Seismic ground improvement: stone columns performance for a power plant in the southern alluvial plains of Guatemala.

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ABSTRACT: The southern alluvial plains of Guatemala adjacent to the Pacific´s subduction zone, has been a very high intensity seismic zone. The area is historically related to settlement problems due to peat layers and high ground water level. Also, there is liquefaction history in the area. This case study presents the complete process and sequence for the design and construction of a ground improvement using vibro replacement by means Stone Columns. It also presents the lessons learned during this process. The geotechnical investigation included CPT, SPT, undisturbed sampling, geophysical characterization and shear wave velocity profiling. The laboratory program included classification tests, grain size, moisture content, direct shear tests and consolidaton tests. Liquefaction assessment was performed using the criteria presented by Robertson and Wride in 1998. The treatment was defined based on the vibrocompactability chart modified from Brown, 1977. The monitoring was performed through survey monuments located in different points of the building using a total station and a laser level. A 7.4 magnitude earthqueake occurred on november 2012 after the treatment compleation. The settlements after this event are presented, as well as the general trend curve.

RÉSUMÉ : Les plaines alluviales du sud du Guatemala sont adjacentes à la zone de subduction du Pacifique, une zone sismique de très haute intensité. La zone est historiquement liée à des problèmes de peuplement en raison des couches de tourbe et du niveau élevé de l'eau souterraine. Il ya aussi l'histoire de liquéfaction dans la région. Cette étude de cas présente le processus complet et la séquence pour la conception et la construction d'une amélioration du sol en utilisant le remplacement de vibro par des colonnes de pierre de moyens. Présente également les leçons apprises au cours de ce processus. L'étude géotechnique comprenait le CPT, le SPT, la caractérisation géophysique non perturbée et le profilage de la vitesse d'onde de cisaillement. Le programme de laboratoire comprenait des tests de classification, la taille des grains, la teneur en humidité, les essais de cisaillement direct et les essais de consolidation. L'évaluation de la liquéfaction a été effectuée en utilisant les critères présentés par Robertson et Wride en 1998. Le traitement a été défini sur la base du tableau de vibrocompactability modifié de Brown, 1977. Le suivi a été effectué à l'aide de monuments d'enquête situés en différents points du bâtiment en utilisant une station totale et un Niveau laser. Un tremblement de terre de magnitude 7.4 s'est produit en novembre 2012 après la fin du traitement. Les règlements après cet événement sont présentés, ainsi que la courbe de tendance générale.

KEYWORDS: Alluvial plains, Guatemala, liquefaction mitigation, earthquake, stone columns, vibro replacement, settlements, CPT.

1 INTRODUCTION

The southern alluvial plains of Guatemala adjacent to the Pacific´s subduction zone, has been a very high intensity seismic zone. The area is historically related to settlement problems due to peat layers and high ground water level.

The slide collection about the 1976 earthquake, a 7.6 magnitude event (Plafker 1978), includes photographs that show the occurrence of liquefaction in different locations. Also, the report about the 2012 earthquake, a 7.4 magnitude (EERI-AGIES, 2013), presents evidence of liquefaction occurrence.

The stone columns technique had proven its effectiveness for settlement control/reduction, drainage enhancement and therefore seismic induced liquefaction risk mitigation (Adalier and Elgama 2004, Aguado et al 2011, Elias et al 2006) Even in presence of soft clays, the improvement characteristics of the stone columns have been studied and proved (Guetif et al 2006).

Based on the geotechnical scenario described above, the solution of stones columns as liquefaction risk mitigation solution was analyzed for a power plant foundation.

2 CONSTRUCTION OF GENOSA POWER PLANT

2.1 *Genosa Power Plant*

Genosa is a 12.4 MW thermal power plant. Consisting of a mai n structure with three power units (engines), auxiliary structures as cooling tower, electrical substation /switch yard and a storag e tanks area. Its construction started in march 2011 and finished by July 2013.

2.2 *Site Investigation and Characterization*

The regional geology consist of Quaternary alluvium (Qa) withi n the Maria Linda watershed. These soils, consisting of interbed ded, not consolidated and poorly drained layers of sands and silt s, and to a lesser extent gravels and clays, having erratic deposit ional patterns typical of coastal plains. The Quaternary alluvium (Qa), about 1,000 m thick, is underlain by rocks of the Tertiary and rock and sands of the Cretaceous, followed by Ophiolitic Ba sement.

The site investigation ranged from in situ testing to laboratory te sting program. The field testing and sampling program include d:

- 8 Cone Penetration Tests (CPTu) and
- 1 Seismic Cone Penetration Test (SCPTu)
- 1 Borehole (Rotary drill)
  - Undisturbed samples (Shelby tube)
  - Standard Penetration Tests

A summary of the CPT tests performed is shown in the Table 1.

