Use of centrifuge modelling to improve lessons learned from earthquake case histories.

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Abstract: Current procedures relate potential liquefaction induced settlements to foundation size and liquefiable depth. However, the analysis of field case histories suggests that the influence of foundation bearing pressure may also be a significant factor influencing such phenomena. Data from 24 buildings that suffered settlement and tilting as a consequences of soil liquefaction during the February 27th 2010 Maule earthquake in Chile, are herein analyzed and compared with data from other earthquakes. Although case history data play a crucial role in geotechnical earthquake engineering, in many cases their analysis is limited to speculation therefore experimental verification is often required. Thanks to the significant development in dynamic geotechnical centrifuge modelling in the last 30 years, we are today able to carefully reproduce field motions enhancing the reliability of experimental results. This is the case for the centrifuge tests discussed in this paper, which have been the first tests performed on the University of Dundee’s new Actidyn Q67-2 servo-hydraulic earthquake shaker.

Keywords: Liquefaction, Earthquake centrifuge modelling, Shallow foundation, Servo-hydraulic shaker.

1 INTRODUCTION

Exploring the literature of earthquake geotechnical engineering there are many instances in which hypotheses are formulated based on earthquake case histories. However, due to the large number of variables associated with natural phenomena, it is not always possible to isolate the effect of a given variable on the dynamic behaviour of the ground/structure of interest. For this reason, case history analysis is often limited to speculation. During earthquakes, the behaviour of structures resting on the ground surface (or buried underground) is a function of several variables, such as soil properties, motion characteristics, structure characteristics, etc., and their interaction. Seismic soil liquefaction represents a dramatic example of such a phenomenon; as a consequence of its degree of complexity its effects on the built environment are not fully understood yet. In this framework, geotechnical dynamic centrifuge modelling provides a valuable tool for research by reproducing field conditions in a controlled environment.

Earthquake centrifuge modelling started in the late 1970's with the development of explosive-based actuation systems (e.g. Zelikson et al., 1981) and the first spring-actuated shaker (Morris, 1979). This latter device was a relatively inexpensive and robust mechanical shaker, exploiting a reaction mass connected to a pre-stressed spring which was then released in-flight. Since then several type of actuators have been developed, each improving on the level of control possible, such as the "bumpy road" actuator (Kutter, 1982) and the SAM (Stored Angular Momentum) actuator (Madabhushi, 1998). However, mechanical shakers produce single tone burst with limited reliability and repeatability, and usually have reduced flexibility in terms of frequencies, amplitude, and duration of the motion. In addition, such actuators will often display high levels of acceleration at frequencies that are harmonics of the intended tone. With the development of fast-acting servo-valves servo-hydraulics actuators have gradually replaced mechanical ones to overcome their limitations. Servo-hydraulic actuators are able to better reproduce multi-frequency motions, and therefore they can, within their technical limits, replicate a given historic earthquake motion. Shakers with different numbers of degrees of freedom have been developed, ranging from simple mono-directional shakers to three-dimensional shakers including the vertical component of an earthquake motion. The centrifuge mounted earthquake simulator at the University of Dundee is an Actidyn Q67-2 mono-directional servo-hydraulic shaker (Figure 1) with a payload capacity of 400kg capable of reproducing a scaled earthquake motion containing frequencies within the window 40 to 400Hz (0.4 to 4Hz prototype frequency at 100g or 0.8 to 8Hz at 50g).

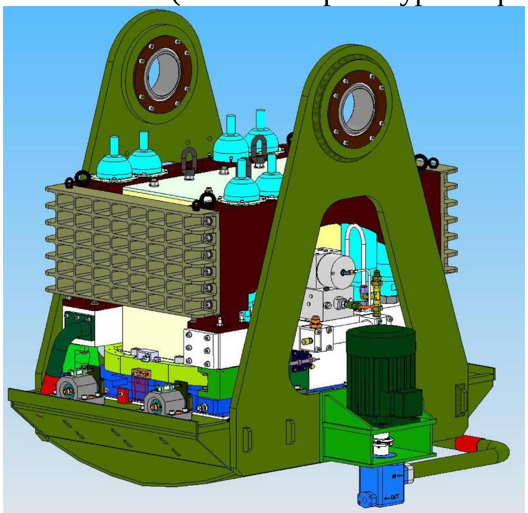

Figure 1. University of Dundee's Actidyn Q67-2 centrifuge mounted earthquake simulator.

