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# Jet Grouting Columns operating as reaction platform for building uplift and soil induced liquefaction mitigation

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ABSTRACT: As a consequence of foundation settlement observed in one of the most important buildings of Christchurch - New Zealand, due to the 2010 and 2011 seismic events occurred in that city; repairs have been made enabling the Christchurch Art Gallery building to reach its original level. Building uplift consisted on the use of soil injection techniques, providing "in-situ" soil reinforcement with grout material and an increase of soil volume by soil fracture. This solution is denominated as "jack on grout" (JOG). The present article describes the adopted soil improvement solution underneath the existent building, using jet grouting columns. The mentioned technique has been used to provide an increase of soil stiffness and strength, enhancing sufficient soil reaction under incremental stresses imposed by JOG during the structure uplifting works. In addition, jet grouting was considered to be an added value on long term soil behavior regarding soil induced liquefaction mitigation.

KEY WORDS: Ground improvement, Jet Grouting, Building Uplift, Liquefaction.

## 1 INTRODUCTION

The 2010 and 2011 Canterbury earthquakes, with magnitudes of 7.1 and 6.3, respectively, struck the South Island of New Zealand causing significant generalized damages, particularly in the city of Christchurch, New Zealand's second largest city. Significant liquefaction affected the eastern suburbs.

The 2011 earthquake and aftershocks, conduced to loss of human lives, making it the second-deadliest natural disaster recorded in New Zealand.

Governmental Authorities, together with the geotechnical community, are presently developing and implementing requalification plans to reinforce existing structures and rebuild those which were severely damaged or had collapsed.

The Christchurch Art Gallery has been affected by the earthquake sequence that occurred between 2010 and 2011, in particular suffering differential settlements of up approximately 150mm.

Building uplift solution to achieve final building levels consisted on the use of

Integrated Computer Leveling technique (ICL), in conjunction with a ground strengthening solution using jet grout columns (JG) in the upper sandy/gravel layer beneath the basement floor of the Christchurch Art Gallery, operating as a reaction platform.

### 1.1 Site Description

The Christchurch Art Gallery is located in Christchurch Centra - New Zealand, between Montreal Street to the east, Gloucester Street to the north and Worcester Boulevard to the south.

The west side of building faces vacant sites. The building does not abut directly with any other building or structure. The surrounding land is characterized by public space and road access, thus it is considered that the sub soil works will not interfere with other structures.

Site location is presented in Figure 1 and 2.

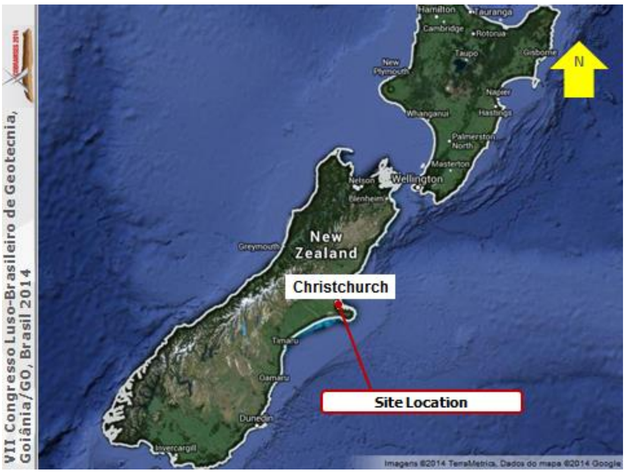


Figure 1. Site Location – New Zealand, Christchurch (*source: google maps*).


Figure 2. Site location (*source: google maps*).

## 2 GROUND CONDITIONS

### 2.1 Ground Investigation

Complementary ground investigation was undertaken at design stage, allowing a better understanding of the geological and geotechnical complex at site. The mentioned ground investigation campaign consisted on the execution of three (3) cone penetration tests external to the building to reach the ‘Riccarton Gravels’ formation with piezometer standpipe installation; three (3) machine boreholes to 10 m depth approximately within the basement of the building; three (3) cone penetration tests from the base of the boreholes to the ‘Riccarton Gravels’ formation and three (3) cone penetration tests from basement level till refusal. Water table level was found using one (1) piezometer standpipe installed through the basement floor.

The external cone penetration tests were able to punch through the upper gravelly soils and investigate the full depth to the ‘Riccarton Gravels’. With the boreholes machine it was possible to recover continuous samples of the gravelly soils beneath the basement and visually evaluate the gravel content of this layer. This predrilling trough the base of the gravelly soils allowed for cone penetration tests of the deeper sand stratum in the basement area.

#### 2.1.1 Cone Penetration Tests

The CPT work external to the building was undertaken with a 22 tf truck (Figure 2), using a cone of 15 cm$^2$ cross-sectional area, and a 225 cm$^2$ friction sleeve area. Where the access was limited (basement area), the CPT work was undertaken with a portable CPT rig, using a cone of 10 cm$^2$ cross-sectional area, and a 150 cm$^2$ friction sleeve area.

Continuous measurement of pore water pressure was undertaken during testing ($u_2$) and dissipation tests of low permeability layers were performed. Tests were undertaken in accordance with A.S.T.M. Standard D 5778-12 procedure.

2.1.2 Boreholes

The drilling work was undertaken with a sonic core machine rig. The core samples recovered from the borehole were logged in general accordance with the NZGS Guidelines "Field description of soil and rock". The machine borehole testing was designated to be taken till the upper gravel stratum was crossed.

2.1.3 Sub Surface Conditions

The interpretation of the exploratory holes suggests that sand with gravels and very dense gravelly soils are overlying sandy soils; this layer is interspersed with silt/clayey silt layers. A clayey silt and a sandy silt layer immediately overlays the Riccarton Gravel, reached at approximately 24 m depth. The sequence encountered is described in more detail in Table 1.

Table 1- Sub-surface conditions

| Geotechnical Unit | | Depth to base BGL (m) | Approx. thickness (m) |
|---|---|---|---|
| Made Ground | | 2.5 – 2.6 | 2.5 – 2.6 |
| 1A | Sand with gravels | 11.0 – 13.0 | 8.5 - 10.5 |
| 1B | Sandy Gravels | | |
| 2 | Silt/ Clayey-silt | 11.5 – 13.7 | 0.5 – 0.7 |
| 3 | Sand/Silty-sand | 15.0 -15.8 | 2.0 - 4.2 |
| 4 | Silt/ Clayey-silt | 16.5 – 17.2 | 0.8 – 1.5 |
| 5 | Sand/Silty-sand | 17.8 - 18.4 | 1.0 – 1.7 |
| 6 | Silt/Clayey-silt | 18.6 - 19.2 | 0.6 – 1.0 |
| 7 | Sand/Silty-sand | 20.1 – 21.6 | 1.5 – 2.5 |
| 8 | Sandy-silt with sand pockets | 21.9 - 22.6 | 1.0 – 2.0 |
| 9 | Clay/Silty-clay | 22.6 - 23.2 | 0.3 - 0.6 |
| 10 | Sandy-silt | 23.4 – 24.0 | 0.8 – 1.2 |
| 'Riccarton Gravels' | | Unknown | Unknown |

2.1.4 Geotechnical Parameters

Based on the results of the ground investigation campaign and taking into account the technical judgment and experience of local soils, the average geotechnical parameters presented on Table 2 have been assessed for the local geological units.

