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METHOD FOR FORMING A STABLE FOUNDATION GROUND: THE LOGES 3 PROJECT IN SAINT-NICOLAS QC, CANADA

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ABSTRACT

Uniform soils cannot be stabilized by known methods such as dynamic compaction, vibro-flotation, stone columns or piling. Since 1966 authors has confirmed the liquefiable behavior of dense uniform soils under cyclic loading [1] and more recently by centrifugal testing [2, 3, 4]. Survey inspection reports from various post-earthquake sites [5,7] confirmed the liquefaction of uniform soils and impacts on land and/or structures stability. It is therefore imperative to apply an efficient stabilization method [6] that addresses the disadvantages of soil uniformity, guarantees stability against liquefaction and offers an improved bearing capacity.

LOGES 3 is a 7-story concrete building plus a basement being developed over a 4700 sq. m. area (Figure 1). A 2017 GHD soil investigation report exposed liquefiable layers of uniform fine sand and/or silt in the upper 6 to 7m of soil and a high-water table. Compacsol's method was retained to ensure a solid foundation with a 300 kPa (SF=3) bearing capacity and 25mm of maximum settlement. Soil conditioning to obtain a graded soil-mix, including the removal of undesirable organic soils, was followed by densification using high-performance vibrating plates. The conditioned and compacted soils of good quality, which replaced the above liquefiable soils across the site, are graded and homogenous. The conditioned soils possess a high shear strength, a high bearing capacity and a reduced compressibility. They offer high safety factors for ground stability against liquefaction that meet significant strong-motion requirements.

Reduced depth of stress transfer from building loads is a direct advantage of having a higher bearing capacity. Stress transfer depth can be further reduced by the installation of compressible polystyrene strips over the conditioned soil prior to pouring the concrete foundation. The polystyrene strips minimize the width of concrete directly in contact with the soil, and thereby reduce the depth of stress transfer [6]. Observations during construction and performance testing are presented and discussed.

Keywords: Conditioned soil, homogeneous soil, deep densification, seismic stabilization, high bearing capacity.

INTRODUCTION

Compacsol has experience on several projects in the use of conventional dynamic compaction. Two of those projects were for the Port of Montreal and for Hydro-Quebec in Trois-Rivières, both requiring compaction to a depth of 10m and requiring results of $(N1)_{60cs}$ equal to or greater than 15. The soils at the Port of Montreal were a hydraulic fill mainly composed of uniform fine sand and/or silt with occasional layers of uniform small gravels. In the case of Trois-Rivières, there were alluvial deposits of uniform fine sand and silt in varying proportions. During borehole performance testing, the desired results could not be met despite the use of high energy compaction and large surface settlement. We observed that the lower $(N1)_{60cs}$ values corresponded to layers of soil with uniform gradations in either fine sand, silt or even small gravels. Dewatering and additional densification did not significantly improve the results at either job site. Following those experiences, we recognized that conventional dynamic compaction methods were unable to provide safe seismic ground stabilization for uniform soils and decided to stop using such methods of compaction in those soil conditions.

Searching for new means to achieve a safe stabilization method for uniform soils, it became obvious that prior conditioning followed by densification of the soil were necessary steps to attain the goal of seismic stability. We were subsequently able to provide viable options for our Clients.

The thickness of uniform soil deposits can vary at depth and across a given area. The overburden may contain a combination of uniform granular layers and soft to firm sensitive clay layers, located either near the surface, at mid-depth or below the uniform soil deposits. These conditions present obstacles to the use of spread foundations due to the risk of excessive total or differential settlements.

Structures foundations may be built at various elevations and be of different types such as footings, rafts, piles, deep walls and others, selected according to soil behavior analysis.

Failures of piles during earthquakes due to the liquefaction and spreading of uniform soils has led to numerous collapses of structures, including buildings, bridges, overpasses and ports [4,7]. Liquefaction has also led to damages of industrial piping, structural installations [8], and hydraulic structures including reservoirs, dams and containment dykes [9]. Lifeline installations are also very vulnerable to settlement and spreading displacement of uniform soils during earthquakes [10].