Figure 2 shows a typical recorded table acceleration (green trace) compared to the desired motion (brown trace, E-W component of the Maule earthquake at the Concepción station, scaled at 50g), both in time and frequency domain; the device is able to reproduce the motion with a RMS error of 3.46%. Such devices enable reproduction of field motions and hence provide the opportunity to more confidently affirm or reject hypotheses based on case histories.

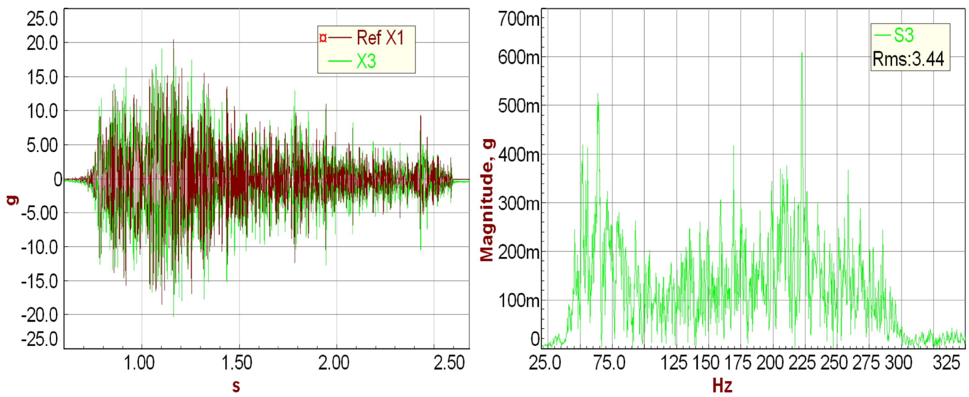

Figure 2. Shaker reproduction of EW component of Maule earthquake motion (San Pedro de la Paz station) at 50g. Ref X1, Reference motion; X3, table acceleration (inverse polarity); S3, frequency spectrum.

The aim of this paper is to check a hypothesis, based on data collected following Chile's 2010 Maule earthquake concerning the liquefaction induced settlement of buildings with shallow foundations. Centrifuge modelling is used in order to assess a specific trend observed in the collected field data in which the settlement of shallow foundations on liquefiable soil was seen to depend significantly and nonlinearly on bearing pressure.

2 FIELD EVIDENCES

Field data on the settlement of buildings with shallow foundations as a consequence of the liquefaction of the foundation soil are scarce and often incomplete. Most of the reconnaissance reports following major earthquakes involving soil liquefaction include only settlement measurements in the absence of adequate information on the underlying soil properties. To be interpreted correctly, settlement measurements must be complemented by a careful estimation of the extent of liquefaction (thickness of liquefied soil) at the specific site. This is possible only if a few essential properties are known in addition to the ground motion characteristics, such as: soil type and relative density, fines content and characterization, water table depth. This is further limited by the fact that soil properties are generally measured after the earthquake, rather than before, when it is known that a particular site is of interest.
For this study, data collected during a field campaign which took place in the city of Concepción (Chile) after the 2010 Maule earthquake are used. This allowed the creation of a database of buildings that suffered excessive settlement due to soil liquefaction in this specific event. Moreover, two additional datasets on the liquefaction induced settlement of shallow foundations were selected from the literature to complement these data, namely those found in Yoshimi & Tokimatsu (1977), concerning the Niigata earthquake of 1964, and in Adachi *et al.* (1992) and Acacio *et al.* (2001), concerning the Luzon earthquake of 1990 in the Philippines.