Plasticity Index (PI) of each layer and the shear wave velocity (Vs) were derived using recognized current engineering relationships and collected information from the Canterbury Geotechnical Database (Table 3).

Table 2- Geotechnical parameters.

| Geot. Unit | $q_t$ (MPa) | $\gamma$ (kN/m$^3$) | c' (kPa) | $\phi$' (°) | $E_s$ (MPa) |
|---|---|---|---|---|---|
| 1A | 6 - 20 | 18.0 | - | 30 - 35 | 12-50 |
| 1B | 20 - 30 | 19.0 | - | 33 - 38 | 50-90 |
| 2 | 1 - 3 | 17.5 | 10 | 24 | 2-6 |
| 3 | 15 – 25 | 19.0 | - | 32 - 36 | 30-70 |
| 4 | 1 - 4 | 17.5 | 10 | 24 | 2-8 |
| 5 | 6 - 25 | 18.5 | - | 30 - 36 | 12-70 |
| 6 | 1 - 4 | 17.5 | 10 | 24 | 2-8 |
| 7 | 4 - 14 | 18.5 | - | 28 - 32 | 10-30 |
| 8 | 2 - 8 | 18 | - | 27 - 30 | 6-15 |
| 9 | 1 - 2 | 17.5 | 20 | 15 | 4-8 |
| 10 | 2 - 8 | 18 | - | 27 - 30 | 6-15 |

Table 3- Plasticity Index and shear wave velocities.

| Geotechnical Unit | PI | Vs (m/s) | Design value Vs (m/s) |
|---|---|---|---|
| 1A | 0 | 175 - 210 | 187 |
| 1B | 0 | 280 - 320 | 300 |
| 2 | 15 | 130 - 155 | 136 |
| 3 | 0 | 230 - 250 | 240 |
| 4 | 15 | 135 - 170 | 147 |
| 5 | 0 | 200 - 250 | 220 |
| 6 | 15 | 140 - 175 | 152 |
| 7 | 0 | 195 - 240 | 218 |
| 8 | 0 | 175 - 220 | 195 |
| 9 | 25 | 130 - 155 | 143 |
| 10 | 0 | 180 - 225 | 200 |

2.1.5 Ground Water

Piezometer standpipes were installed in the basement and in the external cone penetration tests. These have subsequently been monitored using a dip meter.

During monitoring period, significant changes on the ground water level were observed due to dewatering works on a property near to the Christchurch Art Gallery building. The water level subsequently stabilized at a RL between 12.9 m and 12.6 m.

## 3 METHODOLOGIES FOR BUILDING RELEVELLING

### 3.1 Introduction

Building uplift solution to achieve the required levels consisted on the use of Integrated Computer Leveling technique (ICL), in conjunction with a ground strengthening solution with jet grout columns (JG) in the upper sandy/gravel layer beneath the basement floor, operating as a reaction platform.

There were two areas where, due to access constraints, jet grouting was not possible and compaction grouting (out of the scope of the present article) was used to improve the soil characteristics and create the necessary reaction platform.

### 3.2 ICL – Integrated Computer Levelling

The ICL technique is an integrated computer-controlled levelling method that manipulates grout rheology, controls the viscosity, fluid state, setting and cure times of its range of injected cementitious jacking grouts and, as a consequence, can control the grout's ability to permeate the soil and allows control of the generated uplift force acting directly against the underside of the structure / foundations.

Injecting at multiple locations sequentially exponentially reduces the single point energy required to overcome the structure's initial inertial forces and allows a continuous, balanced, controlled and very gentle lift over large areas.

Injection of grout with a suitable viscosity enables the formation of multiple thin layers of grout below the building foundation. As the initial grout sets, new grout is injected and flows over the previous grout layer as it sets, resulting in lift; the successive injection of grout creates layers which build up progressively in a random radial and laminar manner.

Grout injection is continued until the specific lift amount for the building is met. The computer control allows closing and opening the valves in the injection needles in way to deal with different amount of lift across the building.

When injection process is completed, it is expected to have a more or less uniform layer of 0.5 m thick grout material.

### 3.3 Jet Grouting

Jet grouting uses a high kinetic energy jet of fluid to break up and loosen the ground, and mix it with thin cement slurry. This hydrodynamic mix-in-place technique produces a soil-cement material, commonly referred to as a jet grout column.

Jet grouting makes use of three physical processes, singly or in combination: the very high speed jet loosens the soil; the jetting fluid washes some of the soil to the surface; the slurry adds a binder to the soil mix.

During jetting, material in excess of the soil cement mix must rise freely to the injection collar, in order to prevent the excess material fracturing and disturbing the surrounding ground. The excess grout slurry is removed a rate to ensure excess pressure does not build up in the fluid column being formed. The final resulting jet-grout columns (diameter, composition and strength of the columns) are dependent on drill string rotation and raising speeds, jet pressure and flow, grout mix, soil type, grain size distribution, composition and compactness and nozzle configuration, among others. Figure 3 shows the jet grouting columns being executed at the basement of the building.

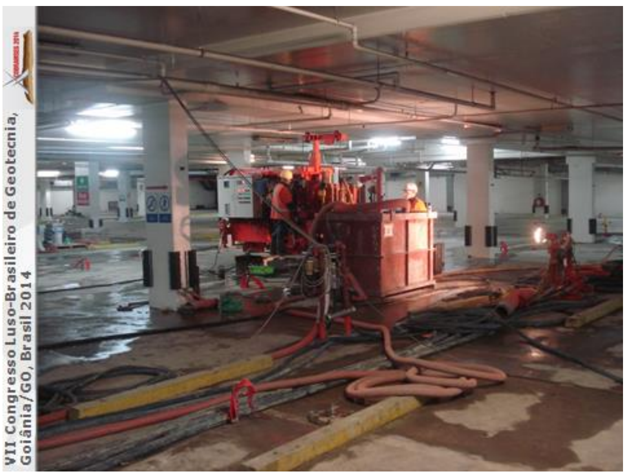


Figure 3 – Execution of jet grouting columns at the basement of the building.

## 4 GROUND IMPROVEMENT SOLUTION

### 4.1 Introduction

The ground strengthening with jet grout columns consisted on the installation of 3.0 m diameter jet grouting columns, with a 7.50 m distance between columns, in a triangular grid pattern. However, due to site and structural restrictions, this grid was rearranged in some specific areas.

The position of the jet grouting columns was defined so as to allow a load transfer layer between the foundation of the building and the columns to optimize stress distribution, providing partial transfer of load directly to the jet grouting elements, relieving soil compression.

Around the perimeter of the building, the jet grout columns were increased to 4.0 m diameter to improve the stiffness of the reaction platform at the edges.

The columns have been installed from within the basement of the existing building, and reached the stiff layer characterized by sands and gravels (unit 1A/1B). The reference level for the jet grout columns was the base of the concrete raft, considered for design purposes as at level 0.00 m. The estimated length of the JG columns is 4.0 m, with the top positioned at level -2.50 m and the bottom at level -6.50 m. A plan location of the JG columns grid and typical cross section are presented in Figure 4.

