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Kilamba building foundation design

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ABSTRACT: This paper addresses the jet grouting plug proposed by the Contractor for the foundations of Kilamba building in Luanda (Angola). The general geological characteristics of the deposits, the field and laboratory tests are addressed. The design of bottom slab and plug on jet grouting is discussed. The construction issues, the quality control and quality assurance plan are referred. The monitoring plan is addressed. A survey of adjacent structures is introduced. A summary of conclusions is presented

1 INTRODUCTION

This paper addresses the jet grouting plug proposed by the Contractor for the foundations of Kilamba Project in Luanda. After this introduction the general geological characteristics of the deposits, the field and laboratory tests are addressed.

The design of bottom slab and plug on jet grouting are discussed.

The construction issues, the quality control and quality assurance plan are referred.

The monitoring plan is addressed.

A survey of adjacent structures is introduced.

A summary of conclusions is presented.

2 GEO-RUMO FDO-ABB ALTERNATIVE SOLUTION

The Contractor GEO-RUMO-FDO-ABB has proposed an alternative solution for Kilamba project foundation with a diaphragm wall 0.6 m thick supported by pieces of slabs located at level 0, level –1, level –2, and a jet-grouting plug at the base of the excavation (JETSJ, 2009).

This solution was considered more feasible that the execution of anchors under water level proposed by DAR designer. In addition the jet grouting plug, located 1.8 m deep the level of excavation, was considered a better solution due to the geotechnical scenario of sandy materials. This plug was materialized by jet grouting columns 1.8 m diameter and 3.0 m long distributed in quinconce and 1.3 m apart.

Under the last cave a bottom slab with 0.40m thick and reinforcements of 1.2 m minimum high were proposed. A drainage system was installed to pump the water percolate through the foundation and diaphragm wall. The loads transfered by the superstructure were supported by bored piles with 1.0 m diameter and lengths between 20.0 m to 25.0 m.

The structure was analised by SAP 2000 code using a 3D model.

The soil –structure interaction was based in Winkler model.

Based on the geotechnical characterization the following subgrade modulus were considered:

- soil around pile: subgrade reaction modulus 40 000 kN/m^3.
- soil at the massif base: reaction modulus 6000 kN/m^3.
- soil at the buttom of the micropile: reaction modulus 8 000 kN/m^3.

The analysis has given the values of tension forces, compression forces and bending moments.

The forces and deformations of the diaphgram wall simulating the different construction phases were obtained by PLAXIS code. Horizontal displacements of 25 mm and vertical displacements of 23 mm were computed. The calculated values have shown that ultimate limit sates and serviceability limit sates were verified. The bearing capacity of the piles and micropiles was defined based in the partial safety factors defined in Eurocode 7. For the diaphragm wall the following limit states were verified: (i) loss of global safety factor; (ii) failure due insuficient resistant capacity; (iii) failure by sliding; and iv) combined failure of ground and structure.

The static water pressure causes a destabilising uplift force on the structure, which is resisted mainly by the self-weight of the structure, but there may also be some resistance from the ground around the structure due to the pile role. It is important to stress that CPRF (Combined Pile Raft Foundation) a composite foundation of 3 elements namely pile, raft and soil has proved very successful for the tall buildings with low settlements and economic costs. The MAIN TOWER in Frankfurt with 57 storeys is a good example.

3 GEOLOGICAL AND GEOTECHNICAL SURVEY

The first geotechnical campaign performed by Teixeira Duarte (2007) comprised the execution of 4 boreholes with depths between 20 to 28 m. In the boreholes SPT tests spaced 1.5 m were performed and also the water level was recorded. The SPT sampler was used to collect disturbed samples.
Based in the interpretation of the boreholes the following units were considered:

- sand landfills (C1A), with grey material and thickness less than 2.5 m;
- alluvial deposits (C1B), with thickness around 6 m, composed by fine sands with mud with SPT values between 3 and 11 blows;
- underlying this layer occurs fine sands and silty clay materials (unit C2A of Miocene), with SPT values between 12 and 38 blows mixed with stiff clay materials with SPT values between 13 and 33 blows;
- for higher depths occur fine sands very compacted with SPT values higher than 60 blows and stiff sandy clays with SPT values higher than 24 blows classified as unit C2B of Miocene.

