Reclamation Project of a Brownfield Site at Rio de Janeiro State, Brazil

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ABSTRACT: The paper presents the geotechnical solution proposed for remediation and further redevelopment of a Brownfield site from a bankrupted zinc processing industry. All installations were abandoned without a post closure technical plan, including a 1.2 million cubic meters mineral waste pile inside a 260,000 m^2 liquid waste pond. Both structures were settled upon a soft clay deposit contaminated with Zn and Cd. The technical solution proposed involved the complete drainage of the liquid pond, accompanied by on site treatment, a hydraulic barrier of pump & treat wells and the construction of an engineered waste containment facility using the mineral solid waste as compacted earth fill material combined to geosynthetic products.

KEYWORDS: Contaminated soil, Waste containment, Soft clay, Reinforcement with geosynthetics

1. INTRODUCTION

In 1997 a zinc processing industry closed in a bankruptcy process, leaving behind all installations, materials and residues, including a 260,000 m^2 liquid waste pond and a 1.2 million cubic meters mineral waste pile. With the end of operation, the industrial plant was abandoned and operation and safety maintenance services were no longer available, increasing the risk of accidents, particularly in the rainy summer season. An administrator was appointed by law to manage the site, but since the financial resources were very limited, the management of the risk and the corresponding expenses of control measurements were transferred to federal and state governments over the years. It became a well-known Brownfield site in the State of Rio de Janeiro, Brazil, the Ingá site.

In 2007, ten years later, the second administrator of the site hired a team from two universities, PUC-Rio (Catholic University of Rio de Janeiro) and COPPE-UFRJ (Federal University of Rio de Janeiro), to control the risk of accidents and to propose a technical solution for the site that would allow selling the area in a public auction with the agreement of the environmental agencies involved. The public auction was successfully realized in July 2008.

The paper presents the investigations performed in the 2007-2008 study and the engineering solution proposed and approved by the environmental agencies (COPPETEC, 2007-2008; PUC-Rio, 2005 and 2008). The solution combined environmental protection and geotechnical technologies.

2. SITE DESCRIPTION

2.1 Geotechnical Characterization

Figure 1 is an aerial view of the site at the time of the study, with the waste pile and the liquid waste pond (identified as C-130 when the industry was in operation), and showing Sepetiba Bay water at the upper side of the image.

To the right there is an elevation with a granite quarry that is still active. To the left there is mangrove vegetation covering both margins of a meandering river towards the bay, named as Saco do Engenho.

2.1.1 Field and Laboratory Investigations

Geotechnical investigation reports realized in the 80s for the design of improvements in installations and environmental control systems were partially recovered and used (NATRON, 1987). These investigations were combined to 60 borehole investigations performed in 2003 along the C-130 dike for the project of dike elevation by the State of Rio de Janeiro government (TECNOSOLO, 2003). The purpose at the time was to increase the height of the dike and consequently the pond water storage capacity to avoid overflow in the rainy season.

Figure 1 Aerial photograph on November 23rd 2007 (Courtesy by José Araruna Tavares Jr.)

A set of 8 inclinometers and 9 piezometers were also installed at the time of the heightening of the dike in 2004, and were tested and used from March 2007 to May 2008.

The waste pond was set upon a soft clay deposit with a history of dike and waste pile failures. To complement the existing data, a field and laboratory investigation program was established, comprehending 11 new boreholes in the waste pile and inside the pond, field vane tests and the sampling of 6 undisturbed Shelby samples (ϕ = 4") of the soft clay layer for laboratory tests in two positions inside the C-130 pond.

The waste material was investigated through borehole SPT tests in the pile and by laboratory tests performed on disturbed waste samples collected at the top and at the base of the pile, the last one to characterize the waste material in a slurry condition. The major objective was to investigate the mechanical, hydraulic and chemical behaviour of the waste after compaction for the design of the geotechnical solution for final disposal.

Although the dike was condemned since the beginning, a few laboratory and field tests were also performed in the existing dike material, to give support to the monitoring service for controlling the risk of accidents while the team was in charge at the site. Sand-cone tests were performed to measure field density at selected points in the dike, based on the 2003 borehole investigations results, which showed $N_{SPT} < 3$ in the earth fill layer at 64% of the boreholes along the dike, thus indicating that the compaction of the material of the dike was inadequate. Characterization and laboratory tests in soil specimens compacted at optimum condition were realized in a disturbed soil sample collected at the borrow pit site, to obtain parameters for the geotechnical design of the remediation solution.

All field investigations and collection points were located using a DGPS equipment and the UTM SAD69 coordinate system, and plotted on the site map obtained by combination of recovered 1988 topographic and project maps in 1:1000 total and 1:500 detail scales (PUC-Rio, 2005).

Table 1 summarizes the field and laboratory investigation program and methods applied, and Figure 2 presents the location map of the investigations performed in 2007-2008 and the industrial plant installations.

