6th International Conference on Earthquake Geotechnical Engineering
1-4 November 2015
Christchurch, New Zealand

The Benefits and Opportunities of a Shared Geotechnical Database

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ABSTRACT

The Canterbury Geotechnical Database (CGD) is an online database that has been developed for the rebuild of Christchurch following the 2010 - 2011 Canterbury Earthquake Sequence (CES). It was designed as a searchable repository for sharing existing and new geotechnical information along with supporting geotechnical applications for building and resource consents. As at March 2015, the database contains over 18,000 cone penetration test records, 4,000 boreholes, 1,000 piezometers with accompanying groundwater monitoring records, 6,000 laboratory test records plus other data. This data can also be used for more strategic purposes such as assisting with the recovery for future natural disasters, increasing the resilience of other areas of New Zealand, catastrophe loss modelling and informing regulatory processes. The extensive geotechnical database when combined with other data sets enables close examination and modelling of ground and built infrastructure performance. The lessons learnt from these analyses can be applied to improve resilience and also used to inform regulatory policy decisions in other areas of New Zealand. This paper provides examples of how this extensive dataset can be used and outlines the benefits to geotechnical and hazard management practice in New Zealand if it were expanded to a nationwide database.

Introduction

The Canterbury Geotechnical Database (CGD) is an online database (available via the website: https://canterburygeotechnicaldatabase.projectorbit.com) that has been developed for the rebuild of Christchurch following the 2010 - 2011 Canterbury Earthquake Sequence (CES). This sequence includes four main earthquake events; 4 September 2010; 22 February 2011; 13 June 2011; and 23 December 2011. These events caused widespread liquefaction related land, infrastructure and building damage, affecting approximately 50% of the horizontal infrastructure (roads, electricity, waste water and fresh water), 51,000 of the 140,000 residential properties in Christchurch as well as damage to the commercial land and buildings. As a result of the damage, the NZ government classified residential land in Canterbury into red or green zones. The residential Red Zone includes land where the repair and rebuild process was identified by the Canterbury Earthquake Recovery Authority (CERA) to be not practical to implement, because the required land repair and improvement works would be difficult to implement, prolonged, and disruptive for landowners. These properties were able to sell their properties to the NZ government to manage the withdrawal process. The balance of the residential land on the plains was further categorized by the Ministry of Building, Innovation & Employment (MBIE) into three technical categories (TC1, 2 and 3) to assist with the rebuilding of residential houses. The

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spatial location of the technical category land areas are shown on Figure 1a and differentiate the levels of specific geotechnical investigation and foundation design options that are required to address the potential liquefaction issues.

The CGD was designed as a searchable repository for existing and new geotechnical information along with supporting geotechnical applications for building and resource consents. While the data is primarily used for geotechnical design of ground improvement, building foundation repairs, foundations for new buildings and geotechnical design for infrastructure repairs, it can also be used for more strategic purposes such as, assisting with the recovery for future natural disasters, increasing the resilience of other areas of New Zealand, catastrophe loss modelling and informing regulatory processes. Scott et al. (2015) outlined the benefits of the CGD and this paper further expands on some the advantages of sharing such geotechnical information by providing some additional examples and looks ahead to how a nationwide database could be used not only to benefit the engineering and planning profession but also for the strategic purposes outlined above.