2.1 Maule earthquake case study

The 8.8 moment magnitude (M_w) Maule earthquake hit Chile on the February 27th 2010 at 6.34 am (UTC time). The hypocentre was at a depth of 30.1 km and was located offshore the coastal town of Cobquecura, around 100 km north of the city of Concepción, at latitude 36.29ºS and longitude 73.239ºW (Barrientos, 2008). The earthquake affected a very wide area because of the considerable length of the seismogenic fault, which was estimated to be around 500 km along the Pacific Coast and 100 km wide. Despite significant earthquake induced ground failures over the entire affected area, the occurrence of soil liquefaction was relatively limited considering the high magnitude of the Maule earthquake. The seasonal variation of the water table, which reaches its minimum during the summer, may have contributed in reducing the liquefaction risk in many areas.
The city of Concepción and its surroundings were among the locations mostly affected by liquefaction. This can be ascribed to the geological setting of this area characterized by the presence of the Bío Bío River delta which has formed wide lowlands along the Pacific coast by depositing Quaternary soils. The Bío Bío River, one of the longest and largest of the country, has been continuously depositing basaltic sand carried from the Andes around the mouth area since before the retreat of the sea. The Bío Bío sand is in general clean (*i.e.* without fines content and uniformly graded) although it can be found mixed with silts deposited by the Andalién River or with soils resulting from weathered rocks. These soil deposits when saturated and loose present a high potential for liquefaction. Soil liquefaction risk has been observed to be higher in proximity of the many water bodies, such as swamps and lagoons, characterizing the most recent deposits in the area. Further details of the event may be found in Verdugo *et al.* (2010) and Villalobos *et al.* (2010).
Several cases of liquefaction induced ground failures were observed in the Concepción area ranging from lateral spreading of sloping ground and bridge abutments to building settlement and tilting and uplift of

buried structures (e.g. sewage tanks and manholes). This paper is concerned with the behaviour of shallow foundations on liquefied ground for level ground conditions, therefore only the cases meeting this description are considered. A total of 23 buildings were selected. None of the surveyed buildings has a basement, resulting in a maximum footing embedment of 1 m, and all consist of isolated buildings with no adjacent structures that might influence their behaviour. The analysis of damaged reinforced concrete buildings included the review of building and foundation plans and the evaluation of the foundation soil liquefaction potential. Local construction companies and surveyors provided data regarding settlement and tilting of buildings as well as ground exploration data consisting of SPT profiles, USCS soil classification and sieve analysis. It is worthwhile to note that most of the ground exploration data were collected pre-earthquake. This is very important especially for the estimation of the soil's relative density which may have since increased significantly due to densification induced by ground motion.
Since lateral variability of soil profiles may be relevant, one of the main criteria in the analysis has been to have localized ground exploration information for each of the buildings considered. Therefore damaged buildings were excluded where no relevant borehole (and hence SPT investigation) was available in their proximity. The liquefaction potential of the foundation soil was evaluated by means of the simplified procedure proposed in Youd et al. (2001). This method has evolved through the years and has become standard practice; it consists in the calculation of an index for the susceptibility to liquefaction (FS_L= *Cyclic Resistance Ratio* (CRR)/*Cyclic Stress Ratio* (CSR)) based on the ground motion magnitude, peak acceleration and the site's SPT blow-counts. Soil having a FS_L <1 is considered liquefiable (Figure 3).

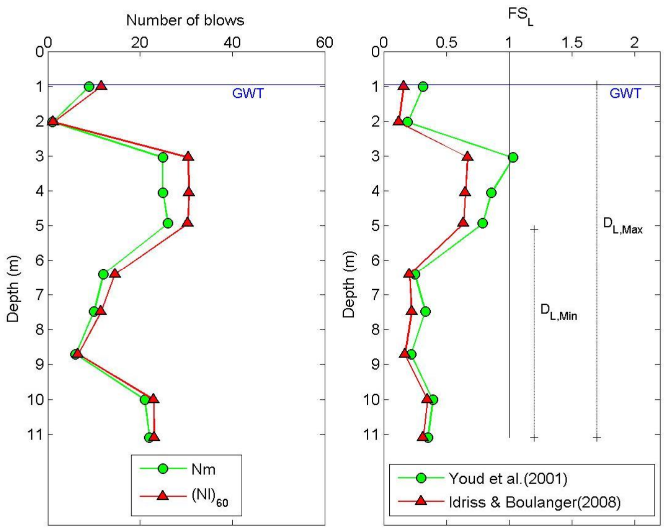

Figure 3. SPT profile (building "Los Presidentes", Concepción) and FS_L calculation.