Table 1 Geotechnical investigation program and methods

Type	Test	Method	Notes
Field investigation	Borehole investigation with SPT tests	Brazilian standards NBR6484, NBR6502, NBR7250, NBR8036	Natural water content determined in all soft clay and waste SPT samples; boreholes stopped at impenetrable to percussion
	Field Vane test	Brazilian standard NBR10905	In one of the boreholes only 1 test was realized because of a sand layer
	Undisturbed sampling of the soft clay material	Shelby tube sampling (ϕ 4"); NBR9820	The Shelby tube sampler remained in the soil for 24 hours before extraction
	Sand-Cone test	Brazilian standard NBR7185	At five locations along the C-130 dike
Laboratory investigation	Physical index characterization: grain size distribution, specific gravity, Atterberg limits	Brazilian standards: NBR7181, NBR6508, NBR6457, NBR6459, NBR7180	Waste samples were tested with both sodium hexametaphosphate (conventional) and NaOH solutions as dispersant agent in grain size analysis
	Mineralogical analysis	X-Ray diffraction (Co radiation)	Only for the mineral waste material
	Compaction test	Normal Proctor energy; NBR7182	Borrow pit soil and waste material
	Permeability test	Falling head; NBR14545	Compacted samples of borrow pit soil and mineral waste material
	Consolidation test	Oedometer; NBR12007	Soft clay (undisturbed) and compacted mineral waste material
	Direct shear test	Brazilian standard NBR6122; specimen 6.0 x 6.0 x 2.5 cm	Compacted mineral waste material; shear rate 0.044mm/min (drained)
	Triaxial compression test	Undrained-Unconsolidated; Head (1982) procedure	Only one of the Shelby soft clay samples collected allowed moulding good quality specimens for the triaxial test
	Laboratory Vane test	Same procedure as in the field test	In substitution of the UU triaxial test

2.1.2 Soft soil geotechnical characterization

Soft soil deposits occur throughout the Brazilian coast as described by Massad (1999) and Almeida and Marques (2013). The geotechnical characteristics of the soft soil deposit of Ingá site are very similar to those of western Rio de Janeiro city (districts of Barra da Tijuca, Recreio dos Bandeirantes and Guaratiba). These deposits are from continental (fluvial and colluviums) and marine depositional systems, thus the areas are very poorly drained and under tidal influence (Almeida et al, 2008).

Western Rio de Janeiro soft clays are highly compressible, with low undrained strength and permeability, which means high settlements and a long time for stabilization. A soft soil deposit up to 11m of thickness was observed at Ingá site (Figure 3). Figure 4 shows the geotechnical profile at a particular critical section, with a history of geotechnical problems. Although the soft clay deposit thickness is not so large (5m) at that section, the corresponding N_{SPT} is lower than zero.

Table 2 summarizes the measured ranges of the soft clay soil parameters obtained from characterization, oedometer, field and laboratory vane tests and laboratory triaxial tests.

The compression index values obtained from the consolidation tests curves (C_c) were corrected by the Schmertmann factor. The compression index for Ingá soft soil deposit presented the same correlation with natural water content reported for the western Rio de Janeiro city soft clays ($C_c = 0.013w_n$). The range for undrained strength values includes field and laboratory vane tests and undrained unconsolidated triaxial tests results. The horizontal coefficient of consolidation was estimated as 1.5 times the value of the vertical coefficient of consolidation for a vertical effective stress of 100 kPa in the oedometer tests (2.35×10^{-8} m^2/s).

Table 2 Soft clay soil geotechnical parameters – measured ranges

Parameter/ characteristics	Values
Soft soil thickness (clayey silt or silty clay) (m)	2 – 11
OMC (organic matter content) (%)	5.0 – 8.3
% < #200 ASTM (fines content)	70 - 90
G (gravity density)	2.59 – 2.78
w_n (natural water content) (%)	105 – 227
I_P (plasticity index) (%)	55 – 80
γ_n (specific weight of natural soil) (kN/m^3)	11.5 – 14.5
S_u (undrained strength) (kPa)	0.6 – 11.2
CR (= $C_c/1+e_0$) (compression index)	0.24 – 0.43
c_v (m^2/s) (vertical coefficient of consolidation)	$(1.5–4.5)x10^{-8}$
c_h (m^2/s) (horizontal coef. of consolidation)	3.53×10^{-8}

Table 3 presents the undrained strength measured with different tests at different positions in the deposit.

Table 3 Soft clay soil measured undrained strength S_u (kPa)

Borehole	Depth (m)	Field Vane	Lab. Vane	UU Test
SP64	1.20-1.80	3.45	2.8	-
	2.70-3.30	11.20	2.8	-
	4.00-4.30	-	0.6	-
SP65	1.00	1.37	-	
	3.03-3.63	-	5.7	-
	4.13-4.73	-	-	4.9
	5.13-5.73	-	2.7	-