The water level was detected at ground level. In borehole S2 samples were taken at depths 10.5 m, 15.0 m and 20.0 m and in boreholes S3 and S4 the samples were taken at depths 5.0 m, 10.5 m, 15.0 m and 20.0 m. The collected samples were used to perform laboratory tests, namely sieve analyses and Atterberg limits. For the clay materials the values of liquid limits were between 41 and 44% and the plasticity index were between 16 and 17%.

For the sandy materials the Atterberg limits gave nill values. The percentage of fines (material passed in ASTM seive 200) was between 0.9% and 26.8%, and the sandy materials have exhibited the lower values. A second geotechnical campaign was performed, for the occasion of the alternative solution proposed by the Contractor, and 3 boreholes were performed by Tecnasol (2009). One borehole was located in the centre of the site and two other were located near the boundaries.

The boreholes have reached depths between 30 to 31.5 m and SPT tests were performed 1.5 m apart. In boreholes SC1 and SC3 hydraulic piezometers to measure the variations of water level were installed. Tecnasol report gave design parameters, namely friction angle, cohesion and elasticy modulus. Taking into account that the initial geotechnical characterization by Teixeira Duarte has not characterized the permeability properties of sandy silty clays materials underlying the bottom of excavation the Contractor has performed a pumping permeability test to address this issue.

4 JUSTIFICATION OF JET –GROUTING PLUG POSITION

The solution proposed by DAR (2009) was a curtain wall with anchors extending into neighboring properties. The implementation of this solution creates the following difficulties:

i) Permission must be obtained from the owners of the affected properties;
ii) Extra measures need to be taken to minimize any adverse effects;
iii) In some situations anchors extending under existing buildings may not be feasible;
iv) In addition there was a need to perform a huge dewatering of water surface around 10 m.

Within this framework the Contractor has proposed an alternative solution with the construction of a jet grouting plug 3 m thick to avoid the detrimental effects of the dewatering (JETSJ/GEO-RUMO, 2010a, 2010b). By grouting walls and base rafts slabs the permeability of the excavated faces was drastically reduced to decrease the amount of groundwater flow into the site (Geo-Institute of ASCE, 2009). For the design and execution the Constructor has followed the recommendations of EN 12716 (2001) .

The position of jet grouting plug was defined after a careful analysis of all the arguments for and against and so the decision was based in the following factors:

(i) Cohesion in the soil tends to reduce the ease with which the particles are eroded. As the diameter of the grout tends to reduce as the silt and or clay content increases, the jet grouting plug was located at depths around 12 m from the ground level where sandy silty materials were occurring;
(ii) As the stiffness of the cohesive soil is important and only soil with low values of SPT should be regarded as being suitable, it was not recommended to locate the jet grouting plug at high depths where stiff Miocene clays were occurring;
(iii) Soils with high stiffness requires high pressure water jet that could be detrimental to the neighbour buildings and so it was not advisable to locate the jet grouting plug at high depths due the occurrence of materials with SPT values higher than 60 blows;
(iv) Based in the geotechnical survey due to the existent silt/clay materials with high values of SPT it is not advisable to built the jet grouting plug at high depths;
(v) At level - 18 the soil was so stiff that there was a need to use the jack hammer during 2 hours to dismantle a layer of soil with 0.5 m thick.
(vi) Sealing between piles at the top will construct a pile raft foundation. It is important to stress that CPRF (Combined Pile Raft Foundation) a composite foundation of 3 elements, namely pile, raft and soil has proved very successful for tall buidings with low settlements and economic costs;
(vii) Following the experience of Delf Institute construction of a jet grouting plug at high depths will increase the displacements.

In order to minimize any detrimental situation during the excavation the following contingency measures were proposed:

i) The placement of piezometers will allow the monitoring of water level;
ii) Through the holes placed in the jet grouting plug water could be pumped, if necessary;
iii) As alternative injection grouting can be used to control the leaks;
iv) Ground excavations will be performed in segments.

