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Lessons learned and need for appropriate philosophy for the design of stone column ground improvement under seismic scenarios

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ABSTRACT

The Deans Stand at Lancaster Park (formerly AMI Stadium) in Christchurch, New Zealand was of modern reinforced concrete design and construction. It was largely supported on a hybrid foundation system comprising ground beams and stone column ground improvement that extended part way through a relatively thick layer of liquefiable sands and silts. The Deans Stand was subjected to severe earthquake shaking during the Canterbury Earthquake Sequence (CES) of 2010-11. Damage at the Deans Stand included bulging, loosening and contamination of the stone columns and loosening of the densified ground between them. Using a large dataset of pre- and post-improvement site investigations, this paper firstly presents insights into the behaviour of stone columns under multiple strong earthquakes. The paper then demonstrates how liquefaction has led to loss of the densification of the surrounding ground achieved by stone column construction. Important consequences for the design of ground improvement that relies on densification to mitigate against liquefaction are then discussed. In particular, the current focus on design to meet

Paper 112

code prescribed life safety performance is questioned in the context of economic repairability considerations.

1 INTRODUCTION

The Canterbury Earthquake Sequence (CES) of 2010-2011 caused widespread significant damage to Lancaster Park stadium. The most severe shaking was due to the 22 February 2011 magnitude M_w 6.2 earthquake with an epicentre approximately 6km from central Christchurch. According to the Canterbury Earthquakes Royal Commission reports (2012), response spectra derived from ground motions measured at stations around the central city exceeded the 1 in 2,500 year acceleration response spectra given by the relevant NZ structural design standard, NZS 1170.5:2004 (2004). To put this in context, the stands were designed for 1 in 1,000 year shaking.

Lancaster Park Stadium was the largest sporting and events venue in Christchurch. The stadium sustained substantial damage to foundations and structure as a result of the earthquake sequence, and in particular the February 2011 earthquake and as a result has been demolished.

Engineering damage assessments following the earthquakes included site inspections/observations, testing of various materials in the building and foundations and theoretical predictive analysis of foundation and structure damage due to shaking and ground movements.

Although all of the facilities were damaged to varying degrees, this paper concentrates only on the foundation performance of the most recently built of the two large stands, the Deans Stand. More complete details of the entire stadium and the earthquake effects on it are presented in Whittaker et al. (2017) and Alexander et al. (2017). The results of further testing at the site are presented in Alexander et al (2019, in press).

This paper documents important consequences for the design of ground improvement that relies on densification to mitigate against liquefaction and in particular, the current focus on design to meet code prescribed life safety performance is questioned in the context of economic repairability considerations.

2 LANCASTER PARK STADIUM

2.1 Description

At the time of the Canterbury earthquake sequence Lancaster Park stadium comprised four main spectator stands and associated access structures including ramps, stair towers and link bridges. The Hadlee (north), Tui (south), Paul Kelly (west) and Deans (east) stands are named after either prominent local sports people or commercial sponsors. Figure 1 shows the Deans Stand which is the subject of this paper.

2.2 Typical Ground Profile

A generalized pre-earthquake soil profile beneath Lancaster Park is presented in Table 1. The sand unit presents a wide range of relative densities and varying silt content. As a result, parts of the unit are more susceptible to liquefaction than others. Groundwater levels at the site are high, typically within 1-2m of the ground surface.

Figure 1: Deans Stand after Canterbury earthquake sequence

Table 1: Generalised Ground Profile

Soil layer	Depth to top of layer (m)	SPT N value (measured)
Upper Silt, with sand layers, some organics, soft in places*	0	2-15 (5 typ.)
SAND, loose to dense interbedded layers	2-3	3-50+ (22 typ.)
LOWER SILT, soft	16-19	0-24 (7 typ.)
Riccarton GRAVEL, dense to very dense	23-24	50+

* Partially replaced with hardfill during construction

2.3 Foundation and Structural Form

Deans Stand, designed in 2008 and opened in 2010, was a three level covered stand, curved in plan. The stand superstructure comprised radial shear wall/frame structures and a circumferential moment frame along the length of the stand (refer to Fig. 2). Foundations beneath the main stand were a composite system comprising shallow reinforced concrete ground (grade) and tie beams on stone columns. Ground improvement was undertaken to limit static settlement and to protect the structure against liquefaction induced ground and foundation failure.