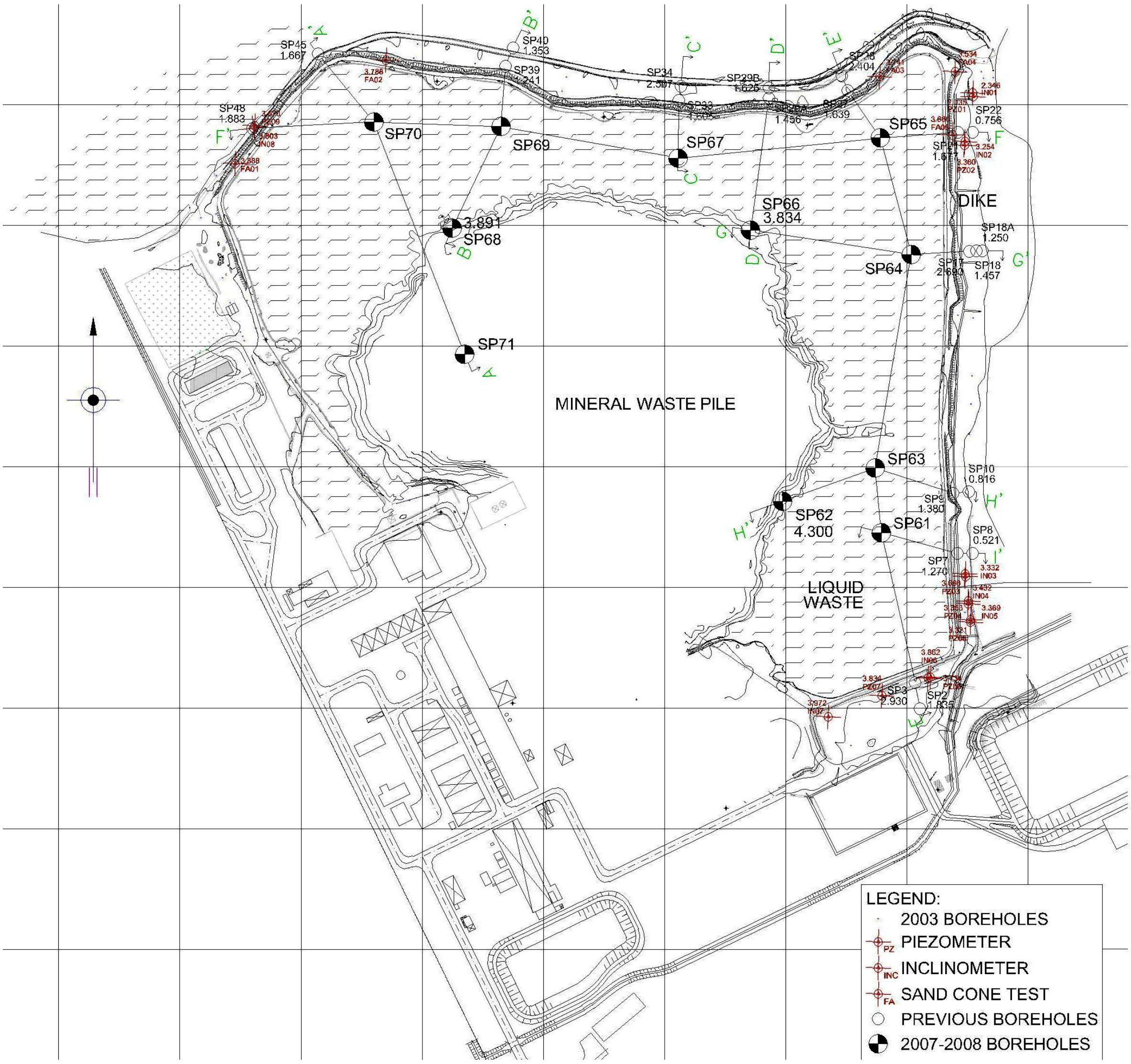

Figure 2 Site map with 2007-2008 investigations position and former plant installations

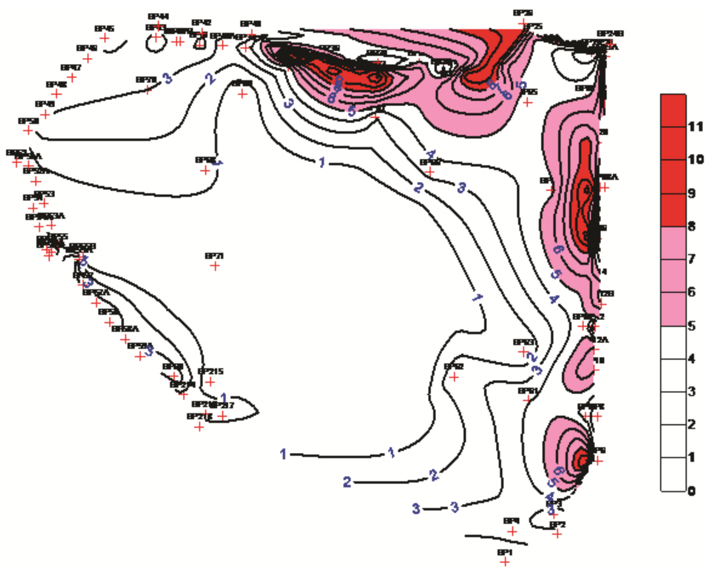

Figure 3 Thickness of the soft clay layer within the C-130 waste pond limits (vertical scale in meters)