5 DESIGN OF JET GROUTING PLUG AND BOTTOM SLAB

For the design of jet grouting plug and bottom slab the following limit states should be respected.
The UPL ultimate limit state is "*loss of equilibrium of a structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions*". The most common UPL ultimate limit state is when static water pressure causes uplift of a structure with a deep basement. In this situation the static water pressure causes a destabilising uplift force on the structure, which is resisted mainly by the self-weight of the structure, but there may also be some resistance from the ground around the structure due to the strength of the ground hydrostatic pressure, HYD is when hydraulic gradients cause failure by the force of the upward seeping water exceeding the weight of the soil.

In design against uplift failure or UPL ultimate limit states, EN 1997-1 states in *2.4.7.4(1)* that the equilibrium equation to be satisfied is:

$$V_{dst,d} \leq G_{stb,d} + R_d \qquad (1)$$

where the design destabilising action, $V_{dst,d} = G_{dst,d} + Q_{dst,d}$ can be a permanent and/or a variable uplift force. In the case of uplift due to groundwater pressure, this may be interpreted as the groundwater pressure having a permanent and a variable component. The design stabilising action is the design self-weight of the structure, $G_{stb,d}$, which is a permanent load, and there may also be some additional design resistance, R_d between the soil and the structure.

Table 1. Partial factors for UPL ultimate limit states

Partial factors	UPL
Actions	
$\gamma_{G,dst}$	1.0
$\gamma_{G,stb}$	0.9
$\gamma_{Q,dst}$	1.5
Soil Parameters and Resistances	
$\gamma_{\phi'}$	1.25
$\gamma_{c'}$	1.25
γ_{cu}	1.4
$\gamma_{\sigma;t}$	1.4
γ_{a}	1.4

Partial factors on the Uplift Force

In most of the solutions the uplift force from the groundwater pressure was treated as a permanent action and thus the partial factor $\gamma_{G,dst}$ = 1.0 was used. However, in some solutions the uplift force was treated as a variable action with $\gamma_{Q,dst}$ = 1.5, giving rise to very large base thicknesses. One solution stated out that, if the mean groundwater level were known, the uplift force due to the groundwater at this mean level could be treated as a permanent action and the uplift force due to the difference between the mean groundwater level and the higher nominal groundwater level could be treated as a variable action. In another solution, the uplift force was treated as an accidental action, with $\gamma_{G,dst}$ = 1.0, which is equivalent to treating the uplift force as a permanent action (Pinto, P. S.S., 2011).

The main differences between the solutions arose due to different assumptions regarding the resistance between the sand and the side walls of the basement. In general in the solutions based on Eurocode 7, some resistance is assumed and this is determined from the shear stress, given by an equation of the form $\tau = K\sigma_v'\tan\delta$, where K is the coefficient of horizontal earth pressure and is a function of ϕ', σ_v' is the vertical effective stress, and δ' is the friction angle against between the soil and wall. Since σ_v' is calculated from the weight density of the soil, no assumptions are required to calculate it and no partial factors were applied to it. However, different assumptions are made in the solutions regarding the values of K and δ and regarding what partial factors should be applied to obtain the design values of the combined parameter $K\tan\delta$.

The wall friction angle δ is set equal to (2/3) ϕ' in the majority of the solutions, in accordance with the maximum value stated in *9.5.1(6)* (EN 1997-1) for precast concrete walls, while the others used $\delta = \phi'$, which is the value that may be assumed for concrete cast against soil.

Regarding the coefficient of earth pressure K the solutions are divided between those who chose $K_0 = (1 - \sin\phi')$ and those who chose the more conservative K_a value, mostly obtained from *Figure C1.1* of EN 1997-1 taking account of the wall friction angle.