2.2 Field data analysis

The analysis of the collected data together with those available in the literature suggests how different parameters, such as building width *B*, building bearing pressure *q* and depth of liquefaction D_L, influence the liquefaction-induced settlement of shallow foundations. Data collected in Chile after the 2010 Maule earthquake are in good agreement with those from the Niigata and Luzon earthquake, although in the Maule case the liquefaction-induced settlement seems to be of smaller magnitude with respect to the other two events. This may be due to the fact that the average depth of liquefaction estimated in the Chilean cases (D_L=3.75m) was smaller than those recorded in Japan (D_L=11.4m) and the Philippines (D_L=8.7m). However, considering the normalized settlement (absolute settlement over liquefied soil thickness, S/D_L) the data from the three different events become comparable. The normalization of settlement by the depth of liquefaction is justified by the hypotheses that a direct proportionality between the induced settlement and the depth of liquefaction at the site. However, as suggested by Dashti *et al.* (2010), this proportionality might not be linear. In particular for small depth of liquefaction D_L the normalized settlement appears to be greater with respect to similar cases having higher D_L. For these reasons, the use of normalized settlement in the analysis of such data may be in some cases misleading if not accounted for correctly. Two main trends have been noticed in the selected data:

- The liquefaction-induced settlement (or normalized settlement) reduces with increasing building width (Figure 4).

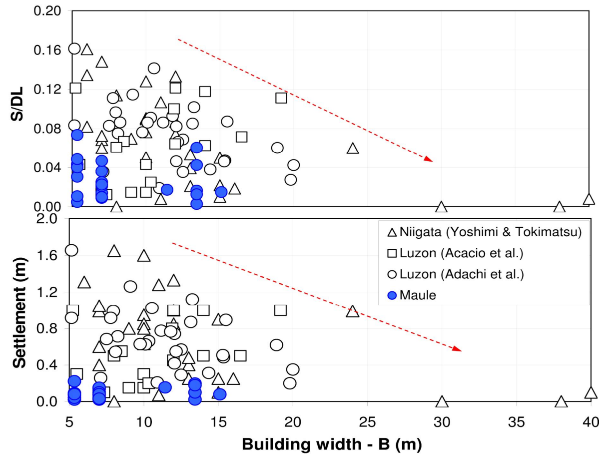

Figure 4. Influence of building width on its liquefaction-induced settlement.

- Settlements increase with foundation bearing pressure, up to a point. However, beyond that point, higher and/or heavier buildings (whose foundations exert a higher bearing pressure on the ground) suffer smaller settlements (or normalized settlement) with respect to lighter ones (Figure 5).

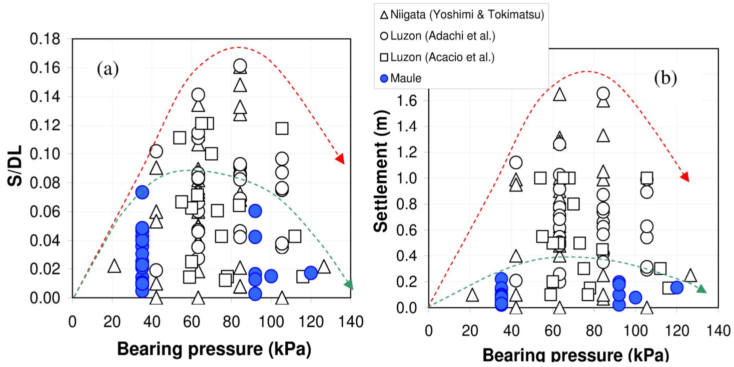

Figure 5. Influence of building bearing pressure on its liquefaction-induced settlement.

While the first trend has been already confirmed by several experiments (Yoshimi & Tokimatsu, 1977 and Dashti *et al.*, 2010), the second one has yet to be thoroughly investigated. For this reason the investigation of the effect of shallow foundation's bearing pressure on their liquefaction-induced settlement will be one of the goals of a series of geotechnical centrifuge tests.

3 CENTRIFUGE MODELLING

3.1 Model layout

In order to verify and better understand the effect of the bearing pressure on the settlement of the footing induced by liquefaction suggested by the collected field data (Figure 5), each centrifuge model tested consists of four rigid square footings (B=55mm at model scale) resting on level loose liquefiable sand (Dr=30%) deposits of different thickness. Despite having the same dimensions, each model footing exerts a different bearing pressure, namely 30, 60, 90 and 130kPa (Figure 6).