Table 1. Summary of CPT testing program.

CPT Type and Number	Location / Structure	Depth (m)	Ground Water Table (m)
CPTu1	Engine # 3	2.8	1.3
SCPTu1a	Engine # 3	13.3	1.3

CPTu2	Engine # 2	24.7	1.3
CPTu3	Engine # 1	25.1	1.3
CPTu4	Stack # 1	23.3	1.3
CPTu5	Cooling Tower # 3	24.3	1.3
CPTu6	Farm Tank	16.1	0.5
CPTu7	Warehouse	25.1	1.3
CPTu8	Transformer	25.6	1.0

The laboratory testing program included:

- Classification tests (plasticity, moisture content, grain size analysis)
- Direct shear
- Consolidation

Grain size analysis is very important for solution suitability analysis (Brown 1977). A summary of the testing program is presented in tables 2 and 3.

Table 2. Laboratory Test Program Results Summary.

Sample number	Depth (m)	Natural moisture content (%)	Wet density (kN/m^3)	Cohesion (kPa)	Internal Friction Angle
S-1	4.95-5.60	53.4	16.8	42.16	20°30'
S-2	8.00-8.75	69.0	14.0	23.96	26°00'

Table 3. Laboratory Test Program Results Summary.

Sample number	Depth (m)	Wet unit weight (kN/m^3)	Consolidation Test (According ASTM D2435)			
			Pre-consolidation Pressure (kPa)	Voids Ratio e_o	Specific Gravity	Compression Index cc
S-1	4.95-5.60	15.47	578.79	1.81	2.67	0.42
S-2	8.00-8.75	15.04	529.74	2.12	2.71	0.79

2.3 *Simplified Soil Profile*

From the collected data a simplified soil profile, SSP, was determined.

		Elev. 0.00
1 – Clayey Sandy Gravel		
	q_t = 5 to 20 MPa	
	R_f = 0.5 to 1%	Elev. -6.00
2 – Sandy layer with Silty and Clayey Lenses		
	q_t = 1 to 5 MPa	
	R_f = 1 to 5%	Elev. -10.00
3 – Silty Sand and Sandy Layers with Silty and Clayey Lenses		
	q_t = 0.5 to 4 MPa	
	R_f = 0.5 to 1%	Elev. -15.00

2.3 Risk of *Liquefaction Assessment and Mitigation*

The liquefaction risk was assessed determining the ratio between Cyclic Resistance Ratio, CRR, and Cyclic Stress Ratio, CSR using the methodology presented by Robertson and Wride in 1998 for CPT. The earthquake magnitude used was 7.6 as per 1976 earthquake.

Based on the geotechnical scenario described above, the solution of stones columns as liquefaction risk mitigation solution was determined. The proposed inclusion factor was 10% by means 0.95 m diameter columns distributed in a 2.70 square grid. The columns depth was set up to 20 m. An important feature of the solution is the inclusion of an upper fill layer that works as working platform but most important as a load spreading layer.

2.3.1 *Settlement Calculation*

The estimated load of engines and structures is 60kPa.

Settlement with no improvement:

Average tip resistance q_c in silty layers: 1.4 MPa

Ratio E_{oedo} / q_c = 4.0 as per Menard 1998; Cassan1988 (1)

E_{oedo} = 5,600 kPa
Average tip resistance q_c in sandy layers: 15 MPa
Ratio E_{oedo} / q_c = 3.3
E_{oedo} = 50,000kPa
Total settlement:
W_{total} = 60 kPa x (10.00m / 5,600kPa + 15.00m / 50,000 kPa) = 12.0 cm

Settlement with improvement:

$E_{improved} = [IF^* E_{stone\ column} / E_{in\text{-}situ\ soil} + (1- IF)] * E_{in\text{-}situ\ soil}$ (Priebe 1995) (2)

One assume an average ratio $E_{stone\ column} / E_{in\text{-}situ\ soil}$ of 8

Therefore, with a 10% inclusion factor the ground improvement factor is 1.7

In addition, a 50% improvement of the surrounding ground is taken into account because of the sandy and silty natures of the material

The overall and global ground improvement factor that can be assessed is thus: 1.7x1.50 = 2.55

Settlements with no ground improvement / ground improvement factor = 12.0 / 2.55 = 4.7 cm

This settlement value was presented to the structural engineer of the project to be considered in the design as the maximum differential settlement.

2.3.2 *Load Bearing Capacity*

The pressumeter method appears to be the most suitable approa ch for determining the load bearing capacity. It was estimated th at the load will be mostly distributed in the upper 5.00 m. These first meters are dense sandy material and therefore the followin g rules can be applied

Ratio between cone penetration test resistance (qc) and pressur emeter resistance (pl) in sandy to clayey silt material:

q_c/p_l = 10 (3)

Average tip resistance to reach after ground improvement of 8M Pa:

qc = 8 MPa which leads to p_l = 0.8 MPa

$$q_{adm} = \frac{p_l x1{,}2}{3} = \frac{0.8\,x1.2}{3} = 0.320\ MPa\ = 320\ kPa \quad (4)$$

The estimated load bearing capacity exceeds the required minimum capacity of 200 kPa established by the structural engineer of the project.

