forecast of the self-weight time delayed settlements, together with the determination of the soil compressibility under the new loads. This has been done by conventional geotechnical exploration, including boreholes and geophysical methods, and monitoring the settlements during several months.

The performance of the first industrial plants built on the area is checked, and compared with the predictions.

RÉSUMÉ

Un nouveau parc industriel est en cours de développement, sur un remblai de mine, avec un épaisseur de plus de 100 m. Le remblai a été construit pendant les derniers 25 ans, en couches de 10 m, compactées seulement avec la circulation des camions.

Pou l'aménagement du site, on a besoin de prévoir les tassements de consolidation du remblai, ainsi que la déformabilité sous le nouvelles charges. On présente les résultats des reconnaissances géotechniques et géophysiques, et les comparaisons des prédictions avec le comportement des fondations.

Keywords : mine dumps, self-weight consolidation, backfill

1 INTRODUCTION

Open pit mine give rise to large volumes of excavated ground, usually disposed in landfills of large extension and thickness. After mine closure of the mine, there is an interest to recover these landfills for alternative use, either as recreation areas or as foundation of light or medium industrial facilities.

The zinc mine of Reocín has been in operation for the last 200 years in the outskirts of Torrelavega, a city with 40000 inhabitants in Cantabria, Northern Spain. The mine included a large open pit (as well as an extensive underground network), and it has been closed in 2004.

The main landfill, more than 100 m thick, was risen in the period 1978-1993. The top surface, of triangular shape (Figure 1), has an area of 450000 m^2, and elevation +200 to +230, and it will be used for industrial development (the boundary of the open pit can be seen in the upper part of Figure 1).

2 LANDFILL CONFIGURATION

The total landfill volume is 25-30 millions of cubic meters, coming from the nearby open pit, excavated in Cretaceous materials. The material has been placed in 10 m layers, with the only compaction produced by circulation of trucks and construction machinery. Landfilling was performed in 10 m thick layers, dumping the material and levelling it with bulldozer with a minimal compaction. The natural side slope of each layer is about 35º, and a 15 m berm was left every two layers, leading to an average slope of 25º.

The North and East slopes are 120 m high, and the West, 20 m. The South border leans against the natural hillside.

The landfill material is mainly dolomitic limestone (80%), with minor parts of sandstone and shales. This leads to a rockfill of angular blocks, with a relatively small clayey fraction.

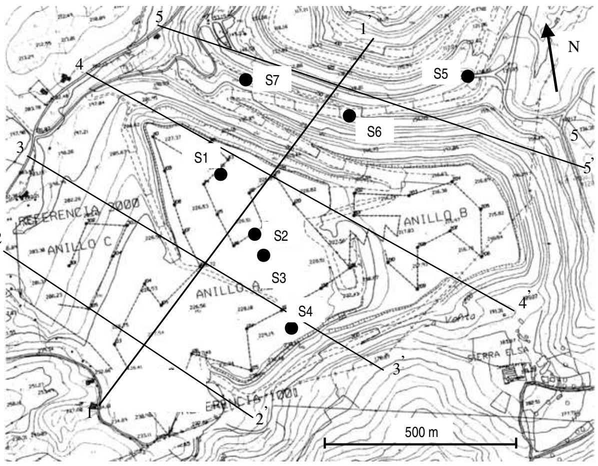

Figure 1. Plan view of the landfill

In the upper layer (from 1987), ultra-cycloned tailings (silty sands) were deposited in random locations, representing less than 10% in volume. This material was considered unsuitable, and it was eliminated in zones A and B by excavation, selection and replacement of the upper 10 m of fill.

Figure 2 shows the longitudinal (1-1) and transverse (4-4) profiles (see Figure 1) of the landfill.