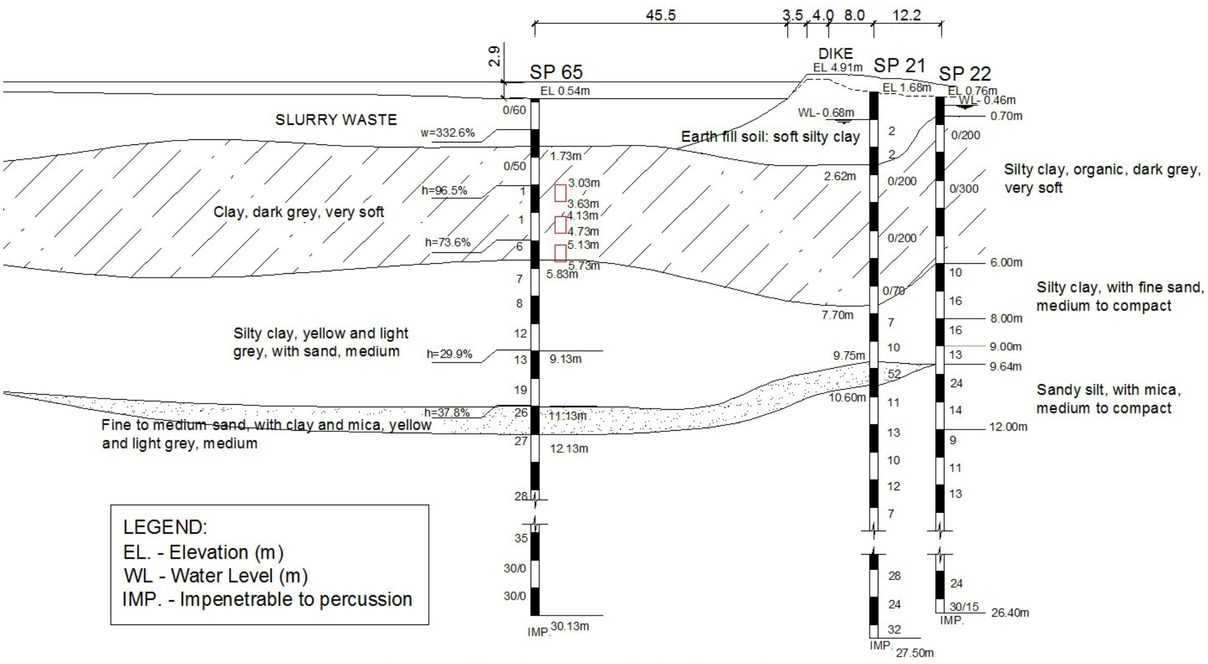

Figure 4 Geotechnical profile below dike at critical section

2.2 Site Contamination

Earlier investigations comprehended the industrial plant area and the surrounding sediment and water environments (Saco do Engenho and Sepetiba Bay). It was concluded that the relevant environmental risk at that time and for the future was limited to the plant area soil and groundwater contamination, and particularly to the risk of accidents (dike failures and/or overtopping) at the C-130 pond (PUC-Rio, 2005).

The characteristics of the plant processing units, procedures, incoming materials and resulting products, wastes and effluents were studied, and the most probable contaminants were defined as Zn (by far the most important) and Cd.

2.2.1 Investigation Methods

Budget limitation restricted the investigations to individual samples collection at selected locations inside the plant, and to the construction of 25 monitoring wells. Temporal variations were not determined.

Liquid and solid material samples were collected in the existing ponds and abandoned installations, stored in plastic bottles (liquids) or bags (solids), labelled and taken to the laboratory for chemical analysis by atomic absorption spectrometer (liquid samples) and by X-Ray fluorescence (solid samples).

The monitoring wells (D = 6") were excavated with a direct push hollow auger, with 4" PVC tubes for groundwater sampling. The construction profile followed standard recommendations (NBR13895; Fetter, 1999), and is presented on Figure 5. The depth of each well was determined during construction by measuring the presence of Zn in small soil samples taken at the bottom of the direct push sampler. The analysis was made on site in twenty minutes, and the well was terminated 1.0 m below the last negative result. Position and details of the monitoring wells are presented in the section describing the hydraulic barrier design.

Soil samples were collected in HDPE liners 0.9 meter long, during construction of the monitoring wells. Delmonte (2011) evaluated the collected soil samples potential for groundwater contamination through batch extraction method with acid solution (NBR10005) and with deionized water (NBR10006) as indicative parameters. The acid leached and water solubilized extracts were analysed by the PIXE (Particle Induced X-Ray Emissions) method at the Physics Department of PUC-Rio. A similar leaching and extraction procedure with deionized water was realized by Wesson et al (2007) to measure the metals release from a contaminated soil.

Groundwater samples were collected in 2008 in the monitoring wells using a low-flow bladder pump (QED T1250 model). The samples were preserved with nitric acid and taken to the laboratory for Zn and Cd analyses by atomic absorption spectrometer (PGI 990I).

2.2.2 Results

Table 4 summarizes the soil and groundwater ranges of concentration for Zn and Cd, including C-130 pile waste, liquid and bottom sediment results. The two last columns represent the Zn and Cd concentrations measured in the extracts obtained by solubilization with deionized water. The highest concentrations corresponded to the lower pH values in all types of samples.

3. RECLAMATION PROPOSAL

3.1 General Proposal

It was recommended the C-130 Pond closure as a priority, eliminating the rainwater storage and contamination, accompanied by the installation of pump & treat wells along the downstream limits of the site (Hydraulic Barrier), and the construction of an engineered waste containment facility using the mineral solid waste as compacted earth fill material.

3.2 C-130 Pond Closure

Hydrological calculations defined the minimum liquid extraction rate and treatment capacity necessaries to drain the pond accounting for rainwater contributions over time. The average rate applied in 2008 was 4,000 m^3/day.

The system consisted of pumping the liquid waste using a mobile floating base, into a mixing chamber and then to a sedimentation pond. Calcium hydroxide was mixed to the pumped liquid in the chamber to increase the pH to 8.5-9.0 and the Zn and Cd precipitated in the pond as hydroxides.

The treatment system was kept operating afterwards by the new owners of the site until complete drainage and beginning of the reclamation earthworks.

Table 4 Soil, groundwater and waste Zn and Cd concentration ranges in 2007-2008 (in $mgkg^{-1}$ for solids and μgL^{-1} for liquids)

Type	**Depth (m)**	**Total - Solids Zn**	**Total - Solids Cd**	**Soluble Zn**	**Soluble Cd**
GW	Var.	-	-	1200-7.2×10^6	20-6400
Soil	0 - 1	-	-	570-264100	3-71
Soil	1 - 2	-	-	1970-164370	2-83
Soil	2 - 3	-	-	490-114190	3-46
Soil	3 - 4	-	-	11850-77170	11-106
Soil	4 - 5	-	-	168570*	7-60
Soil	5 - 6	-	-	None	57*
Pile waste	C-130	37903-40410	0 - 36.2	-	-
Bottom Sediment	C-130	8300-132720	300-480	-	-
Pond Liquid		-	-	797000-861000	2900

Notes: GW – groundwater; * - Only one sample contaminated.