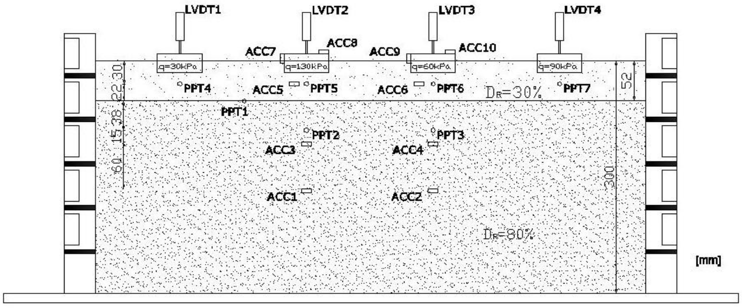

Figure 6. Test DB2 Layout and location of instruments. All dimensions in mm at model scale, N = 50.

This is achieved using materials of different density for the construction of the models (aluminium, steel and lead). Whereas in real buildings their mass is mainly due to the superstructure (i.e. the part of the building above ground), the mass of the models is entirely concentrated in the footing, with no superstructure present. In this way the dynamic soil-structure interaction is minimized, reducing the component of the settlement due to the ratcheting of the rigid footing in the soil which results from the rocking induced by cyclic loading.

Soil models are contained in an Equivalent Shear Beam (ESB) box developed for this specific project, consisting of stacked aluminium rings separated by thin rubber layers providing the desired flexibility (Figure 6). The container is designed so that the overall shear modulus of the end walls matches that of the soil model, minimizing dynamic boundary effect and providing boundary conditions closer to those observed in an infinite horizontal soil deposit.

For a correct scaling of soil permebility a solution of methylcellulose in water was used as pore fluid. In order to achieve a correct scaling of time in both inertial and seepage controlled phenomena, a pore fluid having a viscosity 50 times higher than that of water was required (since the tests were performed at 50 times earth gravity). This was achieved by using a concentration of 2.3% in weight of methylcellulose.

Pore pressures are measured by means of Druck PDCR81 pressure transducers, having a measurement range of 3bars. Preliminary testing showed that the transducers were not able to accurately capture frequency variation of pore pressure above 20Hz. Since the scaled input motion used in the tests has a significant frequency content in the range 30-300Hz, this limitation would have altered the quality of the recorded data. To overcome this problem the transducers had to be modified by removing the porous stone placed in front of the sensing element, responsible for the “smoothing” of the recorded pore pressure. To prevent the soil to get in contact with the sensing diaphragm of the instrument a fine wire mesh with 0.05mm openings was placed in place of the porous stone.

Accelerations are measured by means of ADXL78 iMEMS (Micromachined Electro-Mechanical Sensors) accelerometers, having a measurement range of +-70g. This type of instruments proved to be a valid alternative to PE (Piezoelectric) accelerometers due to their reduced dimensions and cost.

All the instruments were logged through two ADLINK DAQ-2204 data acquisition cards mounted on the centrifuge on-board PC. Each DAQ card can read up to 32 channels in differential mode with a scan rate of up to 3MHz. A scan rate of 2KHz has been chosen as a good compromise between data bulk quantity and resolution.

For the purposes of this paper the results of two tests, DB1 and DB2, will be analyzed. Both models consists of 300mm deep clean HST95 Congleton silica sand soil deposits with a top loose liquefiable layer having a Dr=30% and a bottom dense layer (Dr=80%). Both tests have been performed within a 50g gravity field; therefore the model soil deposit corresponds to a 15m (0.3m × 50) deep prototype sand deposit. In test DB1 the top liquefiable sand layer has a depth of 165mm in model scale (8.25m prototype scale, D_L~3B), whereas in test DB2 the liquefiable layer depth is reduced to 37mm, measured from the foundation plan (1.85m prototype scale, D_L~0.68B). Besides footing settlement, pore pressure and acceleration are measured within the soil (Figure 6) in order to more confidently explain the affirmation or rejection of the hypothesis.