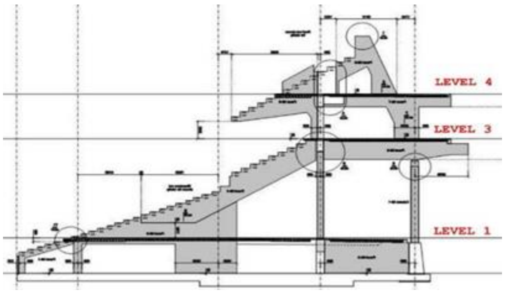

Figure 2: East-West section through Deans Stand

This ground improvement was constructed only beneath the footprint of the main part of the Deans Stand structure. It comprised vibro-replacement stone columns extending to a depth of approximately 9m below ground and concentrated in bands centred under the structural grids. Stone columns were notionally 900mm diameter, constructed on a variable sized (up to 2.7m) triangular grid. The minimum area replacement ratio was approximately 10%. Heavier structural loads were concentrated on defined rows of stone columns. The stone columns are stiffer than the ground around them, so they carry a large proportion of the gravity load from the overlying structure. They improve shallow bearing capacity and reduce settlement under static loads. The liquefiable ground below the stand was only treated to partial depth, with the intention of creating a non-liquefying crust or soil raft. A raft slab was not provided at ground level; instead, flexible pavement was constructed between the radial and circumferential ground beams.

The foundations for the adjacent access ramp hall, stair towers, link structures and an upper level hospitality lounge comprised screw piles extending to a depth of approximately 15m below ground level, with reinforced concrete pile caps and tie beams.

3 PERFORMANCE OF SITE AND STRUCTURES

Widespread damage, a result of both earthquake shaking and foundation/ground settlement occurred to all of the Lancaster Park Stadium structures. The area around the stadium experienced considerable liquefaction due to the 22 February 2011 earthquake. Large amounts of ejected sand (sand boils) were observed in the area. Significant total and differential settlement of commercial and residential buildings was evident, together with uneven land surfaces.

Ground level changes and building settlements were measured by level surveys and by reference to LiDAR data from before and after the earthquakes. Marked differential settlement was apparent between elements of structures on different foundation types, and across the structures themselves. The main body of the Deans Stand settled between 200 and 500mm, with each end tilting backwards around 100mm and the central portion settled approximately 300mm relative to the ends and tilted towards the field by 150mm. Those parts of the Deans Stand supported on screw piles settled less, typically 150 to 200mm.The pattern of settlement is shown on Figure 3. Examples of surface expressions of ground damage at the Deans Stand are shown in the photographs in Figure 4.

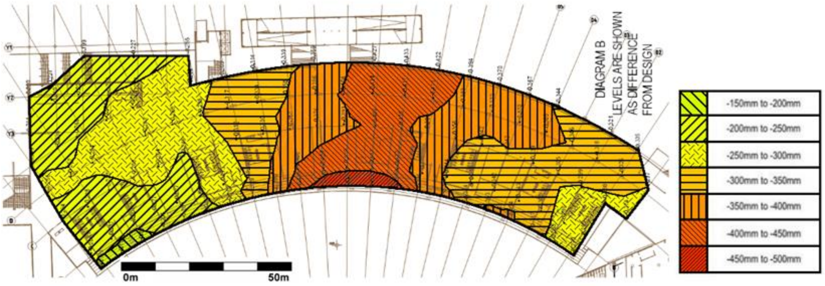

Figure 3: Deans Stand surveyed settlement contours

Assessment of liquefaction potential and resulting settlement occurring within and beneath the stone column zones from the February 2011 earthquake used the method of Idriss et al. (2008). It indicated that significant liquefaction occurred within the stone column zone. Analysis suggested that the upper 3m of the sand layer in the stone column zone liquefied, (3-6m depth) and that where the ground improvement was more successful (6-9m depth) the sand was much less susceptible to liquefaction. Below the ground improved zone, the sand was determined to be more susceptible to liquefaction.

Figure 4: Ground surface disruption (left); pavement bulging between ground beams (right)

4 INVESTIGATIONS

The February 2011 earthquake significantly exceeded design ground shaking levels, and liquefaction was evident from field observations of ejecta and from the settlement which occurred between, beneath and beyond the stone columns. The field investigations comprised test pits to expose the upper portion of stone columns beneath and adjacent to structures, boreholes advanced through the stone columns, and cone penetration testing of the ground between the stone columns and in the playing field, well beyond the stone column treatment area. Laboratory testing of clean and contaminated stone column material was undertaken.