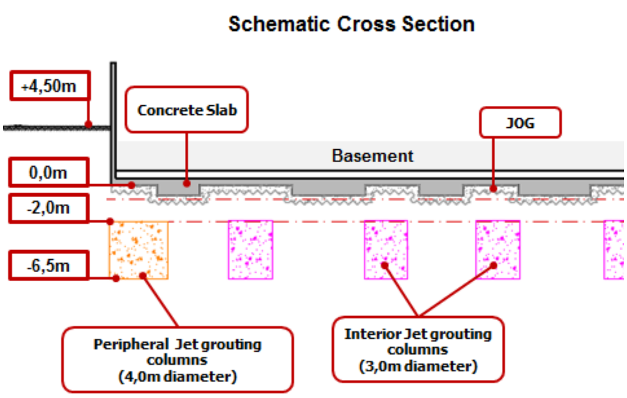


Figure 4 – Typical cross section of the adopted solution.

### 4.2 Load Transfer Layer and Foundation of Jet Grouting Columns

The position and level of the top of the JG columns was defined in such a way as to provide a load transfer layer between the foundation of the building and the columns, taking in account the ground stiffness and in order to allow an adequate stress distribution between the jet grout columns and the surrounding soil.

The load transfer layer aims to optimize stress distribution, allowing a significant percentage of load to be taken directly to the jet grouting elements, relieving soil compression. The thickness of the load transfer layer was determined using the formulation provided by Guido et al. (1987) and in accordance with EBGEO rules and "IREX - Reinforcement des Sols par Inclusions Rigides". (2002).

Considering ground improvement treatment with rigid inclusions (jet grouting) under concrete slabs, EBGEO and IREX recommend that the following relation is observed:

$$H_R \geq 0,5 \times Sm \qquad [1]$$

Where $H_R$ is the thickness of transfer load platform; $s_m$ is the spacing between columns ($s_m$= s – 2r), being "s" the treatment grid spacing and "r" the jet grouting columns radius.

A triangular treatment grid spacing of 7.5 m and a jet column diameter of 3.0 m were considered. In this case, the spacing between columns is taken as $s_m$= 7.50 m – (2 x 1.50 m) = 4.5m.

Following EBGEO and IREX recommendations, the load transfer layer thickness shall meets the following: $H_R \geq 0.50 \times 4.5\ m \geq 2.25$ m. It was considered that a 2.50 m thick load transfer layer, established according with the treatment grid and jet grouting columns diameter is appropriate. Making use of the known "arching" effect between columns, this stress distribution is considered to be sufficient to ensure that the loads transmitted to the ground will be mainly supported by the JG columns, reducing the stress imposed on the soil layer between columns, even if improved by the lateral confinement due to the columns.

Moreover, the load transfer layer will also obviate harmful punching effects on the foundation slab of the building, since the slab was not designed for point load support on the underside. It is to be highlighted that the transfer soil layer, positioned beneath the basement slab, was considered to have enough strength and stiffness to act as a load platform.

The base of the JG columns was designed to be founded on the dense layer Unit 1A/1B (sands with dispersed gravels, medium dense to dense and very dense sandy gravels) but sufficiently above layer 2 (very loose to loose silt with trace of clay) to avoid creating settlements greater than the contractual specification limits.

### 4.3 Design Calculations

Calculations were developed using finite element analysis programs – PLAXIS 2D and PLAXIS 3D FOUNDATION.

The mentioned programs are specially developed to analyze soil – structure interaction behavior on geotechnical works, and take into account the construction stages. Structure geometry was simulated on a 15 node plain strain model and soil properties were defined using *Hardening Soil Model*.

PLAXIS 2D analysis enabled to simulate the overall behavior of the treated ground, as the adopted model represented the longitudinal cross section of the building, based on a width of about 90.0 m, corresponding to the longer dimension of the structure. In order to confirm the results obtained in the 2D model, and to analyze in detail the soil behavior in a representative treatment area in the interior of the building footprint, complementary analysis using PLAXIS 3D was carried out.

Soil behavior was modeled taking into account the stiffness and strength of the soil layers under an imposed vertical stress corresponding to the building loading. Under the slab, a grout treated layer (ICL) of 0.50 m thickness was considered; this being the zone predicted that grout will penetrate.

A soil volume increase, corresponding to the maximum uplift value of 0.15 m, was simulated considering soil volume expansion, enabling the simulation of the grout injection. The ICL grout layer is confined to between the foundation slab and the soil.

Taking into account the structure loading above the ICL layer as well as the stiffness of the soil, it was possible to determine the deformation transmitted to the structure and to the soil.

The deformation imposed into the soil due to soil volume expansion during the leveling process lead to an increase in the imposed ground stresses. Knowing the magnitude of the incremental stress imposed into the ground, it was possible to analyze the maximum load transmitted to the jet grout columns and to calculate the corresponding deformation, confirming the adequacy of the ground improvement solution.

The parameters presented on Table 5 were considered for the jet grout soil-cement composite material, assuming that the columns will be formed in a frictional soil (sands and gravels).

Table 5- Characteristics of composite jet grout soil-cement material.

| | | |
|---|---|---|
| Unconfined compressive strength, 28 days | UCS (MPa) | 2.2 |
| Permeability | k (m/s) | 1.00E-08 |
| Secant Stiffness Modulus | $E_C$ (GPa) | 1.0 |
| Friction angle | Øº | 38 |
| Cohesion | c' (kPa) | 180 |
| Poisson ratio | ν | 0.30 |

#### 4.3.1 Design Assumptions

Design was developed taking into account that underside of foundation slab is at level +0.00 m and external ground surface is at level +4.10m.

Groundwater level is at an elevation coincident with the foundation slab base (elevation +0.00m); this was achieved and maintained during all stages of the relevelling by dewatering works.

The building foundation acts as a relatively stiff and homogeneous raft, with the stresses being uniformly transmitted to the ground. Maximum imposed stress in the ground by the existent building at foundation level is 110 kPa.

The ICL comprises the treatment with grout of the first 0.50 m depth of subsoil below the foundation slab. The uplift was carried out by soil fracture (injection of grout at high pressure) of this zone, creating vertical increase in soil volume, corresponding to the uplift required (Maximum uplift leveling during ICL of 150 mm). It was considered that the volume increase during injection occurs gradually, and with uniform and horizontal spreading of the grout in each layer (checked on site taking into account both the injection stages and the monitoring and observation plan).

### 4.3.2 Effective Vertical Stresses

Vertical effective stresses on the soil were evaluated at a depth of -1.20 m, at a distance of about 1.30 m above the top of the jet grout columns. This level is considered to be representative of the stresses imposed on the jet grout columns and surrounding soil due to the ICL works.

A maximum effective vertical stress of $\sigma'_y$=126 kPa was obtained after building construction, before ICL works and soil strengthening with jet grouting (Figure 5).