Regarding the application of partial factors to $\tan\phi'$ and $\tan\delta$ or to $K\tan\delta$ there are 5 different possible ways this can be done, as set out in Table 2, all of which may be considered to be in accordance with EN 1997-1. The first row in this table is when no factors are applied and provides the characteristic $K\tan\delta$ value, assuming $K = K_a$. The main features of the different ways of applying the factors and the resulting overall factor of safety, OFS, which is the ratio of the characteristic $K_a\tan\delta$ value to the design $K_g\tan\delta$, value, i.e. $K_{a,k}\tan\delta_k/(K_a\tan\delta)_d$, may be summarised as follows:

1. Applying the factor $\square_{\square'}$ to reduce □' and reduce □, causes □ to reduce but causes K_a to increase, which is unconservative. This results in an overall factor of safety for $K_a$tan□ of 0.96; i.e. less than 1.0 and hence no safety because the design $K_a$tan□ value using this method is 0.103, which is greater than the characteristic value.
2. Applying the factor $\square_{\square''}$ to reduce □' and □ and then treating the resulting side friction force, which is an additional resistance to uplift, as a stabilising permanent vertical action in accordance with *2.4.7.4(2)* and applying the action factor $\square_{G,stb}$ to it as an action, results in an overall factor of safety of 1.08. The problem with this method is that there is double factoring, with R being treated as both a resistance and an action, and the overall factor of safety is low.
3. Treating the characteristic side friction force as an action and applying $\square_{G,stb}$ resulting in an overall factor of safety of 1.11. The problem with this method is that it treats resistance as a action what is not logic. Also the overall factor of safety is low.
4. Applying the factor $\square_{\square'}$ to reduce □ but to increase □', so as to reduce K_a and hence provide a cautious estimate of $\square_h$', results in an overall factor of safety of 1.65, which is the largest obtained by all the methods. With regard to this method, it should be noted that, although □□is originally considered to be a function of □', the partial factor is applied differently to □□ and □'. This may be justified on the basis that the soil close to the wall providing the □□ value is different from the general mass of the soil providing the K_a value and hence it is appropriate to treat □□ and □' differently. The problem with regard to EN 1997-1 is that there is no provision in *2.4.6.2(1)P* allowing for a partial material factor $\square_M$ less than 1.0 to be applied to a characteristic soil strength value so as to increase the soil strength, □' when it acts unfavourably, as it does in this example when □' is used to calculate K_a. In the ENV version of Eurocode 7, however, there was a clause *2.4.3(10)P* which stated that *"For ultimate limit states in which soil strength acts in an unfavourable manner, the value of $\square_M$ adopted shall be less than 1.0".*
5. Another fifth method, not specifically mentioned in EN 1997-1 but proposed in the case of the side resistance on a pile in the *Designers' Guide to Eurocode 7* by Frank et al. (2004), is to treat $\square_{\square'}$ as a resistance factor and apply it to the characteristic $K_a$tan□ value. Naturally this results in an overall factor of safety of 1.25.

Conclusions Regarding Uplift

The examination of the solutions of the uplift design example show that most of the variations in the solutions were due to assumptions made regarding the soil resistance on the side walls of the basement. As shown in Table 2, if the resistance is calculated assuming the friction on the side walls is calculated from Ktan□, where □ and K are both functions of the same □', then applying a factor $\square_M$ to reduce □' (Method 1), does not provide any safety as the factored resistance is greater than the characteristic resistance. Treating $\square_M$ as a resistance factor and applying it to the characteristic value of Ktan□ (Method 5) provides a reasonable factor of safety of 1.25 on the resistance, whereas applying $\square_M$ to reduce □ and increase □', and hence reduce K_a (Method 4), which is consistent with the ENV version of Eurocode 7, provides a greater factor of safety of 1.65.

Investigation of the Eurocode 7 equilibrium equations and partial factors for designs against uplift has clarified the design principles for these situations. It has been shown that the use of the HYD partial action factors on the characteristic total stress and total pore water pressure in *Equation 2.9a* results in very conservative designs and is illogical. The partial action factors $\square_{G,dst\ t}$ and $\square_{G,stb}$ should only be applied to the characteristic destabilising action causing heave, which is the excess pore water pressure, and the stabilising effective stress, not the hydrostatic pore water pressure. Alternatively, although not in the current version of Eurocode 7, applying a partial action factor to the excess pore water pressure or an additional margin to the hydraulic head, with no factor applied to terms involving the weight of the soil or the water gives the same result as applying the HYD partial action factors to the seepage force and effective weight in *Equation 2.9b (*Orr, T.L.L , 2005).