Numerical analysis of pore pressure migration and of stone column performance under applied structural loads was carried out. Pre– and post-earthquake conditions were modelled when studying the structural load effects, including consideration of stone column performance under a future design level earthquake. Further details of this analysis are summarized in Whittaker et al. (2017).

5 INVESTIGATION FINDINGS

The investigations demonstrated that earthquake shaking and liquefaction had affected and damaged the stone columns in a number of ways. The stone columns were contaminated with fine silty sand as liquefaction ejecta flowed in to and upwards through them. It was estimated that this has reduced their drainage performance by approximately 50%, and they were left less able to relieve excess pore water pressure build-up in future earthquake shaking. The drainage functionality of stone column systems was not accounted for in the original design.

The stone columns beneath the stands had bulged into the surrounding liquefied soils resulting in them dilating and losing strength and stiffness. This aspect was investigated numerically using simplified finite element model simulations of a single column, as described in Whittaker et al. (2017).

The strength reduction within the stone columns was compounded by contamination with fine grained soils, and the net effect was estimated to be in the order of 15 to 20% loss of original strength. As a result, they were considered to be less able to support structural loads under a future liquefaction event. This loss of capacity is expected to inevitably lead to additional settlement of heavily loaded stone columns during future earthquake shaking.

The perimeter stone columns and the lower portion of the stone columns directly under the stands have been subjected to pore water pressure migration from the surrounding and underlying liquefied soils. When combined with earthquake induced shearing, dilation and loosening of these columns is expected to have occurred. This reduces the ability of the perimeter columns to protect the internal columns from the effects of liquefaction beyond the stands. It also reduces the ability of the lower portion of all of the columns to prevent liquefaction between them and thus increases the potential for bulging of the lower part of the columns in future earthquakes.

Liquefaction between the stone columns reduced the confinement of the columns markedly. Following dissipation of excess pore water pressures, any increase in lateral confining pressure developed during installation has been lost, with confinement recovering from the liquefied condition to the equivalent of normal consolidation. It was concluded that around 30 to 35% loss of confinement of the stone columns has occurred.

When considering future performance of the ground, the investigations carried out beneath the Deans Stand to date indicate that liquefaction that has occurred between the stone columns has led to reduction of the lateral stress and loss of the densification of the surrounding ground achieved by stone column construction. While the loss of densification may not be vital for static performance of the foundation system, it was assessed to have significantly reduced performance during future earthquakes. Two of the more fundamental findings are expanded on in the following sections.

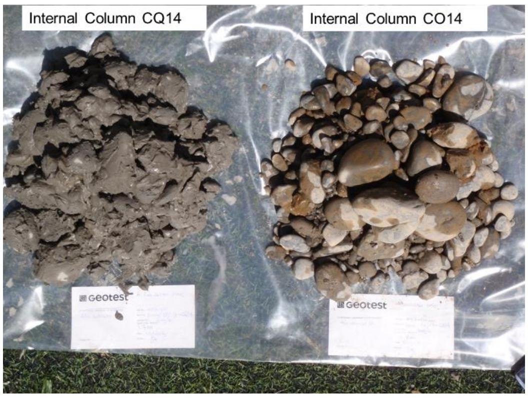

Figure 5: Dirty and clean samples of stone column material

5.1 Contamination

Most of the stone columns exposed in pits beneath the Deans Stand showed evidence of fine silty sand contamination, while most of the pits excavated beyond the stand on the field side show the columns to be relatively clean. Photographs of samples recovered from near the top of stone columns are presented in Figure 5, and deeper samples from bores put down through columns are shown in Figure 6.

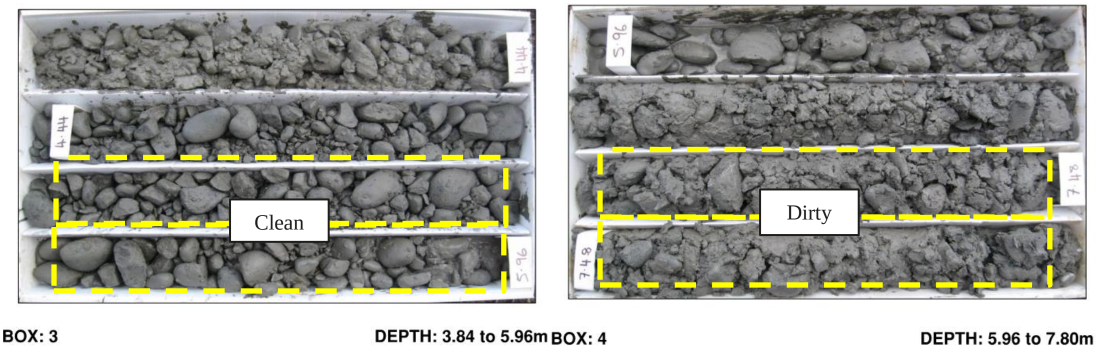

Figure 6: Dirty and clean samples of stone column material – deeper samples (BH3001)

It is postulated that the weight of the structure drove liquefied soils beneath and surrounding the stone columns into and up the columns directly beneath the stand, and that the effect was much less pronounced within the perimeter columns.