3.2 Experimental results

Figure 7 shows the final settlements recorded during tests DB1 and DB2, against footing bearing pressure. The hypothesized bearing pressure dependence of the liquefaction induced settlement is confirmed by the experimental results. In both test DB1 and DB2 the heaviest footing experiences less settlement than the next lighter footing. This is in agreement with what was observed in the Maule earthquake field data. However the absolute settlement (Figure 7b) recorded in the models is higher than that observed in the field cases.

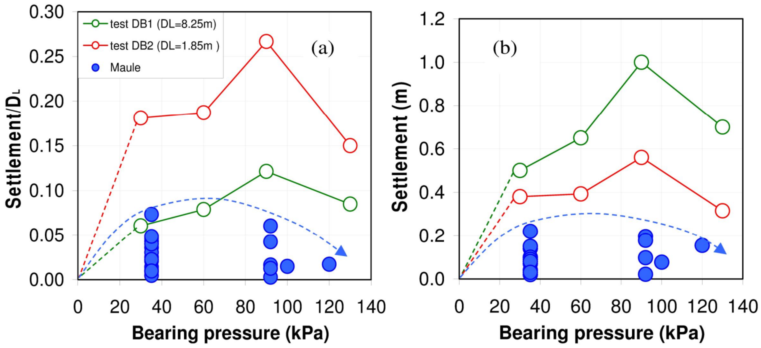

Figure 7. Centrifuge tests results compared with field observations from Maule earthquake.

While higher settlement was expected in test DB1, as a consequence of the thicker layer of liquefiable sand, settlements in test DB2 were expected to be smaller or about the same of those recorded in the Maule event, as the average D_L of these cases is higher. This is not the case, the settlements recorded in test DB2 (D_L=1.85m) are higher than those observed in the field (D_L=3.75m). A possible explanation for these counter-intuitive results is that the model soil deposits were constituted of clean sand with a homogeneous density profile, whereas the soil type observed in the field cases is a silty-sand having varying plastic fines content. Such soil may have a lower liquefaction potential than the uniformly-graded model soil, resulting in potentially smaller induced settlements. A second factor which may have determined such behaviour is the soil's relative density. The liquefiable soil's relative densities estimated from SPT blow-counts in the field cases were on average higher than 30% (value used in the models); a range of relative densities from 20 to 60% is observed, corresponding to loose to medium-dense soil (i.e. see Figure 3). Earthquake magnitude and base motion characteristics would also influence significantly the induced settlement of such structures; however our earthquake shaker allows us to reproduce during the test the earthquake motion recorded in proximity of the analyzed sites. For this reason we can confidently overlook these variables when analyzing the test results, enhancing the reliability of the resulting findings relative to the field data where such site variables are unavoidable.

In Figure 7b the same settlement data normalized by D_L, are plotted against bearing pressure (q). Maximum values of normalized settlement for the Maule field cases are in line with those recorded during test DB1, whereas the normalized settlements observed in test DB2 are significantly higher. This clearly indicates that, despite a cause-effect relationship existing between S and D_L, these two parameters are not directly proportional. In particular, if the footing rests on thin liquefiable layers, the ratio of the induced settlement to D_L will be higher compared with thicker liquefiable deposits. This observation is in accordance with the results presented in Dashti et al. (2010), who first pointed out the inadequacy of such normalization.

The reduction of S for the heavier footing model, although observed in both tests, is greater in test DB2. In presence of a thin liquefiable layer, the heaviest footing model (having q=130kPa) settled less than the lightest one (having q=30kPa), whereas in test DB1 where the liquefiable deposit is substantially thicker, even though the same behaviour is observed, the lightest model settled the least.

Excess pore pressure and acceleration measurements can represent valuable information in the analysis of settlement data. Figure 8 compares the excess pore pressure measurements recorded during shaking underneath each footing model, at a prototype depth of 0.75m from the foundation plan. It is interesting to note that, apart from the instrument sitting below the lightest model, underneath the other three models the maximum excess pore pressures generated during shaking were inversely proportional to the corresponding footing bearing pressure. The reduced excess pore pressure under the heavier footings indicates that they were resting on a "bulb" of non-liquefied soil, providing a possible explanation of the observed reduction in the settlement for high values of q.