2.3.3 *Trial Area*

As part of the design, a trial area was defined to be carried out prior to the start of the production stage. This trial enabled to establish which amperage of the electric motor of the vibroflot must be reached during the compaction procedure in order to achieve the necessary gravel consumption in the various types of layers to obtain the required compaction and/or the required gravel inclusion factor. Figure 1 shows trial area configuration and pre and post CPTs location.

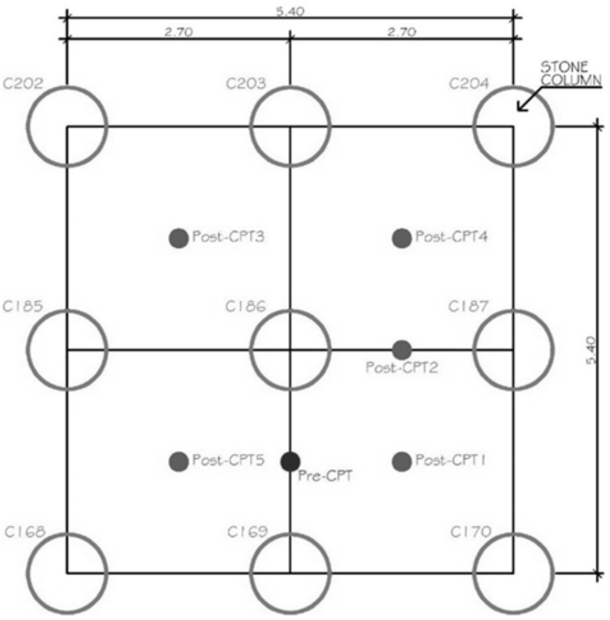

Figure 1. Trial area configuration.

2.3.4 *Construction*

After analyzing the ground conditions, the ground water level a nd equipment availability, the wet top feed method was selected. The first step is to prepare the working platform. This is very important in order to avoid accidents and allow a proper drainage of the area. The working platform elevation was set at +0.50 m, at least 1.50 m above ground water elevation. The working methodology can be described in 3 main steps. The vibroflot is penetrated to the required depth under the combined effects of its own weight, vibration and the jetting action of air, water or both. The vibroflot is then lifted by 0.50 to 1.00 m, and coarse gravel or crushed stone is tipped into the hole. The vibroflot is then re-penetrated to within a short distance of the previously penetrated depth. The filling / compaction cycle is then repeated in upward increments up to the working platform level or up to the upper level of the soft silty ground layers or lenses.

Gravel consisted of elements within the range 20-60 mm of crushed stone with no more than 2% of material out of range. The rate of gravel supply was critical to ensure continuity of works. In order to guarantee material supply and ease construction process gravel was stored in two different sites of the project.

3 MONITORING AND PERFORMANCE

3.1 *Monitoring*

In order to verify and monitoring the long term performance of the solution, 8 survey points were marked around the motors warehouse and an external reference was established using a survey monument. The points are shown in the Figure 2.

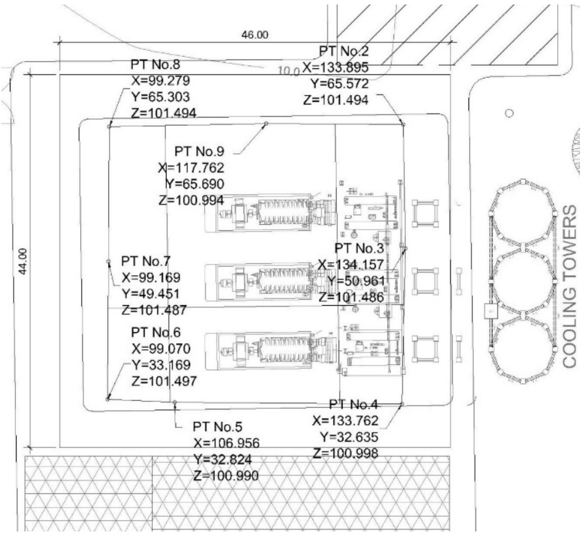

Figure 2. Survey Points Location.

Readings were performed using Topcon DL 102 Electronic Digi tal Level and a Trimble DTM 322 Total Station. The dates of the readings are presented in Table 4.

Table 4. Monitoring Program.