3 SITE INVESTIGATION

The huge thickness of the landfill precluded the use of deep foundations to the natural bedrock. Hence, only direct foundations on the landfill were foreseen, possibly with the aid of improvement or replacement of the upper layers. The main

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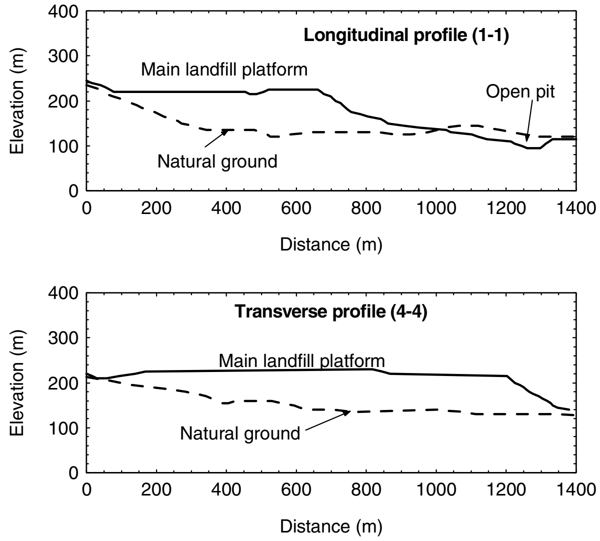

Figure 2. Longitudinal and transverse profiles

Table 1. Friction angle of mine dumps (Ayala and Rodríguez Ortiz, 1986).

$$\phi = \alpha(M + \phi_1 + \phi_2 + \phi_3 + \phi_4)$$

Nature	M	Siliceous	36º
		Carbonate	34º
		Schists	32º
		Clayey	30º
Density	ϕ_1	Loose	-5º
		Medium	0
		Dense	+5º
Particle shape	ϕ_2	Angular	+2º
		Medium	0
		Plates	-1º
		Rounded	-2º
		Very rounded	-3º
Particle size	ϕ_3	Sand	0
		Fine gravel	+1º
		Coarse gravel	+2º
		Boulders	+3º
Gradation	ϕ_4	Uniform	-3º
		Medium	0
		Well graded	+3º
Stress level (height of fill)	α	Low (H<20 m)	1.1
		Medium (20<H<40 m)	1.0
		High (H>40 m)	0.9

goals of the site investigation were the bearing capacity of the landfill, i.e., its deformability under the applied surface loads, and the possible delayed settlements due to its self-weight.

Seven exploration borings were drilled (Figure 1), four of them in the upper platform and three in the North slope.

The nature of the ground made the laboratory tests useless. In situ tests would be also of limited use, due to the presence of boulders larger than 200 mm, which would not allow penetration tests. Also, pressuremeter tests would give random results associated to irregular contact between the probe and the borehole walls. So, only geophysical methods were thought to give a reasonable insight into the global stiffness of the material. The following methods were used:

Seismic: 5 surface refraction profiles, four of them in the landfill and one in natural ground, 2 cross-hole and 4 down-hole tests, on the borings S1 to S4 in the upper platform (Figure 1).

Electrical resistivity: 9 vertical electrical soundings, 1 profile of electrical tomography.

Gravimetric: 286 readings of gravimetric anomalies.

For the delayed deformation due to self-weight, the only reliable approach is the recording of settlements during a representative time. Leach and Goodger (1991) indicate a minimum period of 6 months, for a prediction of settlements along 50 years. In the present case, 41 reference marks were installed in the upper platform, 26 in zone A, 10 in zone B and 5 in zone C (Figure 3). They were controlled monthly, with a precision of ±1 mm, from August 2003 to March 2004.

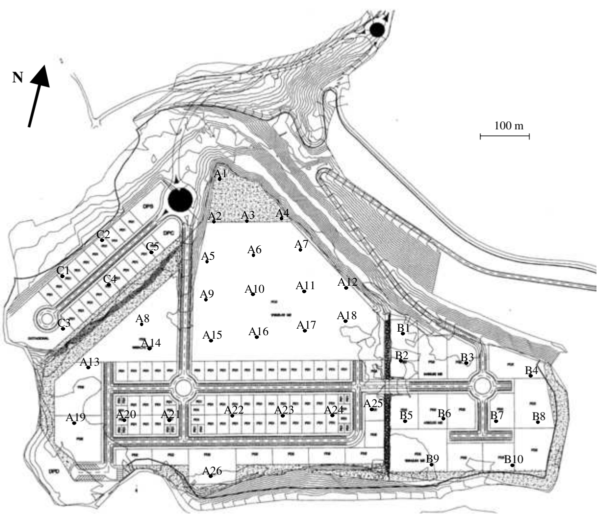

Figure 3. Reference points for control of settlements

4 GEOTECHNICAL PROPERTIES

The material recovered in the borings indicates 80% of dolomitic limestone, in agreement with the information from the landfill construction. The boundary slopes give a reasonable estimation of the fill friction angle of about 35º. This coincides with the existing recommendations for mine dumps, considering the identification data (Table 1). A small cohesion, of the order of 10 kpA, can develop with time, due to degradation of shale fraction.