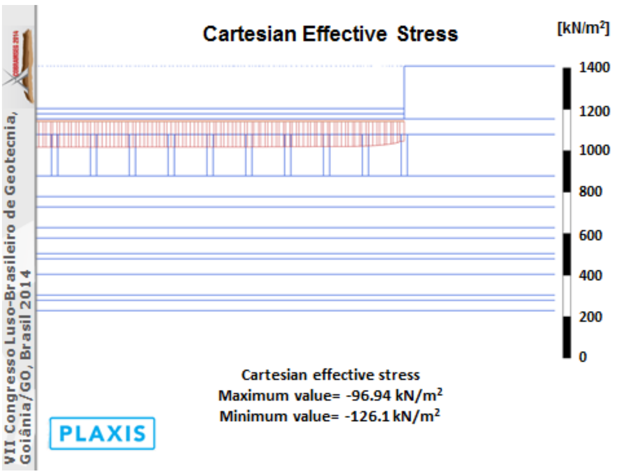


Figure 5 – Effective vertical stress at a depth of 1.20m.

A maximum effective vertical stress of approximately $\sigma'_y$=180 kPa was obtained after soil strengthening with jet grout columns and ICL (Figure 6). Effective vertical stresses transmitted to the jet grout columns are presented in Figure 7.

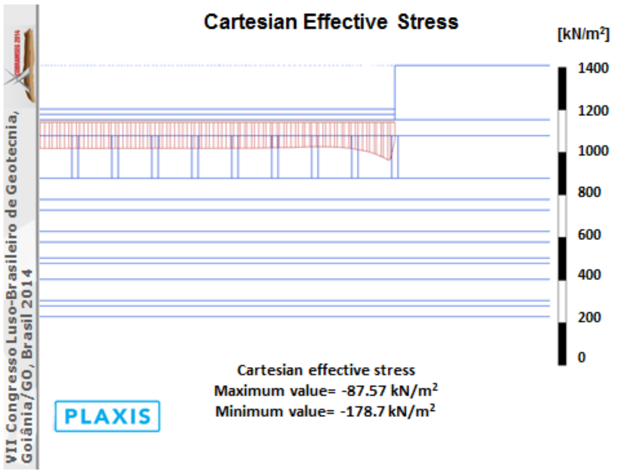


Figure 6 – Maximum effective vertical stress after soil improvement and ICL.

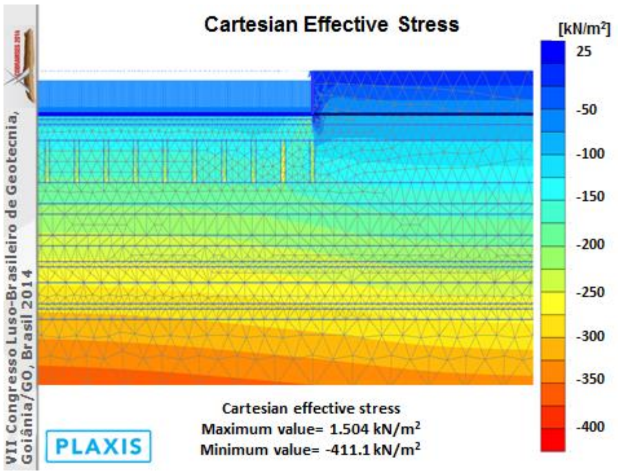


Figure 7 – Effective vertical stresses transmitted to JG columns.

### 4.3.3 Immediate Settlements

The resulting deformed finite element mesh for the 2D model after soil model construction is presented in Figure 8. Soil layers, structural elements and loads are represented in the model, as well as the triangular 15 node finite element mesh generated.

The results obtained show that the volume expansion on the ICL grout layer leads to a 'positive' deformation on the slab of about 0.15 m, as required. The deformation transmitted into the ground, at slab level, corresponds to a settlement of about 10 mm (Figure 9).

Vertical displacements are only originated by the ICL injection process, enabling the building uplift in 0.15 m. The initial settlements due to building construction were not considered as it is expected that those settlements have already occurred during life time period of the structure.

Nevertheless, the incremental effective stress on the ground due to the building load is reflected on the calculated settlements originated by the ICL.

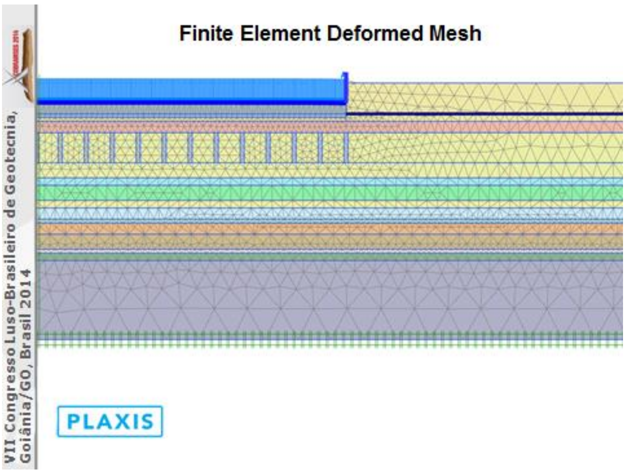


Figure 8 – Deformed finite element mesh.

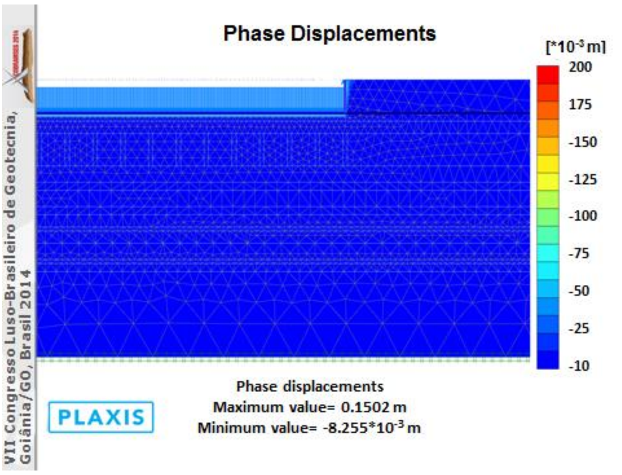


Figure 9 – Immediate calculated deformations after soil improvement.

A better understanding of the expected settlements transmitted to the ground were evaluated at levels -0.70 m, -1.20 m and -7.50 m.

Beneath the ICL grout layer, estimated settlements of about 5 mm were obtained in Unit 1A (level-0.70m). At the top of the jet grout columns, an estimated settlement of 3 mm was obtained in Unit 1B, at level -1.20 m. Underneath the jet grout columns, at level -7.50 m, a settlement of 2 mm was calculated.

According to the estimated ground deformations previously presented, low settlement was determined after ICL, with the settlement magnitude decreasing with depth. It must be pointed out that the existing settlements at this stage are considered to be offset by the ICL leveling process.

### 4.3.4 Long Term Settlements

Long term settlements were calculated considering consolidation characteristics of the soils with low permeability. Considering the difficulties to predict in detail the degree of consolidation that had already occurred during the period lifetime of the building, particularly when the seismic events may have contributed to changes of pore water pressures, the consolidation analysis was based only on the incremental stress imposed by ICL works.