Extensive research on earthquakes points to the need to assess the reliability of existing methods of stabilization and to understand their limitations, while also considering their efficiency and how they could be improved. Furthermore, it is important to understand the large variation of earthquake magnitudes and PGA accelerations across the globe, and the related risks, damages and challenges to mitigate them.

In the Lévis, Quebec, location of the Loges 3 project, the maximum expected earthquake magnitude is Em = 6.5 and the PGA value is 0.28 g, which is significant considering the nature and properties of the natural uniform soils. These uniform soils exist not only in Lévis, but across vast areas of the province of Quebec and beyond.

The most violent and destructive earthquakes recorded in other countries have reached Em = 9.2 and PGA = 3.2 g.

Achieving deep conditioning and densification, from existing soil strata, presents several geotechnical challenges. Ensuring the stability of deep excavations and of surrounding properties to prevent failures or deformations requires a good geotechnical knowledge of loose soil behavior, groundwater impact and the development of adapted methods to secure them. The use of wellpoints dewatering systems, temporary shoring and/or ground sloping, and perimeter conditioning and compaction are all techniques that have become essential parts of our expertise and need to be evaluated and applied, where warranted.

The current method for seismic soil stability assessment -- The NCEER/NSF workshop on evaluation of liquefaction resistance of soils [11] developed a method for predicting the liquefiable behavior of sites, based respectively on the corrected blow-count $(N_1)_{60}$, the cyclic stress ratio CSR and the cyclic resistance ratio CRR at 5%, 15% and 35% of fines, at various successive depths. (The ratio of CRR/CSR represents the safety factor against liquefaction).

When the point of intersection of $(N_1)_{60}$ and CSR coordinates falls to the right of the corresponding CRR curve, that point confirms its seismic stability; if the point is to the left of the corresponding CRR curve, it indicates seismic instability, as shown on Figure 2.

The NCEER/NSF method does not integrate the gradation distribution trend of soil (uniformity or graded) into the evaluation of liquefaction resistance stability. Research has, however, confirmed that uniform saturated soils, even when tested in a dense state [2,3], have a great sensitivity to liquefaction.

Although, the low corrected blow-counts $(N_1)_{60}$ obtained in certain narrow-uniform soils do not qualify as seismic-stable soils, other less narrow-uniform soils gradation could yield higher blow-counts, that could meet the NCEER/NSF criteria for stability against liquefaction, might still liquefy when subjected to dynamic loading. It is therefore essential to revisit the evaluation of liquefaction resistance of soils for the specific cases of soil gradation, void ratios and particle shapes.

LOGES 3 PLANNING AND CONSTRUCTION

Preliminary field testing -- GHD 2017 geotechnical study identified liquefiable layers of uniform soils in the upper 6 to 7 m. GHD CPT soundings had encountered refusal at those depths.

A series of 7 CPTu soundings carried out, by Compacsol, to greater depths across the site indicated that below 6 m depth, liquefaction would not be probable based on Iwasaki (I_L less than 5) and that the dynamic settlement would be less than 5 cm. It was thus decided to condition and stabilize the upper 6 to 7 m layer below the original grade.

Georadar survey – Detection of an organic buried bed -- During construction we ordered a georadar survey to confirm and delineate the organic presence discovered in an exploration trench. The georadar survey identified a significant buried area of organic material, shown on Figure 3; this material was removed during the conditioning phase.

Compressible polystyrene strips installation under the raft's imprints -- The natural soil below the conditioned fill as indicated by the CPTu included stiff clay or fine over-consolidated silt and potential variations. For the large foundations, we recommended the installation of compressible polystyrene strips, in order to reduce the width of the concrete foundation in direct contact with the conditioned soil and as a result decrease the depth of stress-transfer (Figure 4). Picture 11 show the compressible strips under raft #1 reinforcement.

Wellpoints dewatering -- A wellpoints dewatering system was installed all around the site to help stabilize the excavation slopes by lowering the water table and helping dissipate the excess pore pressure generated by the densification.

