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A case study of stone column ground improvement performance during a sequence of seismic events

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ABSTRACT: The Deans Stand at Lancaster Park (formerly, AMI Stadium) in Christchurch, New Zealand is of modern reinforced concrete design and construction. It is largely supported on a hybrid foundation system comprising ground beams and stone column ground improvement that extend 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. Subsurface distress included bulging, softening and contamination of the stone columns, and loosening of the densified ground between them. This paper presents insights into the behaviour of stone columns subjected to shaking levels considerably greater than design values. Cone penetration testing (CPT) and direct push cross-hole testing (DPCH) were used to characterise the stone columns, the soil deposits between the stone columns and the virgin soil outside the improved zone. The investigations indicate that liquefaction reversed the beneficial effects achieved by stone column installation. This has important consequences for the design of stone columns used to mitigate liquefaction.

1 INTRODUCTION

The Canterbury Earthquake Sequence (CES) of 2010-2011 caused widespread significant damage to buildings and infrastructure across Christchurch City. The most severe shaking was due to the 22 February 2011 moment magnitude (M_w) 6.2 earthquake, which had an epicentre approximately 6 km from central Christchurch (Bradley and Cubrinovski 2011). Lancaster Park Stadium (formerly, AMI Stadium) was the largest sporting and events venue in Christchurch, with a seated capacity of approximately 40,000. The stadium sustained substantial damage to foundations and structure as a result of the CES, and in particular the February 2011 earthquake. The stadium has been closed since that time, and as of 2018 is planned to be demolished. Engineering damage assessments following the earthquakes included site inspections/observations, testing of various materials in the building and foundations and numerical back-analyses of foundation and structure damage due to shaking and ground movements experienced during the earthquakes.

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. It focuses, in particular, on the behavior of stone columns and the improved ground between them under strong earthquake shaking that significantly exceeded design levels. More complete details of the entire stadium and the earthquake effects on it are presented in Whittaker et al. (2017). Alexander et al. (2017) provides a wider discussion on the performance of the foundation systems and other effects on the stone columns. This paper documents one element of an important and rare case history of the seismic performance of foundations bearing upon ground treated with vibro-replacement stone columns.

2 LANCASTER PARK STADIUM

At the time of the CES, Lancaster Park stadium comprised four main spectator stands and associated access structures including ramps, stair towers, and link bridges; refer to Figure 1. The Deans (east) Stand (to the left in Figure 1) is the newest, and opened in 2010, just prior to the first of the CES earthquakes. Deans Stand, designed in 2008, is a three level covered stand, curved in plan, with an overall seated capacity of around 13,500 spectators. The stand superstructure comprises radial shear wall/frame structures and a circumferential moment frame along its length as shown in Figure 2.

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 across this area. As a result, parts of the unit are more susceptible to liquefaction than others. Groundwater levels at the site are high, typically within 1-2 m of the ground surface.

Foundations beneath Deans Stand are a composite system comprising shallow reinforced concrete ground (grade) and tie beams on stone columns. Portions of the structure were supported on screw piles taken to greater depth. Stone columns were installed to limit static settlement to 25mm, and to protect the structure against liquefaction induced ground and foundation failure. This ground improvement was constructed only beneath the footprint of the main part of the Deans Stand structure (Figure 2). It comprised vibro-replacement stone columns extending to a depth of approximately 9 m below ground and concentrated in bands centred under the structural grids. Stone columns were nominally 900 mm diameter, constructed on a variable sized (up to 2.7 m) triangular grid. The minimum area replacement ratio was approximately 10%. Heavier structural loads were concentrated on defined rows of stone columns. The liquefiable ground below the stand was only treated to partial depth, with

Figure 1. Lancaster Park stadium prior to the earthquakes looking south. Deans Stand is on the left of the image.