Consolidation analysis and long term settlements were calculated using finite element program PLAXIS 2D. Consolidation process was analyzed until a minimum pore water pressure of less than 1 kN/m$^2$ was reached. Following the consolidation process a long term settlement of 18 mm was determined.

Figure 10 presents the obtained consolidation settlements.

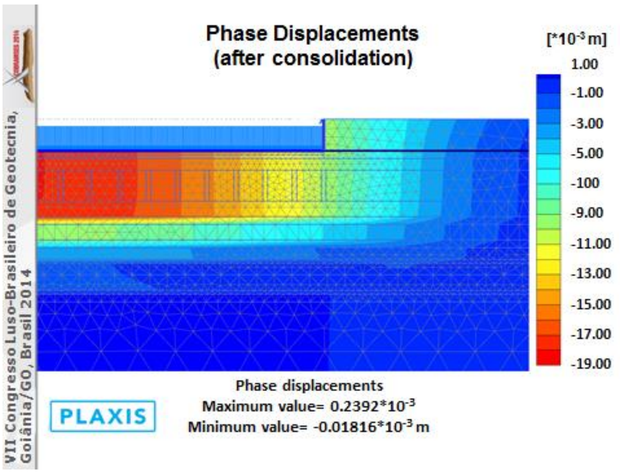


Figure 10 – Settlements after consolidation.

The consolidation settlements on each undrained soil layer are presented in Figure 11.

The estimated time for the pore water dissipation (primary consolidation) on the low permeability soil layers was estimated to be of the order of 1 month, with the most part of the settlements occurring in about 10 days (Figure 12).

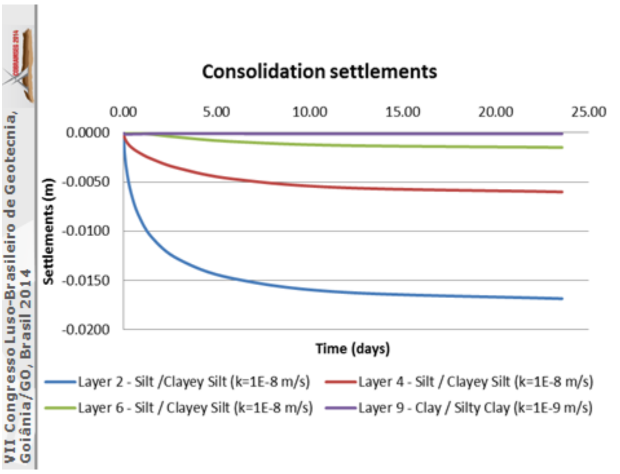


Figure 11 – Consolidation settlements on each undrained soil layer.

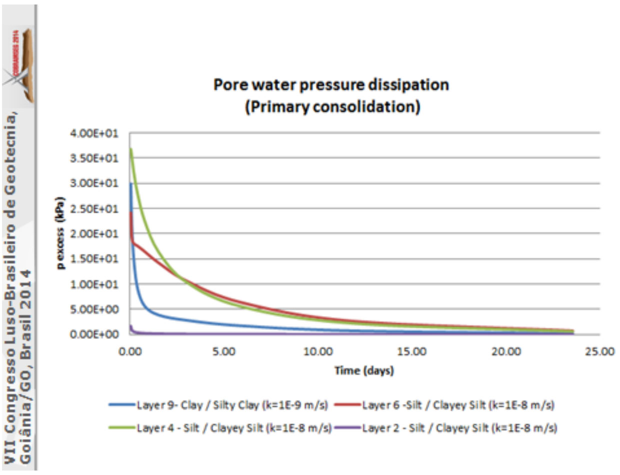


Figure 12 – Estimated period for primary consolidation.

## 4.4 3D Analysis

In order to confirm the results obtained from the 2D model and to analyze in detail the soil behavior in a representative area of 30m x 20m in the interior of the building, complementary analysis using PLAXIS 3D was carried out.

The results obtained show that the volume expansion on the ICL grout layer leads to a 'positive' deformation on the slab of about 0.15 m, as required. The deformation transmitted into the ground is almost non-existent and settlements are considered to be negligible. Nevertheless, it is considered that any significant settlement at this stage is offset by the ICL levelling process.

The deformed finite element mesh for the 3D model and the calculated soil deformations after ICL and soil improvement with JG columns are presented in Figures 13 and 14. In these figures, the soil around columns was erased to allow a better view of the jet grout columns installed under the pavement slab, nevertheless, that soil was considered in calculations.

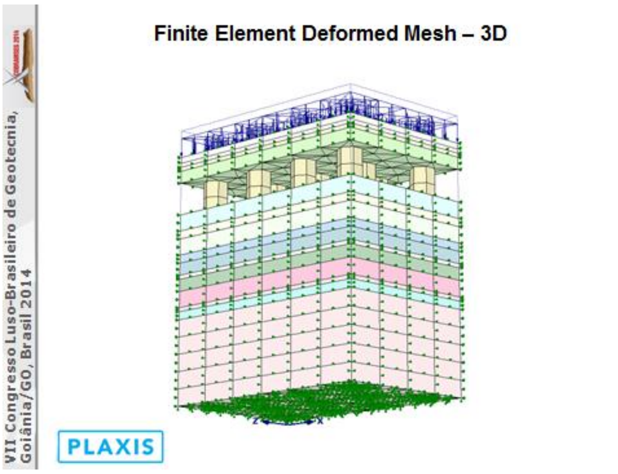


Figure 13 – 3D model finite element mesh.

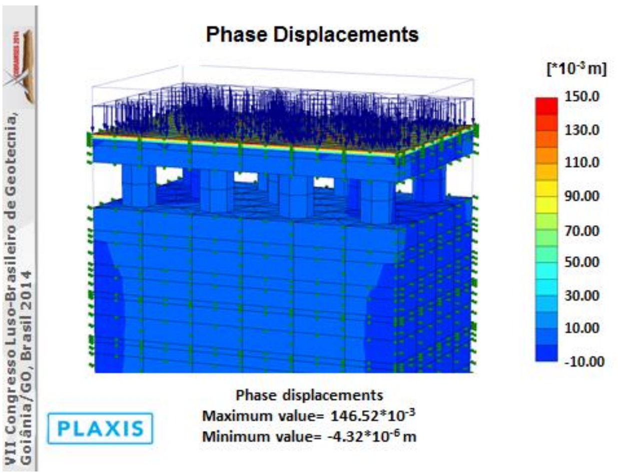


Figure 14 –Soil deformations after ICL and jet grouting columns – Plaxis 3D.

### 4.4.1 Induced forces on foundation concrete slab

Axial and shear forces, as well as bending moments, transmitted to the concrete slab due to ICL construction stages were estimated using Plaxis 3D.

Axial and shear forces were estimated to be in order of 0.11 kN/m and 0.17 kN/m, respectively.

Bending moments were found to be approximately 0.38 kN.m/m, and may be considered as negligible from the structural point of view, attesting the effectiveness of the load transfer layer on the reduction of the forces transmitted to the concrete slab.

# 5 DYNAMIC ANALYSIS

## 5.1 Introduction

The structural response due to earthquake motion was analyzed considering dynamic interaction behavior on the reinforcement elements, namely the jet grouting columns and the surrounding soil.