Natural uniform soils from the LOGES 3 site -- The LiqIt analysis from the CPT soundings on the original natural uniform soils showed insufficient CRR ratios relative to the corresponding SCR ratios and confirmed its seismic liquefaction potential under the expected regional earthquakes for Saint-Nicolas, Lévis. The natural soils exposed during excavation are presented on Picture 1.

Soil conditioning -- During the excavation our team removed the undesirable soils including the organic concentrations revealed by the georadar survey or exposed by the excavation. (Pictures 2, 3). Soil conditioning was carried out by excavating down 6 – 7 m, selecting the fill we wanted to mix with an imported crushed stone (0 – 56 mm) and mixing them together (Figure 5, Picture 4). Picture 5 show the exposed steep excavation face in earlier conditioned soil.

Soil densification -- Conditioned soil was densified by means of high-energy hydraulically activated vibratory plates (Pictures 6,7). Densification was repeated during construction until excess pore pressures were expelled, and plate refusal was obtained

LOGES 3 FIELD PERFORMANCE RESULTS AND SITE OBSERVATIONS

Cyclic TxSS test on site original uniform silty sand sample -- The University of Sherbrooke carried out a cyclic TxSS test on a sample from the original LOGES 3 uniform silty sand. The cyclic resistance ratio CRR decreased from 0.15 after 2 cycles to about 0.08 after 20 cycles to liquefaction (Ru = 0.9) confirming a high susceptibility to liquefaction for the LOGES 3 original soils. The detailed Cyclic TxSS results will be presented in a next publication.

Gradation curves from the conditioned soil -- A series of sieve analyses and sedimentation tests, carried out on samples of the conditioned soils recovered from the performance boreholes FP-C, FP-D and FP-E are presented on Figure 6.

The gradation curves confirm a relatively homogeneous and graded mix with properties of low permeability and good internal stability. Hydraulic fracturing induced by the high energy densification caused excess water to surface without forming the typical fine particles volcanoes. The relative absence of volcano development is a good indicator of the conditioned soils stability against liquefaction.

Modified Proctor density results -- A modified Proctor compaction test carried on the conditioned soil yielded a maximum density of 2066 kN/cu.m. and an optimum water content of 7.8% (Figure 7).

Water transmission through the undrained compacted conditioned soil -- A falling head infiltration test through an unsaturated conditioned soil sample, 10.23 cm high, that had been densified at optimum moisture content of 7.8%, was started with an original water head of 10.27 cm. The computed infiltration coefficient was 8.26 x 10-7 cm/s over a period of 22.4 days, corresponding to a final water head of 8.87 cm. The transmission coefficient is indicative of a low permeability soil, relative to its original environment and to the soil present under the conditioned and densified fill.

Progressive evolution of the conditioned soil from a saturated to an unsaturated state -- We have observed in trenching, borehole testing and repeated densifications that the densified conditioned soil which is initially saturated becomes progressively unsaturated and hard (Pictures 8, 9). The density gained through several periodic densifications and the graded distribution contributes to the development of very small pore diameter. It is believed that the outflow from the excess pore water pressure is followed by a natural osmotic process or partial drying [12] that can reach a significant depth in the small diameter capillaries within the conditioned soil This behavior is of great interest, as it means that the water level will drop like in natural dense and low permeability till deposits, often not saturated to significant depths and not sensitive to liquefaction.

Additionally, the water transmission coefficient is so low that a rise of water level through the low compressibility conditioned soil during an earthquake is unlikely.

The rigidity and strength built-up during the works following the dissipation of excess pore pressure -- As the excess pore pressures dissipate and the osmotic process of partial dewatering through a vapor phase begins, the conditioned soils become more rigid and hard. This process of soil hardening continues until equilibrium is reached between the osmotic process and the slow inflow of water from the building's perimeter and from below.

Efficiency of the multi vibratory plate densifications -- The first compaction may cause the vibratory plate to sink about 1.5 m in 2.5 m new conditioned soil, it will not sink as much during the next stages of densification and will meet refusal with no settlement at the end of the process (Pictures 6, 7). Picture 10 shows that a heavy lift did not leave tracks on the hard surface.