The seismic refraction profiles indicated a shallow layer, 10 m thick, with a velocity of P-waves of 500-700 m/s, increasing to 900-1000 m/s.

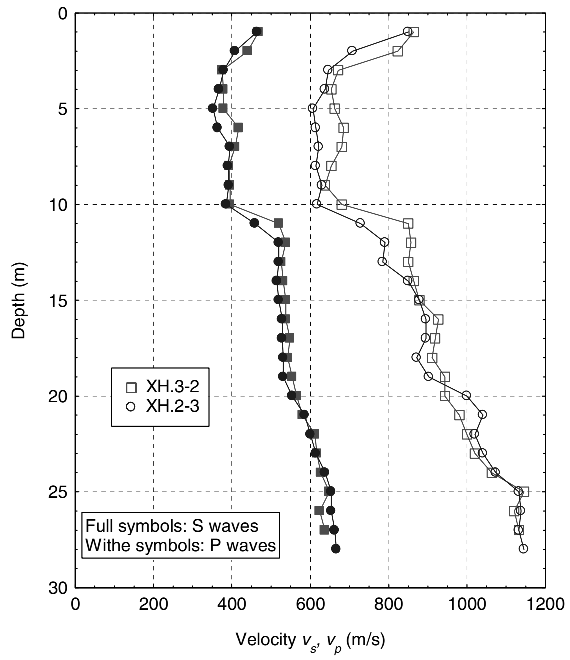

Figure 4. Cross-hole tests results between S2 and S3

The cross-hole analyses between borings S2 and S3 are shown in Figure 4. The 10 m layers are clearly visible. The shear wave velocity, v_s, is 400 m/s in the shallow layer, with an abrupt increase to 550 m/s at a depth of 10 m. At 20 m there is a less marked increase to 600-650 m/s. The same pattern is clear for the P waves velocity. The ratio v_s/v_p indicates a Poisson ratio in the range 0.18-0.26. The results from down-hole tests agree well with the cross-hole arrangement.

From the above values, the dynamic (small strains) Young's modulus was taken as 600 MPa for the first layer (0-10 m), 1200 MPa for 10-20 m, and 2000 MPa for 20-30 m. An average value of 0.23 was considered for the Poisson's ratio.

The above moduli were multiplied by a reduction factor to consider the influence of the stress and strain level. A factor of 0.08 was taken, based on the experience for heavily fractured rock masses (Hendron, 1968). The above values for the first three layers were reduced to 50, 100 and 160 MPa.

5 PREDICTION OF SELF-WEIGHT DELAYED COMPRESSION

The measured settlements during the 6 months control period were in the range 10-20 mm. The final values measured in all the points are presented in Table 2. The highest values correspond to areas with maximum thickness of fill (near the landfill front slope, at the Northeast boundary), whilst very low settlements are recorded in areas subjected to recent unloading. This is the case for points C3, C4, C5 and A1, located along a strip with inclined surface, which had been excavated to a depth greater than 10 m immediately before the control period. These points were excluded from the analysis.

Table 2. Settlements, s recorded at Δt=210 days

Point	*s (mm)*	*Point*	*s (mm)*	*Point*	*s (mm)*
A1	1.8	A15	8.0	B3	17.2
A2	5.2	A16	10.7	B4	18.9
A3	8.7	A17	9.9	B5	11.0
A4	11.5	A18	12.5	B6	13.2
A5	8.6	A19	4.7	B7	15.9
A6	11.5	A20	3.4	B8	18.8
A7	11.1	A21	5.2	B9	11.8
A8	5.2	A22	8.1	B10	15.5
A9	10.1	A23	7.9	C1	9.6
A10	8.2	A24	9.2	C2	12.6
A11	11.2	A25	4.5	C3	0.9
A12	14.6	A26	7.4	C4	1.1
A13	2.4	B1	14.1	C5	1.5
A14	7.2	B2	12.6		

The landfill height had been increased at a roughly constant rate along a period of 12 years, starting by filling local depressions at the lower areas. It was completed 10 years before the measurements.