The response of a finite element numerical model is conditioned by the setting of several parameters influencing the sources of energy dissipation in time-domain analyses, such as material damping: influencing the effects of soil viscosity and hysteretic energy dissipation; numerical damping: as a consequence of the numerical algorithm solution of dynamic equilibrium in time domain; boundary conditions: affecting the way in which the numerical model transmits the stress waves specific energy outside the domain.

For the present study, two software programs were used to analyze the ground dynamic response: Equivalent-linear Earthquake site Response Analysis (EERA) and FEM PLAXIS 2D dynamic.

PLAXIS 2D analysis enables simulation of the soil column behavior as well as the corresponding seismic response of the ground after soil improvement. This provides the following site response information: relative horizontal displacements at surface; relative acceleration at surface; maximum shear stress on the soil; maximum shear strain in the jet grout columns due to the horizontal seismic action; overall safety factor after seismic analysis.

With the previous information an evaluation of three major points was undertaken: the general behavior of the model due to seismic action (determining displacements, accelerations at surface and shear stress profiles); regarding the jet grout column integrity, a safety analysis was carried out to determine shear stress resistance; calculation of overall factor of safety against dynamic bearing failure regarding soil bearing capacity and JG column bearing capacity.

In order to evaluate the JG column shear resistance due to a typical seismic event, the following characteristics were calculated: shear strain and shear stress.

## 5.2 Seismic Action

The seismic motion registered during the 4 of September 2010 earthquake at CBGS Station and obtained from the database provided by “GeoNet”, was considered to be representative of the site, due to the station’s location near to the Christchurch Art Gallery.

It is also located on ground considered to be of similar soil characteristics, as indicated by borehole log information from the Canterbury Botanic Gardens obtained from Canterbury Geotechnical Database (CGD).

A magnitude of 7.1 and a ground peak acceleration of $a_{max}$=0.17g were recorded during the seismic event on the 4th of September 2010 at CBGS station. This dynamic design scenario was used for analytical purposes.

As the earthquake loading is often imposed as an acceleration time-history at the base of the model, for the numerical computational analysis, the seismic input signal (acceleration time-history) was introduced into the computer code.

The SW component of the accelerometer conduced to a horizontal peak ground acceleration of 0.17 g, reached in 25.5 s.

## 5.3 Dynamic Response

### 5.3.1 Introduction

Dynamic behavior calculations of the improved soil with jet grout columns were carried out using the finite element analysis program PLAXIS 2D dynamic.

The structure geometry was simulated using a 15 node plain strain model and soil properties were defined using *Hardening Soil Model*. The model was constructed simulating one single row of jet grouting columns positioned beneath the basement slab of the building. The improved soil around the jet grouting column row was modelled as a soil layer with soil properties equivalent to improved ground.

The simulation of one-dimensional wave propagation and boundary conditions were established by means of vertical fixities applied to the bottom horizontal border of the model only, to enable free horizontal displacements in the soil model.

### 5.3.2 Relative Horizontal Displacements

Three (3) points were selected to provide dynamic response results after seismic wave propagation: point A (positioned at the underside of the basement slab of the building); point B (positioned at a level coincident with the top of the jet grout columns); point C (positioned at a level coincident with base of the jet grout columns).

After seismic motion, an estimated maximum relative horizontal displacement of 170 mm at the basement slab level of the building (Point A) was obtained. With respect to the hysteretic behavior of soils, earthquake motion tends to dissipate with time, giving rise to residual deformations. Following the seismic motion, a residual horizontal displacement of approximately 26 mm was determined.

Relative displacements were evaluated at the top and at the bottom of the jet grout columns, at points B and C respectively. According to the results obtained, a maximum relative horizontal displacement of around 150 mm is expected at the top of the columns (Point B). Residual horizontal displacement after earthquake motion was estimated to be approximately 25 mm (Figure 15).



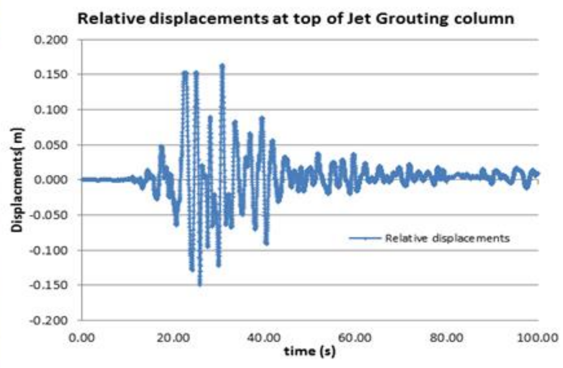


Figure 15 – Relative horizontal displacements at the top of jet grouting columns.

### 5.3.3 Shear Strain and Shear Stress

Shear strain was determined attending to the difference between relative horizontal displacements at the top and at the bottom of the jet grouting column. Shear strain was calculated in accordance with the following expression:

$$\gamma_s = (\Delta\delta / L) \qquad [2]$$

Where $\Delta\delta$ is the differential horizontal displacement between the top and the bottom of the JG column (points B and C, respectively) at the same instant of time and L is the total length of JG column (in this case, L=4.0m).

A maximum shear strain in the jet grout column of 0.03% was obtained, corresponding to a maximum differential horizontal displacement of 1.2 mm.

The results of the differential horizontal displacements between Points B and C, positioned at the top and at the bottom of the jet grout column respectively, are presented in Figure 16.

Following the seismic motion, a maximum shear stress of $\tau_{max}$ = 225 kPa was obtained in the jet grout column, indicating that the composite soil-cement elements are concentrating a significant amount of the imposed shear stress (Figure 17).

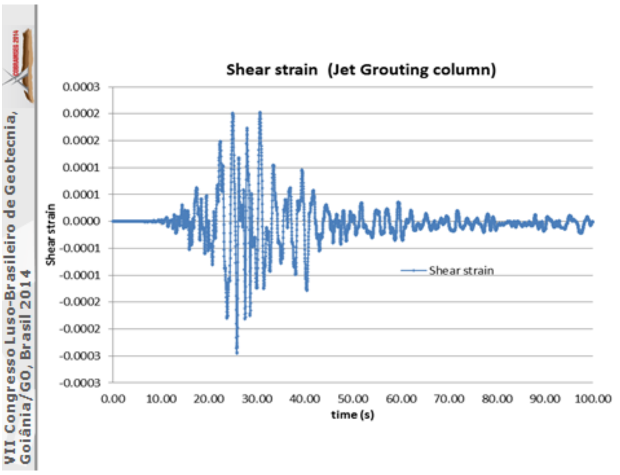


Figure 16 – Shear Strain on jet grouting columns.