The repetitive plate densification in successive stages allows for a denser state, than would otherwise be possible with other methods including conventional dynamic compaction.

Performance boreholes pressuremeter testing and expected bearing capacity and settlement assessments -- A series of 5 boreholes FP-C to FP-G with pressuremeter testing were carried out to assess the bearing capacity performance for square and continuous footings. The results for continuous footings are listed on Table 1.

The performance bearing capacity of 300 kPa (3 bars) with a S.F. = 3 is exceeded significantly and the expected settlement of 0.4 to 0.8 cm is well below the 2.5 cm criteria.

Preliminary CPT testing shortly after soil conditioning and densification – Results from early GHD CPT soundings (CPT-3) after densification indicated clay for conditioned soil layers, where excess pore pressures were not yet dissipated. Trench verifications showed no clay layers as also observed on adjacent later Compacsol CPTu-F-1, to be detailed in a next publication.

CPTu testing and liquefaction analysis through the conditioned soil and lower part of the soundings -- In 2018 a series of preliminary CPT by GHD followed by Compacsol performance CPTu enabled the liquefaction stability analysis of both the conditioned soil and the whole soundings length by means of the Geologismiki LiqIt software with selection of Robertson (1998) analysis and fines correction methods [13]. The selected earthquake parameters specified by GHD are Em = 6.5 and PGA = 0.28 for the LOGES 3 site. The water table was chosen at a depth of 1 m from the foundation level, approximately 3 m below the original ground surface. Table 2 summarizes the findings from GHD preliminary CPT from July 4 and 5, 2018.

The dynamic settlement varies between 2.08 and 5.72 cm and the Iwasaki overall potential I_L varies between 1.68 and 5.03. These results indicate that liquefaction is not probable under the specified earthquake.

Considering the 3-dimensional continuity of the stabilized ground, a local liquefiable pocket should not be of concern, as it would be surrounded and confined by the stable mass of conditioned soil, that would protect it against flow-liquefaction.

Table 3 summarizes the findings from Compacsol performance soundings respectively dated September 6, 2018 and November 19, 2018. The dynamic settlement varies between 0.0 and 4.95 cm and the Iwasaki overall potential I_L varies between 0.0 and 3.95. These results indicate that the liquefaction is not probable under the specified earthquake.

Verification of the stability of the conditioned soil under strong motion earthquake -- Simulations were run using the LiqIt software for dynamic settlement and Iwasaki overall potential $I_{L,}$ on the conditioned soil from CPTu-31-3-1 recorded between depths 1.2 m to 4.2 m on November 19, 2018. A water level at 0.2 m was assumed and varying combinations of earthquake magnitudes and PGA accelerations were applied to test the limit efficiency of the LOGES 3 conditioned soils. A total of 3 simulations were run and the results are listed on Table 4. The safety factor against liquefaction would be equal to or greater than 1.0 for earthquakes of magnitude of Em = 8.0 and PGA = 1.5 g and to one for Em = 8.5 and PGA = 1.25 g. In both cases the settlement found are 1.73 and 1.8 cm and the Iwasaki overall potential is zero indicating no liquefaction and a stable soil under these 2 strong earthquakes.

A third simulation was for an earthquake magnitude of Em = 8.0 and a PGA = 2.0 g; in this last case the safety factor against liquefaction is about 0.75; although the dynamic settlement was only 3.27 cm, the Iwasaki overall potential is 8.38 with a probable liquefaction.

The strongest known earthquake has been recorded at Em = 9.2 with PGA = 3.2 g. It is our opinion that a conditioned soil may be designed to safely meet this level of earthquake.

CONCLUSION

A method of conditioning and densifying uniform soils to yield a high seismically stable ground and to deliver far greater bearing capacity, shear strength and rigidity, as well as to reduce the settlement and displacement of foundations under earthquake forces is now available. Using these stabilization techniques, great engineering challenges can be overcome to sufficiently protect installations to the benefit of the public.