Reading Number	Reading Date (dd/mm/yy)	Comment
0	10/10/2012	Reference reading
1	12/04/2012	After Earthquake reading
2	12/05/2013	
3	12/08/2014	
4	06/01/2015	

3.2 Performance

The maximum settlement recorded was 14 mm (Point 4), that is the 30% of the maximum settlement estimated, 47 mm. The corner points showed the greater settlements with a trend of settlement to SE. The middle points of the building showed the smallest settlements, Points 5 and 7. A maximum settlement difference of 12 mm between Point 4 and Point 5 is probably absorbed by the structure without perceptible signs of movement.

Figure 3 shows the graph of the total settlement of each point and Figure 4 shows the graph of the incremental settlement, both graphs shows a trend that no major settlements occurred after the third reading, about a year after the Power Plant operation began. It is important to mention that on 7th of November of 2012 an Earthquake of 7.4 Magnitude occurred, the epicenter was located in the Pacific coast of Guatemala at about 70 km from the site. During this event liquefaction was observed in Champerico and Ocos, both seafront communities as well as in San Pedro, San Marcos the largest town near to the epicenter.

The maximum settlement recorded after the November 7th earthquake was 4 mm. The construction of the plant was almost completed at that time, but the plant was not fully operational.

Despite that some settlements were recorded after the earthquake, the largest settlements occurred after the initial operation period. In general, no perceptible settlements or movements were observed in the structure as well as no cracks or fissures. The motors are very sensitive to tilt, particularly their axis. The motors presents no sign of movement of misalignment.

The post-CPT testing suffered of refusal (by means qt larger than 20kPa) at an average depth of 2.00 m. This condition suggest that the compaction of the upper layer was more effective than initially estimated.

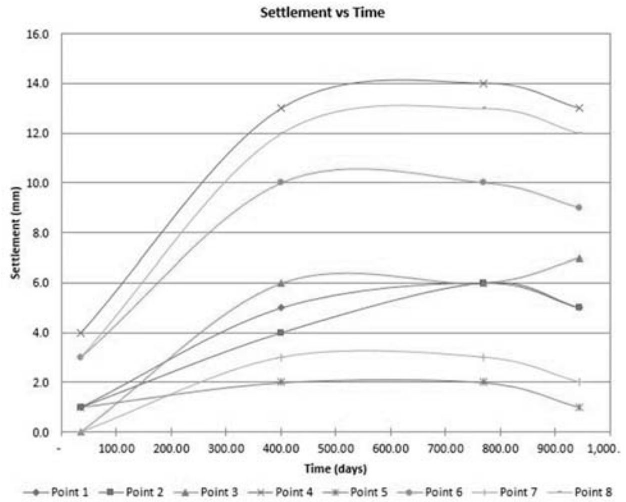

Figure 3. Settlement at each point

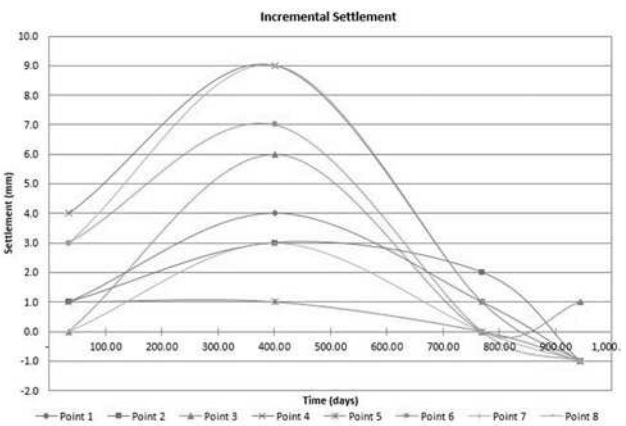

Figure 4. Incremental settlement at each point.

4 CONCLUSION

The settlements of the engine warehouse of Genosa power plant reached an average of 7 mm, about 15% of the initially estimated settlement calculation, with a maximum value of 14 mm. The greater settlement values were observed in the corner areas suggesting a distention or confinement loss in these areas.

The after earthquake settlements are relatively small seemly like a general rearrangement of the ground, no damages were observed due to this event. The largest settlements occurred after the initial testing and operation stage, suggesting a greater influence of the engines vibration. The settlement increment after the initial operation stage are negligible, even observing very small elastic rebound.

The clayey layers seems not be critical in the performance of the foundations. The interbedding of clay and sand layers probably allowed a rapid the consolidation of the upper layer during construction this due to its reduction of the drainage paths. This consolidation can be observed in the post-CPT testing and probably improved the performance of the foundations.

It is highly recommend consider additional drilling methods for post-CPT testing in order to pass through highly densified layers.

The performance of the stone columns as liquefaction mitigation and ground improvement technique in the alluvial plains of Guatemala was extremely satisfactory. The interbedded layers of sand probably increased the effectiveness of the solution.

5 ACKNOWLEDGEMENTS

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