Table 2. Different methods for obtaining the design resistance on a buried structure

Method No. and assumptions regarding $K\tan\delta$	$\gamma_{\phi'}$	γ_{δ}	$\gamma_{G,stb}$	γ_R	ϕ'	δ	K_a	$\tan\delta$	$(K_a\tan\delta)_d$	OFS
0. No factors	-	-	-	-	35.0	23.3	0.23	0.431	0.099	
1. Only $\gamma_{\phi'}$ applied to reduce ϕ' and δ	1.25	1.25	1.0	1.0	29.3	19.5	0.29	0.354	0.103	0.96
2. $\gamma_{\phi'}$ applied to reduce ϕ' and δ and also $\gamma_{G,stb}$ applied	1.25	1.25	0.9	1.0	29.3	19.5	0.29	0.354	0.092	1.08
3. $K_k\tan\delta_k$ treated as an action and only $\gamma_{G,stb}$ applied	1.0	1.0	0.9	1.0	35.0	23.3	0.23	0.431	0.089	1.11
4. Only $\gamma_{\phi'}$ applied to increase ϕ' and reduce δ	1/1.25	1.25	1.0	1.0	41.2	19.5	0.17	0.354	0.060	1.65
5. $K_k\tan\delta_k$ treated as a resistance and only $\gamma_{\phi'}$ applied as a resistance factor	1.0	1.0	1.0	1.25	35.0	23.3	0.23	0.431	0.079	1.25

In summary: (i) The uplift forces that are destabilising action should be equilibrated by the weigth of the structure added by resistance of the ground and the ground reinforcement due the piles; (ii) The partial action factors $\gamma_{G,dst}$ and $\gamma_{G,stb}$ should only be applied to the characteristic destabilising action causing heave, which is the excess pore water pressure, and the stabilising effective stress, not the hydrostatic pore water pressure; (iii) The wall friction angle, δ is set equal to $(2/3)\phi'$ in the majority of the solutions, in accordance with the maximum value stated in *9.5.1(6)* for precast concrete walls, while the others used; (iv) $\delta = \phi'$, which is the value that may be assumed for concrete cast against soil.

The examination of the solutions the uplift design example show that most of the variations in the solutions were due to assumptions made regarding the soil resistance on the side walls of the. Following the analyses of Table 2 the solutions 4 and 5 are more logical.

6 TRIAL COLUMNS

The scope of jet grouting trial columns is the following: (i) To verify the effectiveness of the system; (ii) To provide information on system parameters required to construct the column diameters; (iii) To monitor resultant movements and to assess the effect on adjacent services; (iv) To demonstrate preventative methods of potential grout intrusion; (v) To provide instrumentation of noise and vibration emissions both within ground and the sourroundings.

From past experience the following parameters were estimated (ASCE, 1997):

Drilling: Bit(hole) diameter, drilling slurry makeup;
Drill Settings Lift speed, rotating speed, number of lifts;
Monitor Nozzle/port sizes for all components injected, angle of the nozzle(s), number of nozzles;
Injection Volume and pressure of all components injected;
Material Method of mixing and material components and their concentration(s).

The scope of the jet grouting trials was:

Phase 1: Two jet grouts to determine local jet grouting construction parameters and measure achieved diameters in situ; Phase 2: Two columns to investigate the mass properties of a block of jet grouted and to obtain cores for subsequently laboratory testing.
Variety of methods to sample have been developed as shown in Table 3.
The results of 4 trial jet columns performed by the Contractor and of the three samples taken from column 4 are summarized in Table 4.

7 CONSTRUCTION METHODOLOGY AND EQUIPMENTS

The following construction sequences was adopted: (i) Pre borehole; (ii) Position the drill and jetting tools; (iii) Activate high pressure pump with water to check nozzle and grout line clearance; (iv) Lower the tube into the ground to the required depth; (v) When reaching the desired depth, activate cement slurry, adjust rotation and withdrawal rate, discharge value, following the recorded parameters, a pre-cutting with lense dense grout was used, and this methodology was repeated for the final treatment, continue withdrawal while forming the column; (vi) Shut off cement slurry line and quickly withdraw the grouting tube.