Some laws have been proposed for the development of secondary compression of fills with time. Sanchez et al. (1993) have compared them in relatively recent sanitary landfills and found that in most cases an exponential law reflects reasonably the evolution of settlements:

$$\frac{\Delta H}{H} = \left(\frac{\Delta H}{H}\right)_f \left(1 - e^{-at}\right) \tag{1}$$

where f refers to the final, asymptotic strain, and the parameter a controls the settlement rate.

For old landfills, Leach and Goodger (1991) suggest a logarithmic law:

$$\frac{\Delta H}{H} = \alpha \log_{10}\left(\frac{t_0 + \Delta t}{t_0}\right) = \alpha \log_{10}\left(1 + \frac{\Delta t}{t_0}\right) \tag{2}$$

where α is a secondary compression coefficient; for mine dumps, the authors propose a range 0.5%-1.0%.

The time t_0 is usually taken as the average age of the landfill at the start of the control period. In most cases, it is not an easy task to find an appropriate value for it, particularly in high landfills, with a very long construction period. Unfortunately, the application of eq. (2) is very sensitive to the value of t_0, which acts as a time scale.

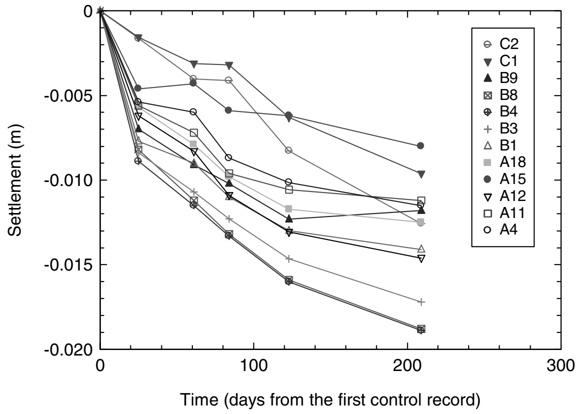

Figure 5. Settlement records in the upper platform (zones A, B and C)

In Figure 5 the settlement records for some representative points are plotted. In zones A and B, there is a clear decrease of settlement rate with time, by one order of magnitude along the control period. This is in contradiction with the reported age of the landfill (more than 10 years), and it reflects the influence of recent earthworks in the upper platform, cited in Section 2. So, the settlements between the first and second readings include a significant component of immediate settlement, and they must be suppressed from the analysis.

Nevertheless, the prediction based on the full recorded settlements was kept and considered as a safe upper bound for the future settlements. The secondary compression coefficient α (eq. 2) was then determined for each of the points, under two alternative assumptions:

- Considering all the readings and ignoring the above comment (assumption A).
- Neglecting the settlements during the first month of the control period, considering the second reading as the initial one (assumption B).

The resulting values for assumption A are:

- Area A: 0.28% to 0.99% (mean 0.60%)
- Area B: 0.71% to 1.36% (mean 1.05%)
- Area C: 0.97% to 1.29% (mean 1.13%)

The mean value was 0.69%. This agrees well with the range proposed by Leach and Goodger (1991), commented above.

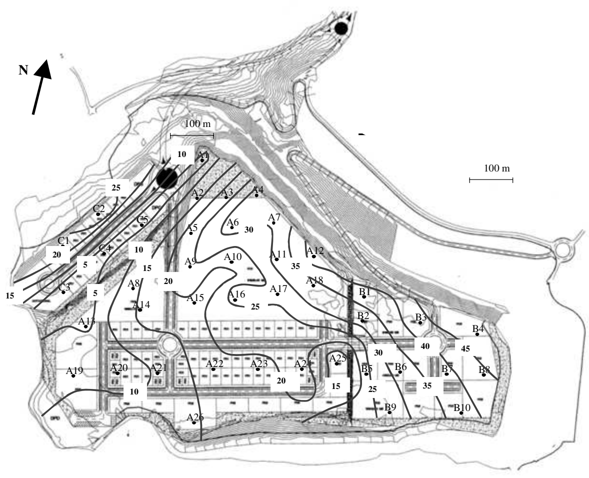

Figure 6. Predicted settlement (cm) for t=25 years (assumption A)

The future settlements at each point during the design life of the development were predicted using the local value of the secondary compression coefficient obtained at the same point. The settlements predicted under assumption A for a time of 25 years are shown in Figure 6. The maximum settlement is 500 mm, and the inclination (angular distortion), 1/800.