3.3 Hydraulic Barrier

Figure 5 presents the proposed hydraulic barrier system with 38 pump & treat wells. The position was based on hydrogeological and hydrological flows at the site. The water level is close to the surface and there are two surface channels at the North and South limits of the area. The surface drainage and the channels discharge into the flooded mangrove plain towards the Saco do Engenho. Groundwater flows towards the corner between Channel North and the mangrove plain (local of the geotechnical critical section).

The average radius of influence of each well was calculated as 20 meters. The field hydraulic conductivity was 10^{-7} m/s along the central row parallel to the mangrove and 10^{-6} m/s along the North and South limits because of the presence of an alluvial sandy sediment. The expected extraction flow of each well was 3.12 m^3/year at the central row and 31.2 m^3/year at the lateral rows. For the projected complete hydraulic barrier it was expected a total flow rate for treatment around 65 m^3/month.

The wells were designed for extraction of contaminated groundwater only. For that reason each well was terminated 1.0 m below the last negative result for Zn in the soil. The unnecessary extraction of uncontaminated groundwater increases the volume of liquid for treatment and dilutes the concentrations, without getting a better performance in the remediation process. The design of pump & treat wells is presented in Domenico and Schwartz (1998).

3.4 Engineered Waste Containment Facility

The foundation soil and groundwater are already contaminated, but soil removal is not feasible. The best alternative is therefore the construction of a new waste containment facility upon the contaminated soil layer. The system to control the advance of contamination outwards the site is the hydraulic barrier.

The new waste containment facility had to accomplish the following requirements:

- The use of the pile waste as earth fill material
- To control the Zn and Cd advance from the waste into the surrounding environment
- To allow the construction of new installations or other facilities on top of the deposit
- To present adequate geotechnical stability and settlements conditions under use, accounting for the presence of the soft clay soil in the foundation layer
- To constitute a flexible design that could be adapted for different activities, since the future land use was not known in advance, depending on the public auction result

3.4.1 Materials and Conceptual Design

Two basic materials were planned to be used for construction of the new deposit: a compacted clay soil from a local borrow pit site, and the compacted pile waste. Complementary materials are also necessary such as sand for drainage systems and different types of geosynthetics for waterproofing liners, drainage and reinforcement.

Table 5 (a) shows the geotechnical characterization properties of both clay soil and waste, and Table 5 (b) the geotechnical properties of interest after compaction.

The conceptual design of the new deposit is represented on Figure 6 at two conditions: (a) external dike section, and (b) internal division dikes section.

Table 5 Borrow pit clay soil and pile waste geotechnical properties

(a) Characterization and index properties

Material	**G**	w_L (%)	w_P (%)	I_P (%)	**Grain Size Distribution**				
					Clay	**Silt**	**Fine Sand**	**Med. Sand**	**Coarse Sand**
Waste/HEXA	3.134	37.8	15.0	22.8	1	56	20	10	13
Waste/NaOH					11	61	21	5	2
Borrow pit soil	2.718	66.5	23.3	43.2	42	16	14	17	10

(b) Geotechnical properties after compaction

Material	**Normal Proctor**		**Consolidation Test**			**Shear Strength**(1)		k_{sat} **(m/s)**	
	w_{opt} (%)	γ_{dmax} (kN/m^3)	C_c	C_r	C_v (m^2/s)	c' (kPa)	ϕ' (°)	w_{opt}	w_{opt} + 5%
Waste	35.3	13.68	0.47	0.06	4×10^{-8}	10.2	41.8	5.6×10^{-8}	2.4×10^{-8}
Borrow pit soil	25.5	15.20	-	-	-	0	28	1.5×10^{-7}	-

Note: (1) Direct shear test for the waste; parameters for the compacted soil from NATRON (1987) document.