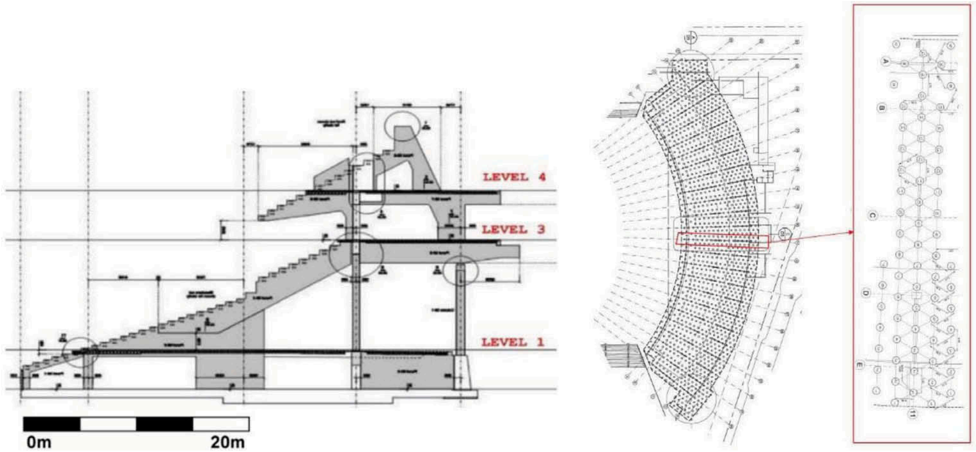

Figure 2. East-west section through Deans Stand (left), stone column general arrangement plan with an enlarged radial grid (right).

Table 1. Generalized soil profile beneath the Lancaster Park site.

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

the intention of creating a non-liquefying crust or soil raft. The stone columns are stiffer than the ground around them, so they carry a larger proportion of the gravity load from the overlying structure. They also improve shallow bearing capacity and reduce settlement under static loads. Figure 2 illustrates the general arrangement of the stone columns with a slice through the middle gridline.

3 CANTERBURY EARTHQUAKE SEQUENCE

The Canterbury earthquake sequence of 2010 and 2011 included several strong and damaging earthquakes. The most severe shaking at Lancaster Park Stadium occurred during the 22 February 2011 earthquake. Although the earthquake was of relatively moderate magnitude (M_w6.2) and the duration of strong shaking was only a few seconds, the earthquake epicentre was shallow and very close – approximately 6 km south-east of the stadium. That event resulted in peak ground accelerations (PGA) at the CCCC seismograph, 700 m northwest of the stadium, of approximately 0.5g horizontally and 0.7g vertically. 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 2500-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 1000 year shaking, represented by a M7.5 earthquake with a peak ground acceleration (PGA) of 0.32g.

4 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 greater area around the stadium experienced considerable liquefaction (large amounts of ejected sand) due to the 22 February 2011 earthquake. Significant total and differential settlement of commercial and residential buildings was evident, together with uneven land surfaces.

Ground surface elevation changes and building settlements at the stadium were measured by level surveys and by reference to LiDAR data from before and after the earthquakes. Marked differential settlement was apparent at the stadium between elements of structures on different foundation types, and across the structures themselves. The main body of the Deans Stand settled between 200 and 500 mm (refer Figure 3) following the February 2011 earthquake, with each end tilting backwards around 100 mm. The central portion settled approximately 300 mm relative to the ends and tilted towards the field by 150 mm. The pattern of settlement is shown on Figure 3. The extent of surface expressions of liquefaction (i.e., the amount of liquefaction ejecta) within the confines of the stadium was generally less than for the surrounding area.

5 INVESTIGATIONS

The February 2011 earthquake significantly exceeded ground shaking levels used to design the ground improvements beneath Deans Stand. Soil liquefaction at the site was evident from field observations of ejecta and from the settlement that occurred between and beyond the stone columns.

The initial physical investigations of ground behavior and stone column condition comprised:

- Test pits to expose the upper portion of stone columns beneath and adjacent to structures
- Machine boreholes advanced through the stone columns
- Cone penetration testing of the ground between the stone columns and on the playing field, well beyond the stone column treatment area, and
- Laboratory testing of clean and contaminated stone column material.

More recent investigations have been undertaken to specifically explore the state of the improved ground between the stone columns. Direct push crosshole testing (DPCH) has been used to provide detailed characterization of the compression wave and shear wave velocities (V_P and V_S, respectively) at the site. The DPCH testing method is the direct-push equivalent of conventional, borehole-based crosshole seismic testing method and has been well documented in Cox et al. (2019). For a schematic test setup and configuration refer to Wotherspoon et al. (2015). Following the CES, DPCH testing played a key role in ground improvement trials (Hwang et al. 2017, Wotherspoon et al. 2017), allowing for high-resolution measurements of V_P and V_S in the zones of improved soil between ground improvements and directly across ground improvement elements, which cannot be achieved with any other seismic testing methodology. At Lancaster Park, DPCH testing was conducted at three locations within the Deans Stand to develop profiles of V_P and V_S at 0.2 m depth increments in the improved ground between the stone columns and

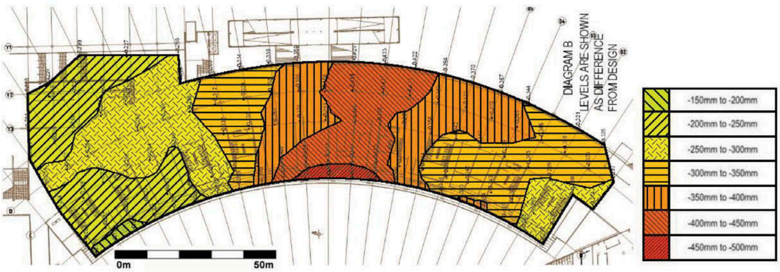

Figure 3. Surveyed settlement beneath the Deans Stand following the 2011 Christchurch earthquake.

directly across the stone columns. Additional DPCH testing was conducted at a location on the adjacent playing field to characterize the virgin (unimproved) soil properties.