If the settlements during the first month of the control period are neglected (assumption B), the obtained values of α are:

- Area A: -0.22% to 0.50% (mean 0.26%)
- Area B: 0.31% to 0.75% (mean 0.55%)
- Area C: 0.92% to 1.27% (mean 1.10%)

The mean for α is 0.38%. With these values, the predicted settlements are considerably reduced (40-60%) with respect to the ones shown in Figure 6.

6 DESIGN PROVISIONS

The above values of the settlements were considered small enough for the development of the area, taking into account that assumption A was clearly on the safe side. Besides that, the strength and stiffness of the landfill (see Section 4) were acceptable for foundation. Nevertheless, the following design provisions were adopted as additional safety measures:

- Ensure a good quality for the first layer of ground, with some improvement if necessary (direct replacement, heavy tamping or preloading were considered adequate for this purpose).
- Use strip footings or mat foundations, in order to bridge possible local weak zones.
- Leave a safeguard strip (20 m wide) from the crest of the front slope.

7 OBSERVED LONG TERM SETTLEMENTS

In 2006, after finishing the construction of the area road network, and just before starting the first buildings, it was possible to make an additional survey of all the reference points (fortunately, none of them had been affected by the roads construction). The elapsed time at that moment had been 902 days after the starting of the six-months control period.

Table 3 shows the measured settlements. The maximum value is now 43.9 mm, again in point B4, located near the front slope. Points C3, C4, C5 and A1 keep showing small movements.

These settlements were substantially less than predicted under assumption A, as it was expected for the above commented influence of recent surface works. With the predictions under assumption B, i.e., neglecting the first reading of the six-months control period, the agreement was considerably improved. Figure 7 shows the comparison between predicted and measured settlement at each point at 902 days. In average, the settlements were still over predicted by about 20%, which can be considered a good approximation.

At the time of preparing this paper, the area has been developed with a good number of medium size industrial facilities, with only light ground improvement. The overall behaviour has been good, and no special geotechnical problems have been reported.

8 CONCLUSIONS

It has been shown that in landfills, the geotechnical investigations must be of special nature. Gathering information about the filling process is the main issue. Geophysical methods are particularly useful, because they tend to reflect the behaviour of the material at a large scale.

It is extremely important to undertake as soon as possible a campaign of surveying the landfill platform, in order to have a long enough period of records of self-weight settlements. The usual recommendation of six months is a minimum requirement. Also, it is important to ensure that no recent works of excavation or landfilling has taken place recently.

Table 3. Settlements, *s* recorded at Δt=902 days

Point	*s (mm)*	*Point*	*s (mm)*	*Point*	*s (mm)*
A1	0.8	A15	14.0	B3	40.2
A2	6.2	A16	21.7	B4	43.9
A3	5.7	A17	17.9	B5	34.0
A4	16.5	A18	24.5	B6	24.2
A5	15.6	A19	24.7	B7	30.9
A6	22.5	A20	15.4	B8	27.8
A7	24.1	A21	12.2	B9	17.8
A8	16.2	A22	10.1	B10	22.5
A9	16.1	A23	6.9	C1	25.6
A10	17.2	A24	7.2	C2	33.6
A11	23.2	A25	1.5	C3	4.9
A12	32.6	A26	5.4	C4	4.1
A13	19.4	B1	26.1	C5	1.6
A14	21.2	B2	28.6		

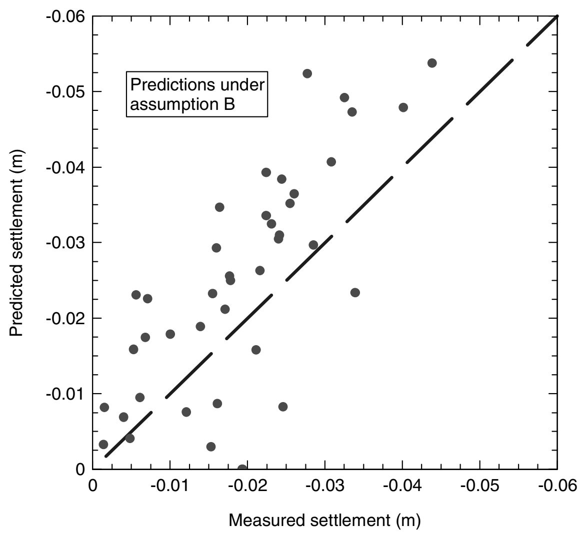

Figure 7. Measured and predicted (assumption B) settlements at 902 days

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