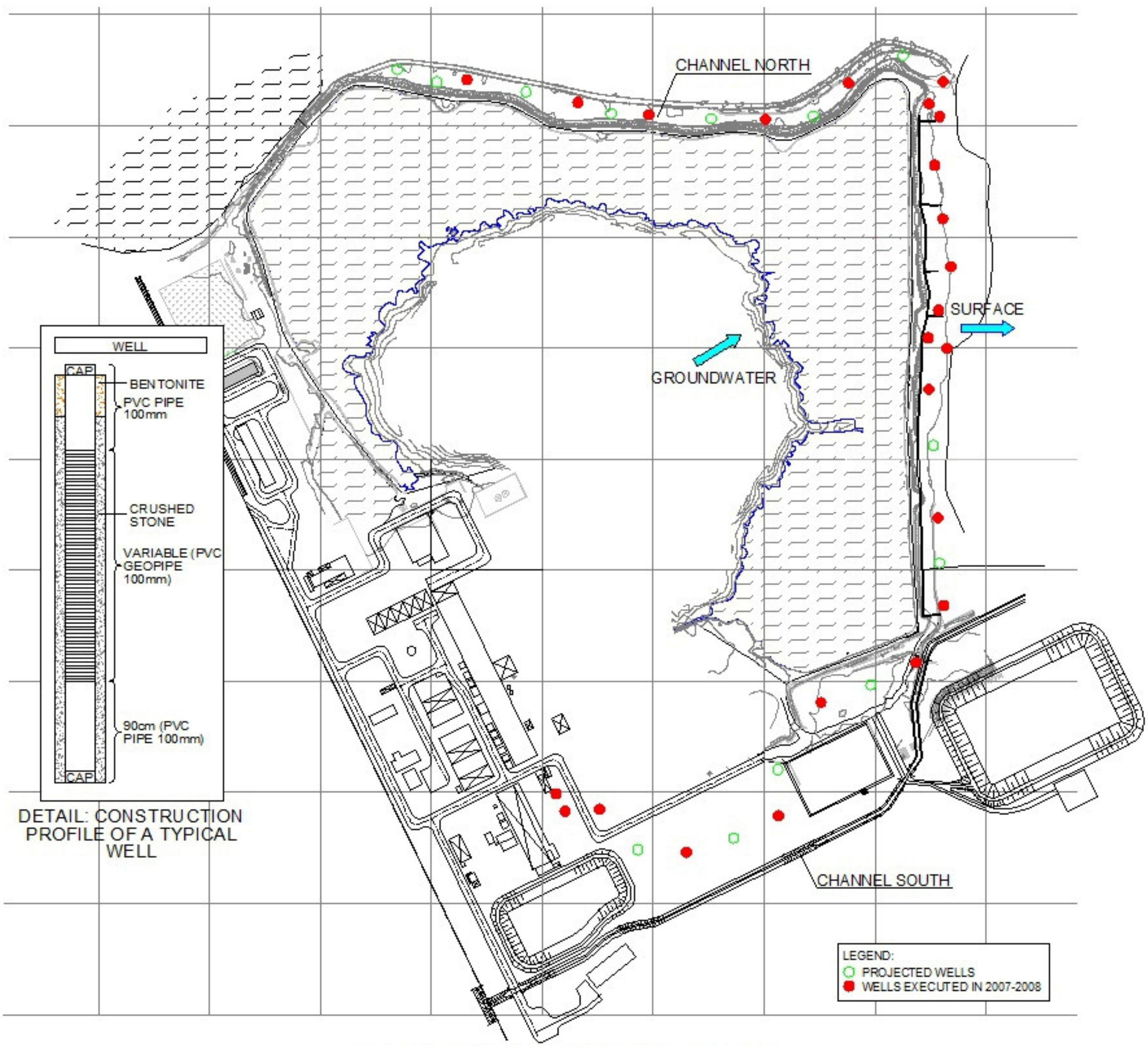

Figure 5 Monitoring wells and the hydraulic barrier proposed

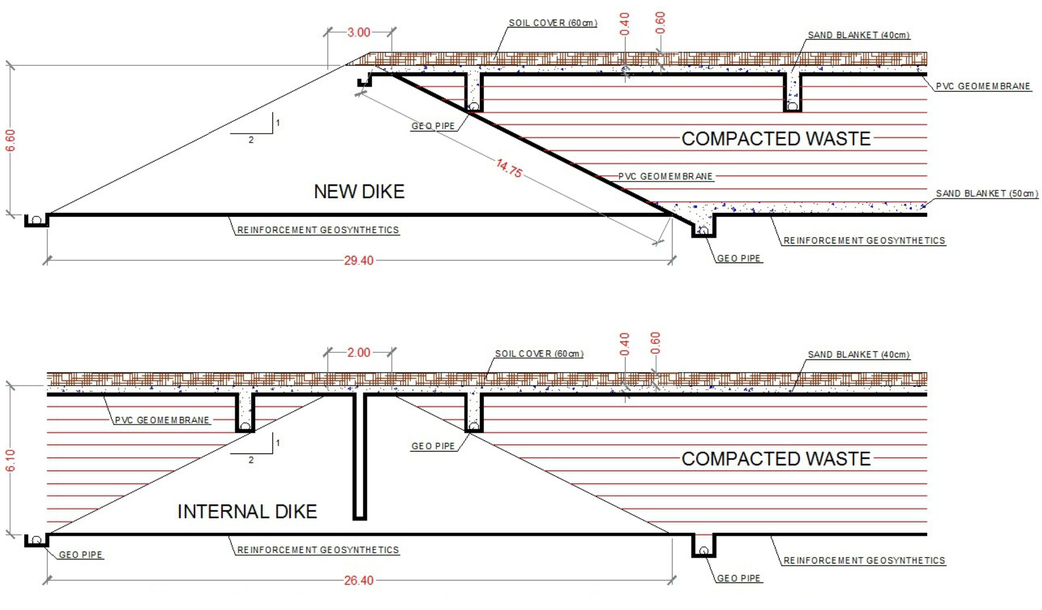

Figure 6 Proposed design for the new waste disposal facility – (a) external dike section, (b) internal division dike section

The internal division dikes are necessary to facilitate the construction of the deposit in stages. The disposal area should be divided therefore in at least four cells. Besides, the construction in stages allows the consolidation and gain of strength of the soft clay foundation layer below the last constructed cell while the next cells are being developed.

The existing dike material cannot be reused since it is already contaminated. It must be treated as a residue instead and deposited together with the pile waste inside the containment facility.

As pointed out by Dunn (1995) for the United States, since the 80s an increasing number of landfill post-closure development projects have been implemented, most of them successful. According to the author' experience the key condition for success is a good investigation and adequate understanding of the waste and subsurface conditions at each particular landfill site.

Deep foundation is usual and driven piles are the preferred systems since they do not result in waste or contaminated soil extraction to the surface. Dunn (1995) points out five important issues of the foundation design: (i) pile capacity; (ii) downdrag loads due to settlements; (iii) constructability and construction impact; (iv) corrosion resistance of the piles; (v) maintenance of the integrity and the environmental protection characteristics of the waste containment.

Keech (1995), discussing post-closure construction over MSW landfills, states three key design goals: (i) achieve acceptable performance with expected settlements over time; (ii) avoid penetration of the barrier layer; and (iii) minimize future maintenance requirements.