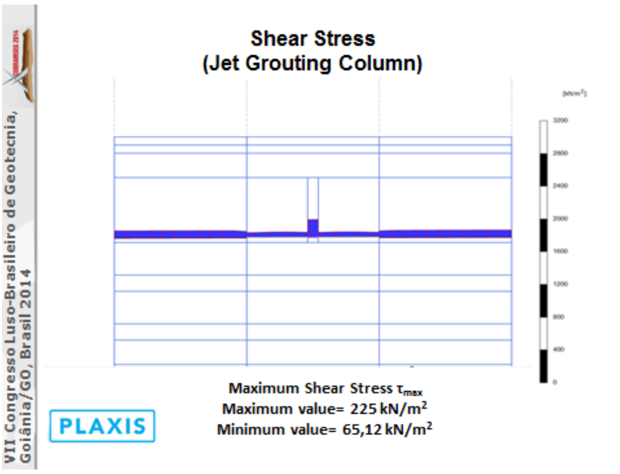


Figure 17 – Shear Stress on jet grouting columns.

### 5.3.4 Shear Stress resistance of jet grouting columns

Evaluation of the column integrity in terms of shear resistance was undertaken, comparing the imposed shear stresses from seismic motion with the maximum shear stress column capacity.

According with Plaxis 2D results, a vertical effective stress on the jet grouting column of $\sigma'_v$=212 kPa was obtained.

Since grout material is considered to have a cohesion and an internal friction angle of $c'$=180 kPa and $\phi'$=38º, respectively, the resulting resistant shear stress of the composite soil-cement jet grout material is $\tau'_{max}$=346 kPa (determined according with the Mohr-Coulomb criteria).

Considering that design shear stress of $\tau$=225 kPa is observed on the jet grout column due to seismic action, the integrity of the column is assured since imposed shear stress ($\tau$) is inferior to the resistant shear stress of the composite soil-cement ($\tau'_{max}$), corresponding to an internal factor of safety for the jet grouting column of SF=1.54.

## 5.4 Soil Induced Liquefaction Mitigation

### 5.4.1 Introduction

The relevel works will provide the existing building with improved behaviour under seismic loads by formatting a stabilized crust down to 6.5 m depth below the building foundations and reducing the amount of the predicted settlement induced by soil liquefaction.

### 5.4.2 Liquefaction Mitigation

The jet grout columns will provide a zone of ground improvement that will reduce soil shear strains during seismic events (due to stress concentration) and therefore reduce the severity of liquefaction of the treated zone.

The stress concentration reduction factor due to soil improvement using jet grout columns ($K_g$) was determined in accordance with the H. Turan. Durgunoglu (2004) formulation. Assuming that the ratio of the jet grout columns shear modulus ($G_{JG}$≈400 MPa) and the shear modulus of the soil ($G_S$≈40 MPa) is around 10 and the replacement ratio is of 19.6 % we achieve a reduction factor of Kg=0.36.

In order to estimate the potential for soil liquefaction, the ratio Cyclic Resistance Ratio (CRR) and Cyclic Stress Ratio (CSR) was determined. If the mentioned ratio is inferior to 1.0, it is considered that soils are potentially liquefiable. On opposite, if CRR/CSR ratio is equal or superior to unit, liquefaction phenomena can be neglected.

In the jet grout column area, the original CSR values were affected by the calculated shear stress reduction factor ($K_g$=0.36). The seismic design requirements adopted for use in the analyses were: NCEER's calculation method (modified for fines content); magnitude M7.5 EQ event; peak ground acceleration of 0.20 g (for annual exceedance probabilities of 1/150 – SLS); peak ground acceleration of 0.44 g (for annual exceedance probabilities of 1/1000 - ULS).

Liquefaction analysis was undertaken using software 'CLiq' and Figures 18 and 19 illustrates the procedure adopted in our analysis.

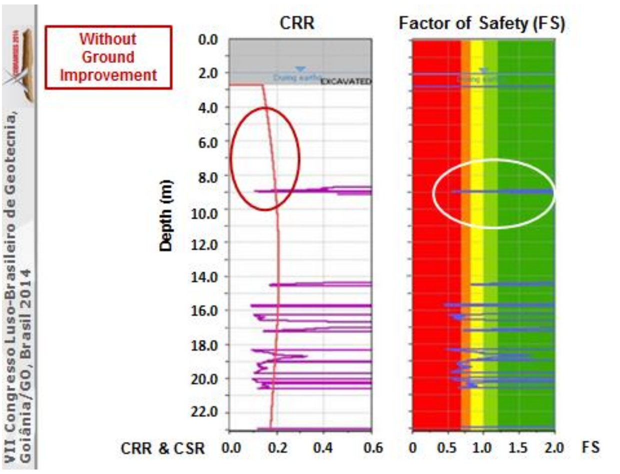


Figure 18 – Liquefaction assessment with Cliq (based on CPT6b results) without ground improvement.

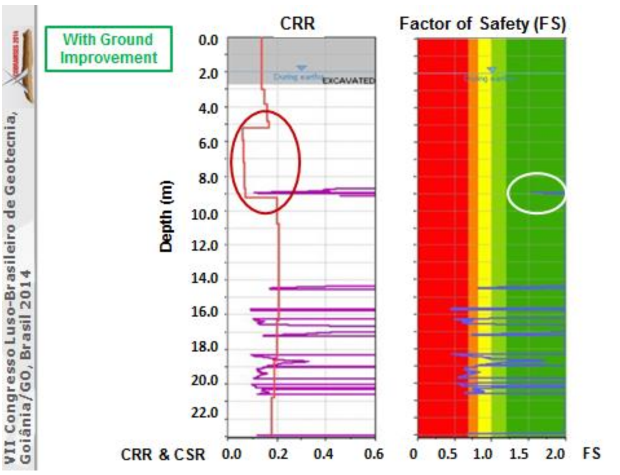


Figure 19 – Liquefaction assessment with Cliq (based on CPT6b results) with ground improvement.

While the upper soil layers (1A and 1B) may be considered generally as non-liquefiable, a few thin liquefiable layers still exist in the sandy soils (1A) between the gravel layers (1B). The ground improvement will mitigate the liquefaction in these layers under the considered SLS event. Figures 20 and 21 shows an overlay of these potential liquefiable layers using the available CPT data, and the effect of the ground improvement mitigation under the considered SLS event using the above procedure.

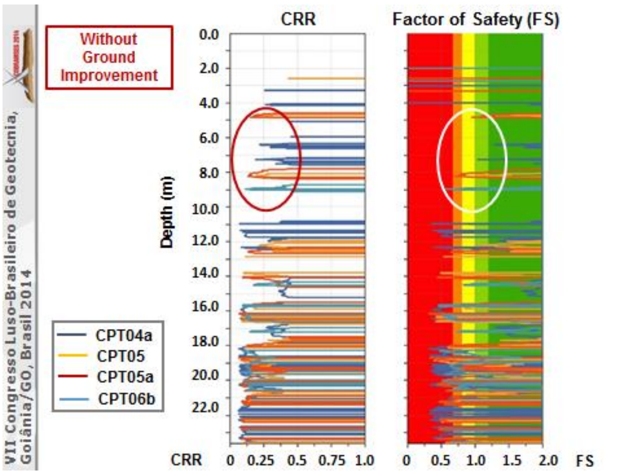


Figure 20 – Liquefaction assessment without ground improvement - overlay of CPT's 4a, 5, 5a and 6b.