6 INVESTIGATION FINDINGS

The initial investigations demonstrated that earthquake shaking and liquefaction had reversed the beneficial effects of the stone columns in a number of ways. These have been presented in Whittaker et al. (2017) and Alexander et al. (2017). At four locations beneath the Deans Stand, stone columns were carefully exposed in September and October 2015 so we could perform post-liquefaction cone penetration tests (CPTs) mid-way between columns. Figure 4 shows several of the exposed stone columns. Figure 5 summarises the range (minimum to maximum) of CPT tip resistance pre- and post-stone column installation beneath Deans Stand, highlighting the general improvement between 3-8 m depth that existed prior to the CES. The tip resistances from the 2015 investigations (post-liquefaction) show the ground between the stone columns to largely have returned to its pre-improvement, normally consolidated state. The densification and locked in lateral confining pressure developed during stone column construction has been lost. This is demonstrated by the upper and lower quartile plots of CPT tip resistance presented on Figure 5. These findings stimulated more recent DPCH testing, which is described later in this paper.

CPT-based assessment of liquefaction potential and resulting settlement occurring within and beneath the stone column zones from the February 2011 earthquake were performed using the methods of Idriss & Boulanger (2008) and the post improvement/pre-earthquake CPT results (refer Alexander et al 2017). Significant liquefaction was predicted to occur within the stone column zone. The upper portion of the sand layer, between a depth of 3-6 m, in the stone column zone was predicted to liquefy under the 0.5g PGA of the February 2011 earthquake. The ground improvement in the sand layer was more successful between 6-9 m depth, where the soil was naturally stiffer, and the sand was much less susceptible to liquefaction. Below the improved zone, the sand was determined to be more susceptible to liquefaction.

In 2018, 7 years after the 2011 Christchurch earthquake, DPCH testing was conducted at three locations at the Deans Stand to characterize the stiffness of both the improved ground across and between the stone column elements. Additional DPCH testing was conducted in the unimproved

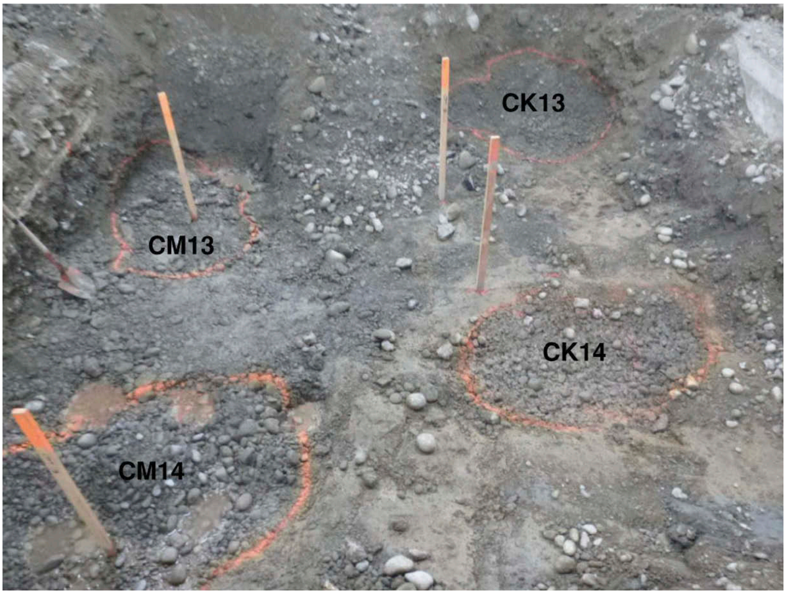

Figure 4. Exposed stone columns beneath Deans Stand (2015 investigations).