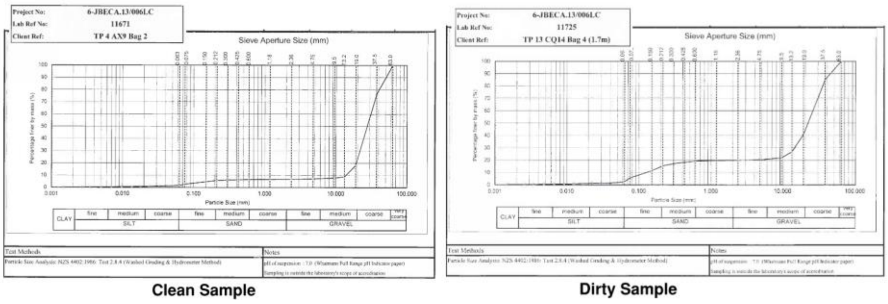

Figure 7: Grading curves of clean and dirty stone column materials (shallow samples)

The stone columns would inevitably have contained a proportion of silty sand introduced from the surrounding ground during the construction process. That fine "contamination" is the residue from flushing by large volumes of water during construction, so is expected to contain a much smaller proportion of silty sand than liquefied materials entering the columns following strong earthquake shaking. It is inferred that construction induced “contamination” would readily be mobilized by groundwater flow from the surrounding soil under earthquake shaking.

It is further postulated that the stone columns filled with fine silty sand during the initial significant liquefaction event. Some of that material was observed to have made its way through the ground level pavement in the Deans Stand and manifested as ejecta causing bulging of the asphalt surface (Figure 4). The liquefied sand is assumed to have stopped flowing once liquefaction induced subsidence ceased. A comparison of gradation curves is shown in Figure 7 and clearly demonstrates the higher fines content in the contaminated (dirty) sample.

5.2 Liquefaction between stone columns

Stone column construction densifies surrounding sandy soils, locking in increased lateral stresses. Liquefaction of soil between the columns leads to the loss of these locked in stresses, returning the ground to a normally consolidated state. Furthermore, the post-shaking dissipation of excess pore water pressure from liquefied layers upwards and downwards to non-liquefied layers leads to a degree of strength loss in those non-liquefied layers.

We initially investigated four locations at the Deans Stand, carefully exposing groups of stone columns so we could locate cone penetration tests (CPTs) mid-way between columns. More recent investigations, (Alexander et al. 2019, in press) have included direct push cross hole (DPCH) and shear wave velocity testing at three locations in the Deans Stand and one location in the playing field. The test results show the ground between the stone columns to largely have returned to its pre-improvement, normally consolidated, state. This is demonstrated by the upper and lower quartile plots of CPT tip resistance presented in Figure 8, which are overlaid on the equivalent range of pre- and post-improvement CPTs.

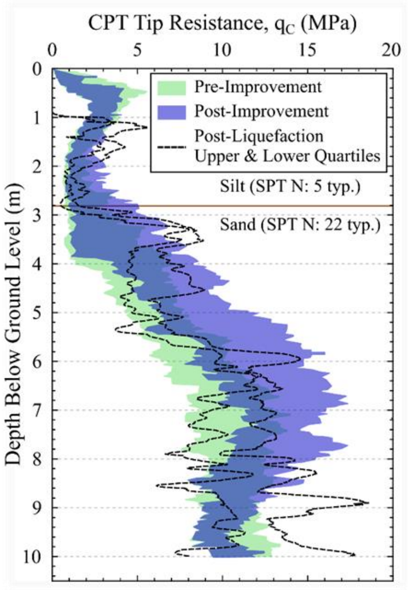

Figure 8: CPT data comparison

6 DISCUSSION

The design intent for the stone column portion of the composite foundation systems supporting both main stands at Lancaster Park stadium was similar – to form a non-liquefying crust that was thick and extensive enough to prevent global instability and reduce post-earthquake settlement. At Deans Stand, there was also a requirement for the stone columns to strengthen and stiffen the near surface silts to limit gravity load settlement to 25mm.