Kamon (2001) emphasizes that geotechnical engineering can give a significant contribution to the solution of many environmental problems regarding waste management and remediation of polluted sites. One of the contributions mentioned is the waste potential for re-use in earthworks, particularly of mineral wastes, in place of natural soils. Evidently, the environmental risk has also to be evaluated in such applications.

All those statements are still valid and were taken into consideration. The design represented on Figure 6 presents the following features because of the specific condition of the very soft clay foundation soil already contaminated:

- No geomembrane at the bottom of the deposit, but a drainage system instead, to collect the contaminated water drained from consolidation of the underlying soft clay layer
- All internal bottom drainage systems linked to the collection tubes of the hydraulic barrier, that transport the extracted groundwater for treatment
- A geomembrane liner installed along all contact surfaces between the external dike and the waste deposited, and also at the surface of the waste
- The geomembrane to be selected must have a good elongation capacity, since differential settlements will probably occur at the surface of the deposit, and between cells
- The risk of Zn and Cd release from the solid waste is reduced by minimizing the water infiltration into the deposit with the geomembrane and the engineered cover
- At the most critical foundation soil conditions (see Figure 3) a reinforcement geosynthetic will be necessary at the bottom of the deposit, below the dikes and the waste.

3.4.2 Geotechnical Project

For the stability analyses it was adopted a minimum factor of safety FS = 1.3 and undrained condition after construction for the soft clay soil. The analysis was performed at 2D condition for a typical section over 5.8 m thick soft clay layer (dominant thickness observed), and dividing the soft clay layer in three sub layers according to the undrained strength measured in field and laboratory tests (Table 3), with vane tests results corrected by the Bjerrum factors (Abreu and Barbosa, 2009).

Three scenarios were analysed: [1] one step construction and no reinforcement; [2] one step construction and reinforcement; and [3] four steps construction, gain of strength at six months after each step and reinforcement.

The scenarios [1] and [2] were analysed by Ehrlichs' analytical method (in Moraes, 2002), presented in Figure 7 and equations (1) (no reinforcement), (2) and (3) (with partial reinforcement), and (4) (for both conditions). As expected, scenario [1] presented a very low FS (close to zero), and scenario [2] resulted in unrealistic values for anchor (171 m) and reinforcement force (1,411 kN/m) to attain the required FS=1.3.

$$FS = \frac{S_u}{\gamma_a . H}\left(\frac{3D + \dfrac{H}{n}}{D + \dfrac{(H*)^2}{2H}.K_a}\right) \quad \text{for } D < \frac{H}{n} \tag{1}$$

$$FS = \frac{S_u}{\gamma_a . H}\left(\frac{3 + \dfrac{L_r}{D}}{1 + \dfrac{(H*)^2}{2HD}.K_a}\right) \tag{2}$$

$$T = E_a + S_u . \frac{L_r}{FS} \tag{3}$$

$$H* = H - \frac{2c'}{\gamma_a}.\tan\left(45^\circ + \frac{\phi'}{2}\right) \tag{4}$$

Where:
FS → factor of safety
H, D, n → geometrical parameters as defined in Figure 7
$H*$ → embankment height without tension zone (m)
γ_a → embankment unit weight (kN/m^3)
c' → embankment effective cohesion intercept (kPa)
ϕ' → embankment effective friction angle (°)
$(E_a)_{EF}$ → active earth pressure in the embankment (kN/m)
E_a → active earth pressure in the soft soil layer (kN/m)
E_p → passive earth pressure in the soft soil layer (kN/m)
K_a → coefficient of active stress in the soft soil layer
S_u → undrained shear strength of the soft soil layer (kPa)
L_r → anchor length (m)
T → maximum pullout force (kN/m)

An adequate factor of safety was obtained for the scenario of construction in four steps: ΔH_1 = 0.5m, ΔH_2 = 1.0m, ΔH_3 = 2.5m and ΔH_4 = 3.1m (total height H = 7.1m). The time interval between successive steps was 6 months.

The total settlement was calculated as 1.94 m by Terzaghi one dimensional consolidation theory. Since the time necessary for stabilization was too long (98% in 69 years, single drainage), the use of vertical drains was studied by the Equal Strain method of Barron (1948). The $\overline{U}$ x T relationship was thus obtained for the radial consolidation condition, which stabilizes in 2.26 years.

For each step the total and partial settlements and the corresponding average consolidation ratio were calculated for 6 months, considering radial drainage (vertical drains) and accounting for the submergence effect by the iterative method of Martins & Abreu (2002). This method uses the effective stress x vertical deformation curve obtained in the consolidation test instead of the compressibility parameters. The submergence effect on the earth fill unit weight is used to correct the earth fill surcharge and the corresponding settlement at that time. The interactions continue until an error less than 1% is obtained between the settlements calculated in two successive interactions.

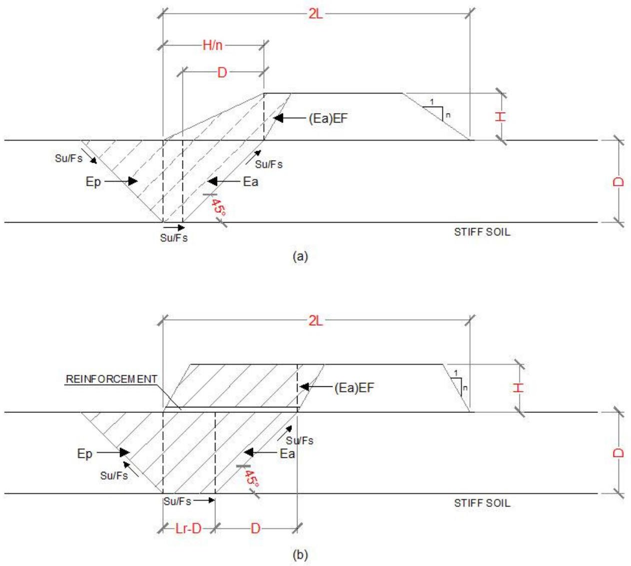

Figure 7 Ehrlichs' analytical method for stability analysis of an embankment upon a soft clay soil (a) without and (b) with partial reinforcement.