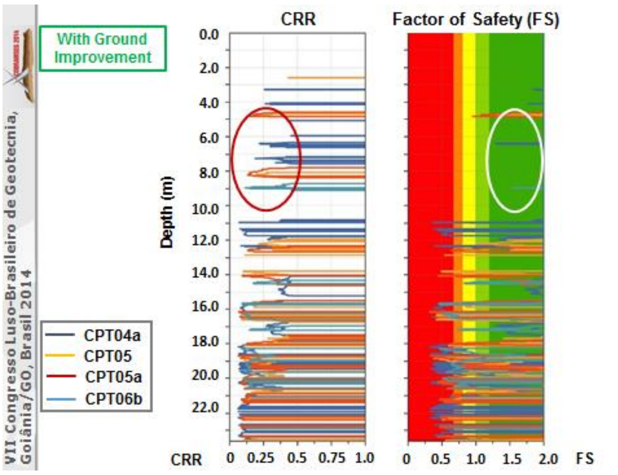


Figure 21 – Liquefaction assessment with ground improvement - overlay of CPT's 4a, 5, 5a and 6b.

### 5.4.3 Stabilized Crust

One of the design added value was to achieve improved behavior of the structure under future seismic events and to increase the strength of the soils between 5 m to 7 m depth below the foundation. This solution is based on the assumption that a non-liquefiable crust has the benefit of mitigating the effects on the structure of deeper liquefaction induced settlements.

Global settlements were considered to be tolerable and expected to occur under a seismic event, but the effects on surface structures should be non-damaging.

Professor Ishihara (Kenji Ishihara, 1985) collected data from numerous case studies where he proved that there is an inverse relation between the thickness of a mantle of non-liquefiable soils and the liquefaction damage observed at the surface. If the non-liquefiable crust is sufficiently thick, the uplift force due to the excess water pressure will not be strong enough to cause a breach in this layer, and hence, there will be no surface manifestation of liquefaction even if it occurs deep in the sub-soil.

Trial tests based in this assumption have been developed in the Christchurch urban area and preliminary results confirm, and sometimes refine the expected results.

The Ishihara charts (Figure 22) show that a non-liquefiable crust of 3 m thickness is sufficient to limit the expected liquefaction induced ground damage under a SLS event of 0.2 g and M7.5.

The relevel solution of the Christchurch Art Gallery provides a non-liquefiable crust of at least 6.5 m below foundation, which is considered to minimize the manifestation of liquefaction damage at foundation level.

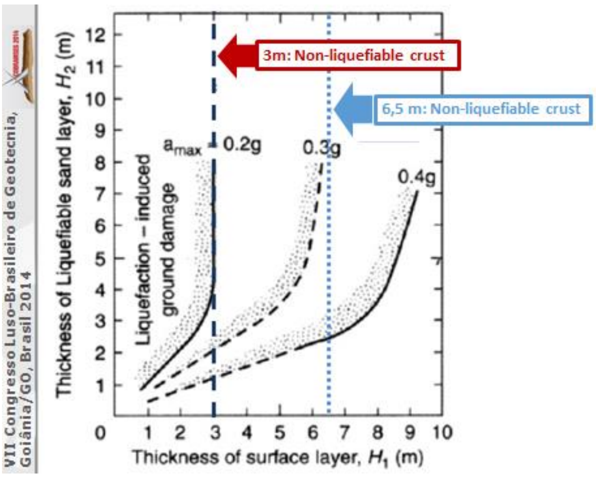


Figure 22 – Boundary curves for site identification of liquefaction-induced damage (Ishihara 1985).

## 5 QUALITY ASSURANCE AND QUALITY CONTROL

The jet grouting method is highly dependent on both ground conditions and equipment characteristics, as well as on quality assurance and quality control. Thus, preliminary testing was undertaken, to ensure that design assumptions could be confirmed.

These preliminary tests were necessary to calibrate installation parameters namely, penetration/retrieval velocity and rotation; injection flow rates during penetration and retrieval; injection pressure and amount of cement. Trial tests also allowed to confirm the maximum penetration depth achieved as well as the column diameter.

During the execution of the jet grouting works, all the main installation parameters were recorded. The permanent spoil volume was checked and a tight and complete monitoring and survey plan was adopted in order to confirm the execution stages, as well the behavior of the building structural and non-structural elements. Considering that it is imperative that jet grout columns are constructed with the required strength and stiffness properties, preliminary analysis of composite soil-cement material was undertaken: fresh samples of mixture and core samples were collected at site and laboratory tested.

On completion of the ground improvement with jet grout columns, core drilling of a representative number of columns was performed to verify diameter and the final resistance of the soil-cement column.

Level monitoring has been carried out during jet grouting works, especially taking into account that columns were installed under an existent structure. A monitoring program was implemented in order to control, in real time, the effects on the structure during the execution of the jet grouting columns.

After the completion of the relevel works, a new topographic survey was undertaken, confirming that the target design displacement was reached. The initial settlement and the final displacement results after the ground improvement and JOG are presented in figure 23.

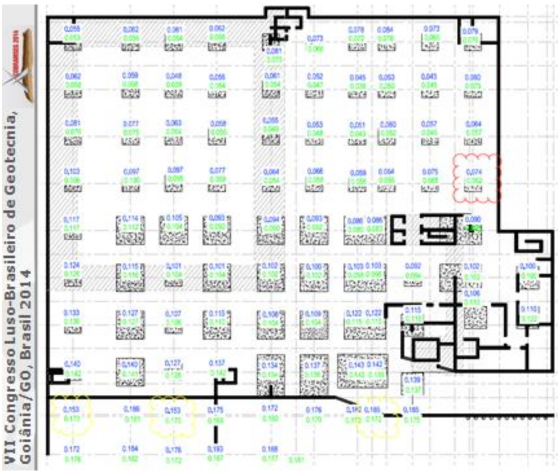


Figure 23 – Final results after completion of the relevel works: initial settlements (blue values) and target displacements (green values).

## 6 MAIN CONCLUSIONS

Taking into account the relevel of the Christchurch Art Gallery building (Figure 24), it is possible to point out the following advantages on the use of jet grouting columns as reaction platform:

– After ground strengthening with JG columns and relevelling of the building due to ICL, the calculated maximum incremental settlement on the ground is about 10 mm, corresponding to an additional local stress on the ground of about 54kPa.
– The estimated ground deformations, considered to be low, prove the adequacy of the JG soil strengthening as a reaction platform for ICL.
– Axial and shear forces as well as bending moments transmitted to the concrete slab during the ICL stage construction are estimated to be low, confirming the effectiveness of the load transfer layer on the reduction of the forces transmitted to the concrete slab.
– PLAXIS calculations using 2D and 3D models present generally similar results. However, some differences were identified due to construction modelling and the accuracy of generated finite element mesh differences. In an overall analysis, it is considered that the small differences between 2D and 3D analyze have no significant influence for the overall analysis of the solution.
– Added value effect occurs due to stress concentration on the jet grout columns that, together with the soil densification, will enable a non-liquefiable crust of soils to at least 6.5 m below building foundations under a seismic event of 0.2 g and M7.5.


Figure 24 – Christchurch Art Gallery building.

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