A major Canadian project for an oil pipeline through the western provinces of Alberta and British Columbia is presently on hold, as opponents to the project claim that it represents a significant threat to the environment. The pipes may leak or break during earthquakes due to poor uniform soil stability. That threat would be improbable if our method of soil conditioning and deep densification was applied, where needed, along the length of the pipeline foundation, preventing any spreading or settlement. The same reasoning applies to every other structure where concerns of failure or excessive deformation could be replaced by safety success, loss prevention and durable economical gains.

The concept of installing compressible polystyrene strips on the conditioned soil prior to pouring wide foundations allows to contain the main stress transfer within the dense conditioned soil and to limit significant stress transfer to lower compressible non-liquefiable layer such as slightly over-consolidated clay.

Conditioning of uniform soils is the right choice for people wanting to raise the level of safety and prevent disaster. It also helps mitigate damages from the destructive power of future earthquakes.

Additional research on the monitoring of conditioned soil dewatering from suction potential and evaporation is a subject of great interest and may be used as a mechanism for seismic mitigation.

We plan to design and install instrumentation in future projects to monitor and track the evolution of soil saturation levels. We can use that data to create a model for predicting the final stabilized water level in the conditioned soil.

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Aerial site view of LOGES 3 excavation and densification works

Picture 1 - Excavation through natural liquefiable deposit

Pictures 2 & 3 - Removal of buried ancient forest organics

Picture 4 - Soil conditioning underway

Picture 5 - Exposed steep excavation face in earlier conditioned soil

Picture 6 - Conditioned soil densification - First stage

Picture 7 - Densification meets refusal on last stage at the right side

Pictures 8 & 9 - Unsaturated SPT samples

Picture 10 - Hardened soil after maturation - No heavy lift tracks left

Picture 11 - Raft #1 construction - Polystyrene strips under the rebars

Figure 1 - LOGES 3 Imprint with performance boreholes and soundings location

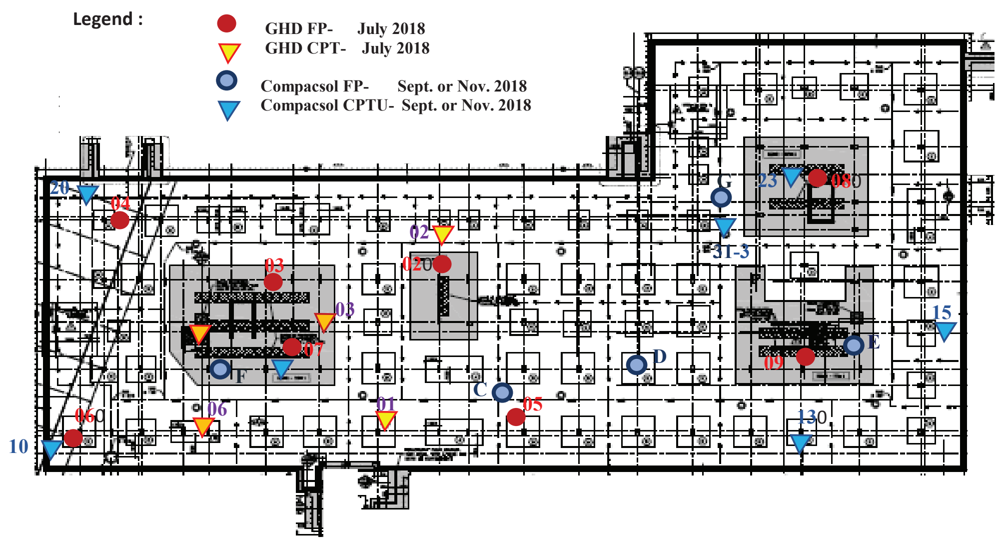

Figure 2 - Present NCEER liquefaction resistance of soils

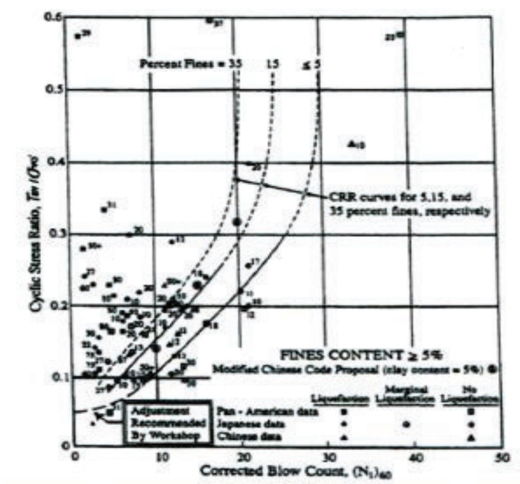

Figure 3 - GPR Georadar survey plan

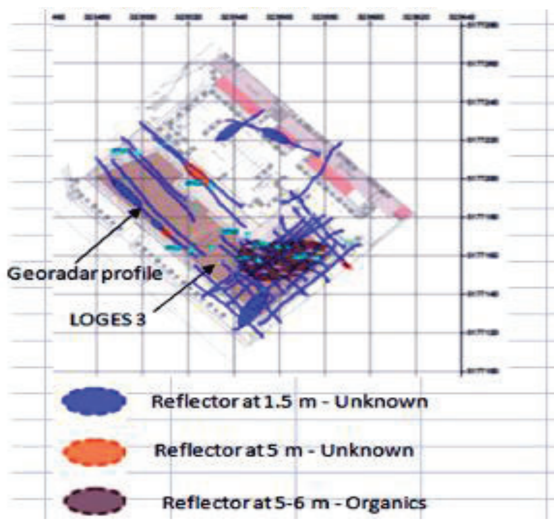

Figure 4 - Large raft foundation # 1 with compressible polystyrene strips between concrete and conditioned soil
Reduced stress transfer depth due to increase of bearing capacity and smaller foundation width in direct soil contact

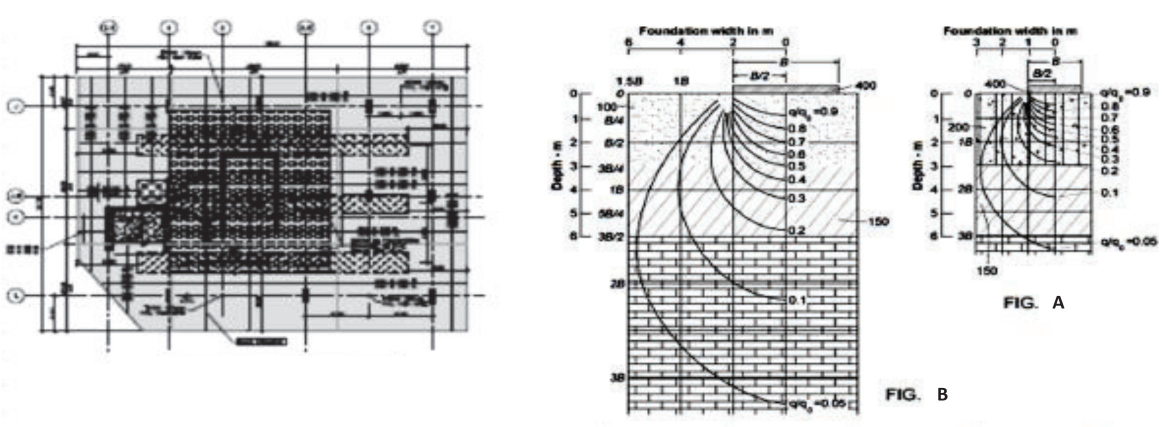

Table 1 - Bearing capacity and settlements
Compacsol Boreholes - Footings 3 m x 10 m

Borehole #	Date 2018	Bearing capacity - bars S.F. = 3	S.F. = 2	Settlement cm
FP-C	26/09	3,7	5,6	0,6
FP-D	26/09	4,3	6,4	0,6
FP-E	26/09	5,2	7,9	0,8
FP-F	19/11	7,3	10,9	0,4
FP-G	19/11	8	11,3	0,4

Performance requirements bearing capacity = 3 bars, settlement = 2.5 cm

Table 2 - Seismic stability analysis from GHD CPT soundings
Em = 6.5, PGA = 0.28 g