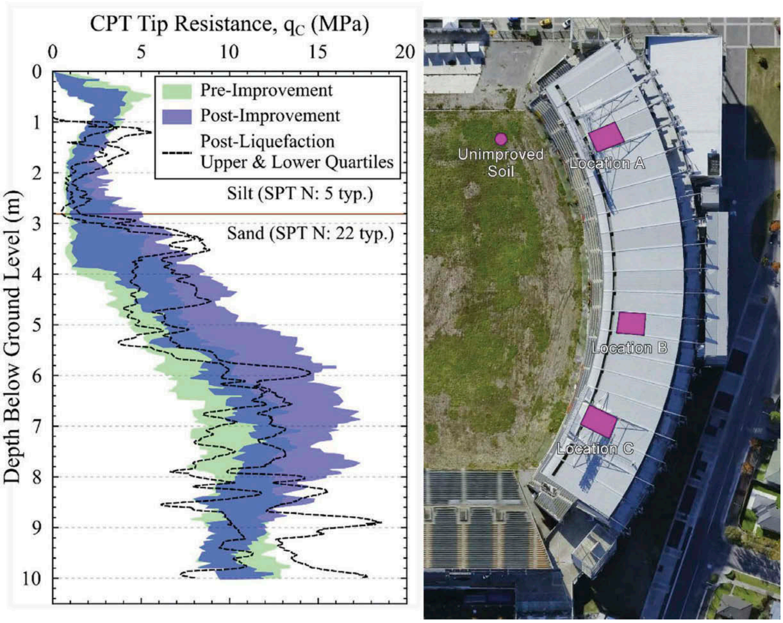

Figure 5. Comparison of CPT tip resistance pre- and post-ground improvement installation during the construction, and those post-CES (7 years after the 2011 Christchurch earthquake) (left), DPCH testing schematic plan (right).

soil at the nearby playing field. The results of these investigations, profiles of V_P and V_S, are presented together in Figure 6. The increased composite stiffness of the stone columns and surrounding soil is indicated by higher V_S values relative to the unimproved ground. However, the V_S of the improved ground between the stone columns is on average similar to that of the unimproved ground, again providing evidence that post-liquefaction, the improved ground between the stone columns has largely returned to its pre-improved state. Two metres below the ground surface, at the expected hydrostatic water table, the unimproved ground and the improved ground between the stone columns at two of the Deans Stand DPCH locations is fully saturated, as indicated by V_P values greater than 1500 m/s (i.e., the P-wave velocity of water). However, across the stone column elements and between the stone columns at location C, the soil does not achieve a V_P value greater than 1500 m/s until much deeper (5-7 m). This suggests a greater depth of partial saturation and is inferred to be due to the increased fines content in the stone columns and surrounding ground, as noted in Alexander et al. (2017). At many other sites, soils have been shown not to be saturated for many metres below the static water table using comparisons between BH/CPT defined depths and depths defined based on a V_p=1500 m/s. This is demonstrated in the virgin soil investigations described in Wotherspoon et al (2017).

Stone column construction is meant to densify surrounding sandy soils and increase lateral stresses. The upward pore pressure migration from the liquefied, unimproved ground below the stone columns and the partially improved soil at depths of 3-6 m between the stone columns are inferred to have reduced the confinement of the columns markedly. This leads to the conclusion that, following dissipation of excess pore water pressures, the increased lateral confining pressure developed during stone column construction was reduced, with confinement being essentially equivalent to normally consolidated soils. This aspect is the main focus of our recent investigations. Most critically, when considering future performance of ground that has been improved by stone columns, the investigations carried out beneath the Deans Stand to date indicate that liquefaction that occurred between the stone columns reversed the beneficial effects achieved by stone column construction, significantly reducing performance of the improved ground during future earthquakes.

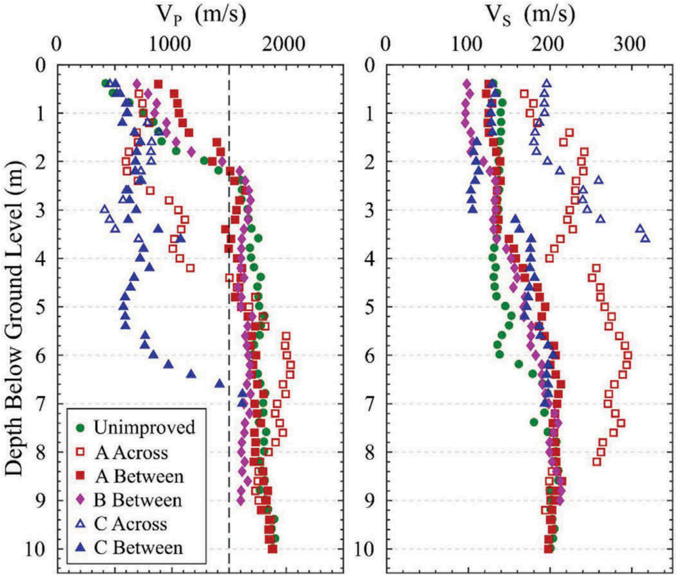

Figure 6. Comparison of Vp and Vs from DPCH testing.