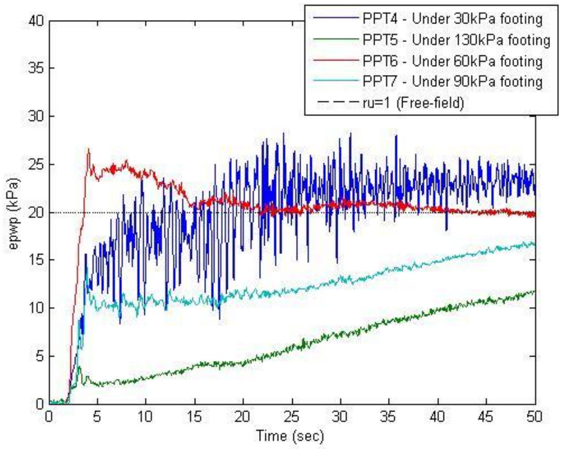

Figure 8. Excess pore-pressure build-up under footing models, at a depth of 0.75m below foundation plan.

Right after the peak excess pore pressure has been reached (5s after the beginning of the earthquake) the pore pressure in the horizontal plane tends to re-equilibrate, with fluid draining out the soil underneath the 60kPa footing toward the heavier ones, soon reaching equilibrium. The trace of PPT4, positioned under the 30kPa footing, shows a marked dilative response, with cyclic sharp drops in the measured pore pressure during shaking. Vertical stresses below the lightest footing model are lower resulting in higher r_u (excess pore pressure ratio, $\Delta u/\sigma'_v$), unlike the other models; in this case the soil right under the footing seems to have liquefied.

The generation of excess pore pressure also influences the vertical propagation of the earthquake motion. Figure 9 shows the vertical propagation of acceleration under the 60kPa footing during test DB2; in this specific case, as soon as excess pore pressures reach their peak value, horizontal accelerations are attenuated to about 80% of the input motion.

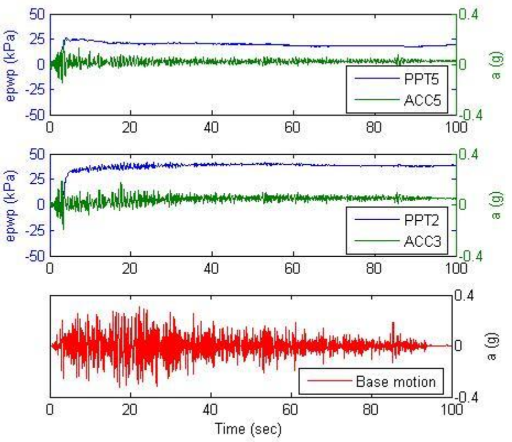

Figure 9. Vertical acceleration propagation and excess pore pressure generation under 60kPa model footing (test DB2).

It is worthwhile to notice that PPT2, which is positioned about 2m deep from the top of the dense sand deposit (Figure 6), shows significant generation of excess pore pressure despite the high relative density of the sand at that location. However, even though values of r_u approaching unity have been observed in the dense sand layer, this soil is close to its maximum density and has a limited potential for deformation, resulting in a negligible contribution to the overall induced settlement of footings resting on the ground surface.

4 CONCLUSIONS

The results of the two centrifuge tests discussed confirm the hypothesis made based on field observations. The dependence of the liquefaction induced settlement of shallow foundations on the footing bearing pressure has been verified experimentally, confirming that as bearing pressure increases then settlement increases until a reduction in the observed settlement occurs for very high values of bearing pressure. Although the trend observed in field data was successfully reproduced experimentally, the magnitude of settlements recorded during the centrifuge tests was significantly higher. This is attributed to the differences in soil properties between the field cases and the experiments: the soil models tested were not meant to carefully reproduce the analyzed field cases but instead represent a controlled and simplified general case (clean loose sand with no fines content). The tests were performed on the University of Dundee's geotechnical centrifuge, equipped with an Actidyn Q67-2 servo-hydraulic shaker. Versatility is the key feature of this device, which is able to carefully reproduce most of the historic earthquake motions. This feature proved very important when analyzing the data as it allowed us to overlook all those variables related to the input motion (such as earthquake magnitude, frequency content, peak acceleration, etc.), since in the experiments we reproduced a scaled version of the 2010 Maule earthquake.

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