CPT #	Date 2018	Length m	Settlement cm	Iwasaki I_L
CPT-P-01	4/7	5,9	5,72	5,03
CPT-P-02	4/7	6,0	4,72	3,99
CPT-P-03	5/7	6,0	4,78	4,70
CPT-P-04	5/7	4,6	2,68	2,68
CPT-P-05	5/7	6,6	2,21	1,81
CPT-P-06	5/7	4,4	2,08	1,68

$I_L < 5$ Liquefaction not probable

Table 3 - Seismic stability analysis from Compacsol CPT soundings
Em = 6.5, PGA = 0.28 g

CPT #	Date 2018	Length m	Settlement cm	Iwasaki I_L
CPTu-AP-10	6/9	2,4	0,00	0,00
CPTu-AP-13	6/9	4,1	0,94	0,41
CPTu-AP-15	6/9	4,3	1,49	0,92
CPTu-AP-20	6/9	6,0	0,45	0,11
CPTu-AP-23	6/9	10,0	4,95	3,95
CPTu-AP-31-3	19/11	5,2	0,95	0,61
CPTu-AP-32-1	19/11	6,7	2,08	0,67

$I_L < 5$ Liquefaction not probable

Table 4 - Strong motions simulations on conditioned soil from CPTu-AP-31-3
Between depths of 1.2m to 4.2 m from November 19, 2018

Em	PGA	Length m	Settlement cm	Iwasaki I_L
8	1.50 g	3,0	1,80	0,01
8,5	1.25 g	3,0	1,73	0,00
8	2.0 g	3,0	3,27	8,36
6,5	0.28 g	3,0	0,00	0,00

$I_L = 0$ No liquefaction
$I_L < 5$ Liquefaction not probable
$I_L < 5$ Liquefaction probable

Figure 5 - LOGES 3 Gradation curves from original soils - GHD soil report 2017

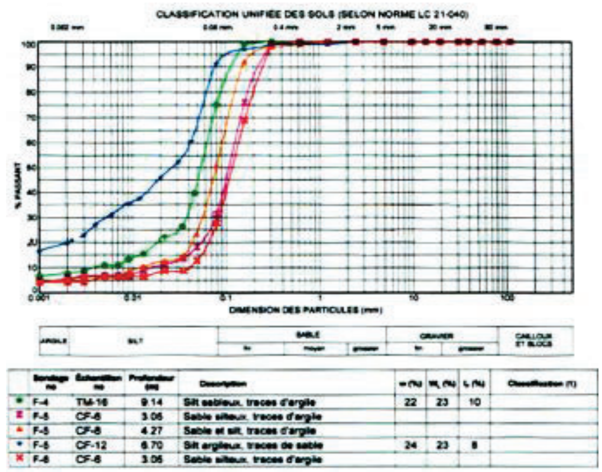

Figure 6 - LOGES 3 - Compacsol performance boreholes samples from conditioned soil
Gradation and sedimentation curves by GHD for Compacsol - 2018

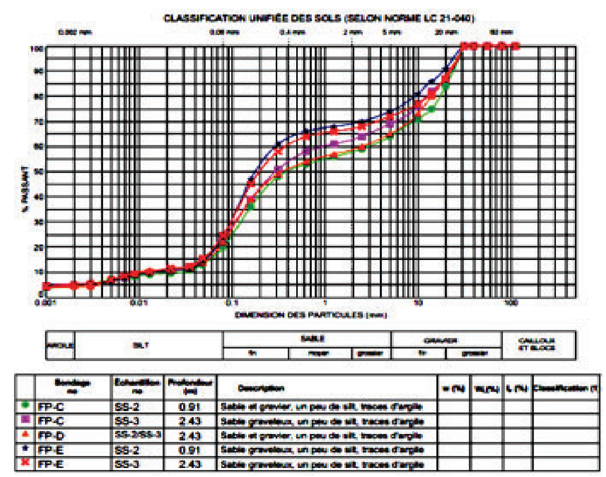

Figure 7 - LOGES 3 - Modified Proctor curve from conditioned soil samples by GIE