7 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 present to a depth of 2-3 m to limit gravity load settlement to 25 mm. 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 achieved its primary (life safety at ULS) design objective. However, reduction of confinement around the stone columns and loss of the improved density and lateral stresses in the intervening soil will likely significantly compromise the future performance of the improved ground. The partial depth of treatment appears to have contributed to reversal of the beneficial effects of the stone columns, with the upward migration of excess pore water pressures in the liquefied substratum likely exacerbating the liquefaction in the soil between the stone columns.

Well-designed buildings may, in many cases, be economically repairable following an earthquake that exceeds design loadings. Ground beneath them that has been improved by stone columns that have been shaken well beyond their design basis is much more difficult (and expensive) to repair. In the case of the Deans Stand, a concrete lattice solution was proposed to remediate the soil after the CES because it was not physically possible to regain the beneficial effects of the stone columns without demolishing the stand. The lattice solution has not been implemented, as the stadium is scheduled to be demolished.

Partial depth ground improvement to mitigate liquefaction can appear to offer significant cost advantages over full depth treatment. In the case of the Deans Stand, the stone columns installed in the upper portion of the liquefiable layer served their intended purpose – a crust was maintained, even though liquefaction occurred within the stone column zone, and the design performance level was achieved. Total and, therefore, potential differential settlement will likely be larger if only a portion of the stratum predicted to liquefy is treated, relative to the full treatment of the stratum, with the settlement in the partially treated stratum being difficult to predict reliably. As a result, there is greater potential for total economic loss following a ULS event if a partial treatment scheme is employed.

8 CONCLUSIONS

The performance of the Deans Stand at Lancaster Park (formerly, AMI Stadium) in the February 2011 Christchurch earthquake has provided 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 design performance level 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.

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REFERENCES

Alexander, G.J., Arefi, J., and Martin, G.R. 2017. Performance of a stone column foundation system subjected to serve earthquake shaking. *Third International Conference on Performance-Based Design in Earthquake Geotechnical Engineering*, Vancouver, Canada

Bradley, B. A. and Cubrinovski, M. 2011. Near-source strong ground motions observed in the 22 February 2011 Christchurch earthquake. *Seismological Research Letters*. 82(6): 853–865.

Canterbury Earthquakes Royal Commission. 2012. *Final Report Volume 1 Summary and Recommendations in Volumes 1 to 3, Seismicity, soils and the seismic design of buildings*, ISBN: 978-0-478-39559-4.

Cox, B.R., Stolte, A.C., Stokoe, K.H., II, and Wotherspoon, L.M. 2019. A Direct-Push Crosshole (DPCH) Test Method for the In Situ Evaluation of High-Resolution P- and S-Wave Velocities. *Geotechnical Testing Journal*. doi:10.1520/GTJ20170382. in press.

Hwang, S., Roberts, J. N., Stokoe, K. H., Cox, B. R., van Ballegooy, S., and Soutar, C., 2017. Utilizing Direct-Push Crosshole Seismic Testing to Verify the Effectiveness of Shallow Ground Improvements: A Case Study Involving Low-Mobility Grout Columns in Christchurch, New Zealand. *Grouting 2017*, No. 288, 415–424.

Idriss, I.M. Boulanger, R.W. 2008. *Soil Liquefaction During Earthquakes, MNO-12*, Earthquake Engineering Research Institute.

Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, J.D., Kayen, R.E. and Faris, A. 2003. Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework, U.C. Berkeley EERC Report No. 2003-08.

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.J, Barnett, J. and Charman, N. 2017. Assessment and Restoration of an Earthquake-damaged Sports Stadium in New Zealand, *16th World Conference on Earthquake Engineering*, Santiago, Chile, Paper No 914.

Wotherspoon, L.M., Cox, B.R., Stokoe, K.H., Ashfield, D.J., and Phillips, R.A. 2015, Utilizing direct-push crosshole testing to assess the effectiveness of soil stiffening caused by installation of stone columns and Rammed Aggregate piers

Wotherspoon, L.M., Cox, B.R., Stokoe, K.H., Ashfield, D.J., and Phillips, R. 2017, Assessment of the Degree of Soil Stiffening from Stone Column Installation Using Direct Push Crosshole Testing, *16th World Conference on Earthquake Engineering*, Santiago, Chile