The ultimate design level of earthquake shaking was exceeded by the February 2011 earthquake, with no global failure of the main stands. On that basis, the stone column ground improvement system has achieved its primary (life safety at ULS) design objective. Damage to the stone columns and the ground between them does, however, appear to have significantly compromised their future performance.

Most clearly, the stone columns have been contaminated by silty fine sand. The partial depth of treatment appears to have contributed to this contamination – a direct connection was available to the liquefied ground beneath the columns.

Less obviously, but clearly apparent from our investigations, the densification of the surrounding sandy soil achieved by stone column construction has been lost as a result of the ground improvement zone “capacity” or design basis having been exceeded. This aspect is of particular relevance for any ground improvement project that relies on densification and that is designed for a relatively low return period event – for example for the same return period as a normal importance level building.

Well-designed structures may, in many cases, be economically repairable following an earthquake that exceeds design loadings. Ground beneath them that has been improved by densification and has subsequently lost that densification is much more difficult (and expensive) to repair. In the case of the Deans Stand, a relatively expensive jet grout lattice repair solution was developed (but not implemented), as it was not physically possible to re-densify the stone column zone without demolishing the stand.

Partial depth ground improvement to mitigate liquefaction can appear to offer significant cost advantages over full depth treatment. In this case, it met its objective – a crust was maintained, even with liquefaction occurring within the stone column zone, and life safety achieved. Total and, therefore, potential differential settlement is larger than with full depth treatment, and will be more difficult to predict reliably. As a result, there is greater potential for total economic loss following a ULS event, particularly where no structural raft slab is provided.

Comparison of the measured pattern of settlement at the Deans and Paul Kelly Stands provides valuable insights into the benefits of a structural raft slab as part of a composite foundation system where liquefaction is expected. Paul Kelly Stand, with a structural raft beneath the heavily loaded portion, remained essentially “straight” along both major axes and predominantly tilted backwards on the underlying liquefied ground in response to the differing gravity load across its footprint. Such a raft slab more effectively confines ejecta than a flexible pavement, and can stiffen the structural response to differential settlement. Deans Stand, with no structural raft, was subject to sagging and twisting in response to local stone column bulging under concentrated structural loads from the ground beams together with settlement resulting from different levels of gravity load and varying thicknesses of liquefied ground beneath its footprint.

7 CONCLUSIONS

The performance of the Deans Stand at Lancaster Park in the February 2011 Christchurch earthquake has provided some valuable data and insights into the performance of stone column ground improvement that has been subjected to shaking levels considerably greater than design values.

While the overarching life-safety as the primary design intent of the ground improvement was achieved, the stone columns have been sufficiently heavily damaged that they cannot be relied upon during a future design event. This has had a significant effect on the economic viability of stadium repair. There appears to be merit in designing ground improvement measures that rely on densification to a higher level of earthquake shaking than the building they are supporting. The authors suggest consideration of an economic repairability design level for these measures, sitting somewhere between ULS and the structural collapse shaking levels.

Elimination of a raft slab, and adoption of partial depth ground improvement, present capital cost savings that may be attractive in the early stages of a project. Both of these apparent cost savings can, however, impact on the economic viability of repairs following a large earthquake.

8 ACKNOWLEDGMENTS

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9 REFERENCES

Alexander, G.J., Arefi, J. & Martin, G.R. 2017. Performance of a stone column foundation system subjected to severe earthquake shaking, *3rd International Conf. on PBD in earthquake geotechnical engineering, Vancouver*.

Alexander, G.J., Arefi, J., Wotherspoon, L.M., Stolte, A.C., Cox, B.R. & Green, R.A. (in press). *A case study of stone column ground improvement performance during a sequence of seismic events*.

Idriss, I.M. & Boulanger, R.W. 2008. *Soil Liquefaction During Earthquakes*, MNO-12, EERI.

Standards New Zealand. 2004. *Structural design actions Part 5: Earthquake actions – New Zealand*, NZS1170.5:2004, Standards New Zealand, Wellington, NZ.

Whittaker, D., Alexander, G., Barnett, J. & Charman, N. 2017. Assessment and restoration of an earthquake-damaged sports stadium in New Zealand, *16th World conference on earthquake engineering, Santigao*.