Jet grouting equipments has included (GEO-RUMO, 2011): (i) Drilling equipment LT 3nx/JTC with equipment of reading, sensors, laser set Diana equipment of placement, and softaware EXTIG; (ii) Grout Mixing and Injection Equipment: grout mixers, holding tanks, water tanks, air compressors and pump of high pressure 7T500J; (iii) Jet Grout pump and injection monitor with horizontal radial nozzles delivering high velocities fluids; (iv) Compressor capable of producing the pressure of 15 bar and to deliver the flow rate values; (v) Jet grouting parameters: pressure of the fluids, flow rat of the fluids, grout composition, rotational speed of the jet grouting, rate of withdrawal; (vi) Equipment- Instrumentation: pressure gauges, flow meter; (vii) Silo of 60 tf.

Table 3. Sampling and testing methods

Requirement	Sample Method(s)	Test Method(s)
Strength	Wet grab(in situ) cast in to molds Cast in place plastic pipe retrieved after cure Core drilling	Uncofined Compression Triaxial Tension Direct Shear CPT(in situ) if soft enough
Permeability	As above plus: Cast-in-Place Piezometer Drilled and cast Piezometer	Permeameter Rising or Falling Head (in situ) Packer Testing

8 QUALITY CONTROL /QUALITY ASSURANCE

Field test programs involve excavation and core drilling to extract samples to evaluate geometric, mechanical and permeability properties and to refine injection parameters.

Table 4. Results of 4 trial columns and samples

Column	**1**	**2**	**3**	**4**
Pre-cut	No	No	Yes	Yes
Grouting Pressure(bar)			250	350
Grout A/C			10	10
Density			1.06	1.06
Noddles (mm)			2x4	1x5
Grouting flow (l/min)			324	299
Upward velocity (cm/min)			21	17
Rotating velocity (min)			3	5
Air pressure (bar)			15	15
Jet grouting				

Theorethic consume (kg/m3)	500	600	500	600
Grout (A/C)	1	1	0.83	0.71
Density	1.5	1.5		
Injection pressure (bar)	350	450	350	450
Noddles (mm)	2x4	1x5	2x4	1x5
Grouting flow (l/min)	323	286	315	274
Upward velocity (cm/min)	19	14	21	17
Rotating velocity (min)	3	4	3	5
Air pressure (bar)	15	15	15	15
Duration of pre-cut	19	14	10.5	8.5
Diametre (mm)	<1800	<1800	>1800	>1800
Recovering r= 900 mm (%)			100	100

Sample	Column	Date	P_i (gf)	H_i (cm)	Di (cm)	$\sigma_{comp.max}$ (kPa)	E_{cl} (GPa)	$E_{cl}/\sigma_{comp.max}$
1	4	31-12-2010	2783.67	18.24	10.25	5557.5	4.73	852
2	4	31-12-2010	2802.45	18.79	10.26	4978.9	3.92	788
3	4	31-12-2010	3114.2	20.80	10.22	6903.8	4.69	679

The samples were submerged in water during 18 hours before the test.

The methodology associated to the correct actions related the execution of jet grouting is given in GEO-RUMO (2010). The quality control monitoring included collection of selected production data, pressure, volume, flow, drill rate, thrust and torque. The unconfined compressive strength of the jet grout columns determined by laboratory testing of core samples has lyed within the range of 2 MPa and 4 MPa, with 90% of values and within the range of 1 MPa and 6 MPa with 95% values.

The ground core recovery was greater than 95%. The permeability measured by insitu constant/variable head testing 90% of all results was less than 5x 10^{-8} m/s. Permeability tests were carried out on piezometers sealed into the cores holes. In addition the overall watertightness of the jet grouted structure plug was assessed by the piezometric readingsDelicate site management to avoid disturbance to the surrounding to reveal that the jet grouting was carried without causing nuisance (Osborne and Chiat, 2010). For most drilling holes the verticality tolerance is unlikely to be better than 1 in 100 and is tipically specified at 1 in 75.