For the second to the fourth construction stages the gain of strength in the foundation clay soil after the last 6 months of consolidation related to the previous construction steps was taken into consideration in the stability analysis at end of construction. The gain in strength was calculated applying Mesri correlation between the undrained strength and the maximum past consolidation stress, as follows:

Effective stress increase by consolidation:

$$\Delta\sigma_v' = \Sigma(\Delta\sigma_v \times \overline{U}) \tag{5}$$

Gain in undrained strength:

$$S_u = (S_u)_o + (0.22 \times \Delta\sigma_v') \tag{6}$$

Where:

$\Delta\sigma'_v$ → increase in effective stress after consolidation at time t(kPa)

$\Delta\sigma_v$ → surcharge stress of each previous step (kPa)

$\overline{U}$ → average consolidation ratio of each previous step at t

T → Time factor (vertical or radial drainage)

$(S_u)_o$ → original undrained strength (measured) (kPa)

For the above conditions: construction in four steps, division in cells, radial drainage (1.5 m spaced vertical drains in a triangle mesh), adequate factors of safety would be achieved with a reinforcement geosynthetic with 447 kN/m of tensile resistance (without reduction factor for deterioration of the geosynthetic) and anchor length of 53 meters at the most critical sectors of the new deposit (see Figure 3). And the submergence effect reduces the total settlement from 1.94m to 1.45m. The stability analyses of scenario [3] were performed with the SLOPE/W2004 program (GEO-SLOPE International Co.).

3.4.3 Alternative of waste physical-chemical stabilization

One of the strategies for remediation of polluted soils and/or wastes is the physical-chemical stabilization method. Two major goals can be achieved: (i) to eliminate or substantially reduce the risk of contaminants release to the environment by fixing the contaminants inside the stabilized matrix; (ii) to increase the stiffness and the resistance of the material after stabilization. Kamon (2001) presents an extensive review on stabilization methods for a diversity of wastes for re-use in geotechnical engineering. The most widespread method comes from soil stabilization practice and consists of employing chemical additives such as cement.

This alternative was studied in the laboratory for the mineral waste of the Ingá pile (Barbosa et al, 2009). The minimum amount of Portland cement (CP-II) to be added was defined by the physical-chemical dosage method proposed by Chadda (1970) and modified by Castro (1995). Three conditions were tested: 3%, 5% and 7% of Portland cement added to dried samples of the waste passing in the ASTM #10 mesh sieve. The mixtures were then humidified and compacted with Normal Proctor energy and submitted to simple compression tests in a hydraulic press (1 metric ton; 0.3mm/min) after 14 days curing, in duplicates. The optimum result was obtained for a water content of 12% and 3% of cement added. A sample molded and compacted at this condition was subjected to Batch extraction tests with deionized water and with acid solution (EPA SW86, NBR10005, NBR10006) to evaluate the metals encapsulation in the stabilized mixture. The results obtained are presented in Table 6.

Table 6 Leaching tests of the compacted waste (w = 12%)

Sample	**pH**	**Leached Extract**	
		Zn (mg/L)	**Cd (mg/L)**
No cement	4.82	487	0.56
3% CP-II	5.34	291	0.34
Sample	**pH**	**Solubilized Extract**	
		Zn (mg/L)	**Cd (mg/L)**
No cement	6.12	1,105.0	1.73
3% CP-II	8.28	0.013	< 0.01

The physical-chemical stabilization method with Portland cement proved to be effective in reducing the risk of contaminants release from the waste to the environment, and the engineered cover and geomembrane would be no longer necessary. On the other hand, the operational requirements on site and the bad foundation conditions, besides the amount of cement necessary for the amount of waste to be stabilized in that particular site, are serious disadvantages for the adoption of the stabilization solution.

4. CONCLUSION

The combination of environmental and geotechnical technologies was able to define a feasible solution for a Brownfield site involving a 1 million cubic meters waste pile, a contaminated liquid waste pond with a history of failures and overtopping accidents, and a very soft foundation soil and groundwater contaminated with Zn and Cd close to an important bay.

A hydraulic barrier of pump & treat wells contains the advance of the contaminated water outwards the site. A new waste containment facility using the industrial waste as earth fill material was designed. The rain water infiltration and leaching of the deposit is controlled by an engineered cover including a geomembrane liner. At the bottom, instead of a barrier system, a drainage system collects the contaminated water from the consolidation of the soft clay layer towards the hydraulic barrier collecting and treatment system.

The solution proposed is technically flexible and permits the execution of driven piles by the new owner through the waste containment facility without compromising the environmental protection features.

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LIST OF ACRONYSMS

ABNT – Associação Brasileira de Normas Técnicas (Brazilian Standards Association)
ASCE – American Society of Civil Engineers
ASTM – American Society of Testing Materials
COPPE – Alberto Luiz Coimbra Institute – Graduate School and Research in Engineering
EPA – Environmental Protection Agency (US)
NBR – Norma Técnica Brasileira (Brazilian Standard)
PUC-Rio – Catholic University of Rio de Janeiro
UFRJ – Federal University of Rio de Janeiro

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