Following EN 12716, 2001 reccommendations the following tests on the grout mix were performed:

- preliminary tests: density, marsh viscosity, setting time, unconfined compression tests on cylindrical samples (height/diameter ratio 2.0).

The Brazilian test was also performed on a cylindrical sample (H/D ratio = 0.5) submitted to compression along a diameter plane and is interpreted in terms of tensile strength of the tested material.

The control of jet columns deviations of verticality and intersection was performed by TIGOR system and software EXJTC.

The Contractor has submitted Engineer Daily Reports with the following information:
i) Jet grout element number; (ii) Time and date of beginning and completion of each grout element; (iii) Grout mix data including mix proportions and unit weight density measurements; (iv) Injection pressure of all fluids used to construct each grout element; (v) Flow rates of all fluids used to construct each grout element; (vi) Rates of rotation and withdrawal of jet rods for each grout element; (vii) Top and bottom elevations of the jet grout element; (viii) Continuous flow of spoils return; (ix) Assessment of column diameters.

9 MONITORING

The instrumentation plan has included piezometers, inclinometers, bench marks , fissurometers and clinometers.

Safety control has included the following tasks (Pinto, 2000): (i) A visual inspection of the foundation and neighbours; (ii) Regular measurements using instrumentation; (iii) Storage of the data; (iv) Interpretation of visual observation and processed data; (v) Issue of an evaluation of the safety of the installation; (vi) Decisive actions to be taken should safety have deteriorated; (vii) Interpretation of the measurements by a comparison of the currently computed data with reference values, or their transformation into standard conditions; (viii) Definition of a criteria of alert and the reinforcement actions.

10 SURVEY OF ADJACENT BUILDINGS

A survey of adjacent buildings has identified:
- Alfandega Building, 2 flooors - Old building from Portuguese time built around 1950.
- Navy Building - 2 flooors - Old building from Portuguese time built around 1950.
- Telecomunications Building - 4 flooors - Old building from Portuguese time built around 1950. Recently reconstructed building under the supervision of DAR:
-Police Building - 2 flooors - Old building from Portuguese time built around 1950.

Some measurements points were placed in these buildings to control the movements.

11 CONCLUSIONS

A summary of the main conclusions related the topics covered by this paper is presented subsequently:
i) The geological and geotechnical survey materialized by two site investigation campaigns was presented and discussed.
ii) Taking into account that the initial geotechnical characterization by Teixeira Duarte has not characterized the permeability properties of sandy silty clays materials underlying the bottom of excavation the Contractor has performed a pumping permeability test to address this issue.
iii) The design of bottom slab based in partial safety of factors proposed by the Contractor was addressed. It is important to stress that the proposed solution by the Contractor that combines the role of jet grouting plug, bottom slab and piles represents a CPRF (Combined Pile Raft Foundation) i.e. a composite foundation of 3 elements, namely pile, raft and soil that has proved very successful for the tall buidings with low settlements and economic costs.
iv) A detailed justification of the jet grouting plug position after weigthing the advantages and disadvantages of its location was presented. From the analysis it was reiterate the position of jet grouting at 1.8 m beneath the excavation level.
v) In addition some countermeasures were proposed, namely placement of piezometers to monitor water level, holes to pump water, injection grouting to control the leaks, excavations in segments, in order to face any detrimental situations.
vi) The Contractor has performed 4 trial jet columns to adjust the technique and three samples were taken from column 4 to assess the mechanical properties.
vii) A Quality Control and Quality Assurance program to verify the jet grouting columns properties was performed by the Contractor.
viii) A monitoring program was implemented in order to analyse the structural safety conditions with a plan of contingency actions.
ix) A survey of adjacent buidings was performed in order to monitor their behaviour during Kilamba construction foundation phase.
x) As the final purpose is to built Kilamba building with high level of safety it was shown that the project and construction respect the existent Codes and Specifications, the State of tha Art and the State of the Practice, as well several published documents.
xi) It was demonstrated that alternative project submitted by JETSJ-GEORUMO fulfills all necessary requierements and exhibits adequate safety.
xii) The jet grouting plug was calibrated through the analyses of case histories with the purpose to respect the rule of 6 P for the design “Proper Prior Planning Prevents Poor Performances”.

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