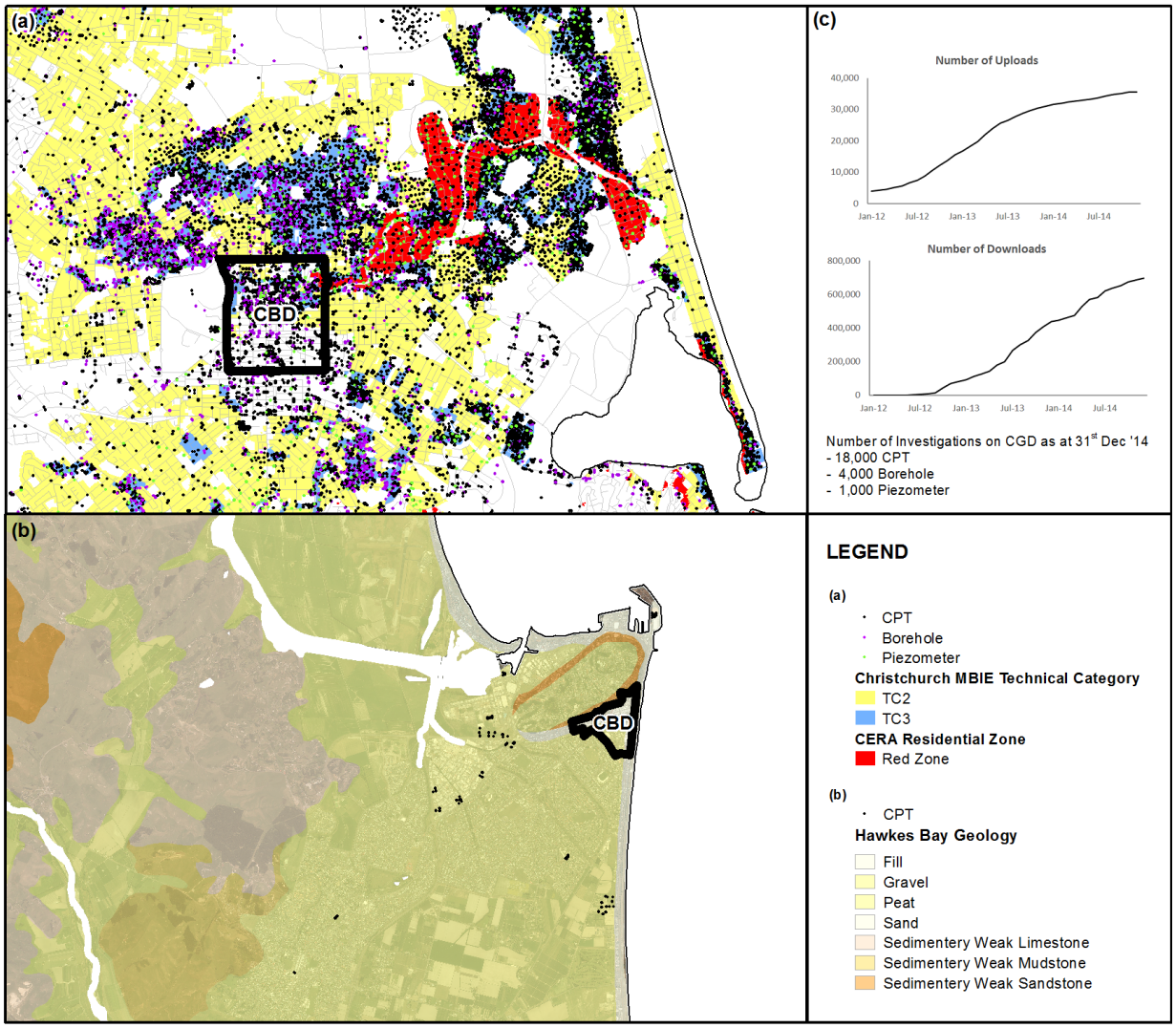


Figure 1. (a) Canterbury geotechnical database and (b) Hawkes Bay geotechnical database.

Extent and Distribution of Geotechnical Data

As discussed in Scott et al (2015) the CES and the impact it caused led to the creation of the CGD by CERA to facilitate and increase the confidence of the greater Christchurch recovery process. As at December 2014, the CGD holds approximately 18,000 Cone Penetration Test (CPT) traces, 4,000 borehole log records, 1,000 piezometers with accompanying groundwater monitoring records, 6,000 laboratory test records and other information such as the mapped land damage after each main event, LiDAR survey data and groundwater mapping information. The geographic distribution of CPT, borehole logs and piezometers is shown in Figure 1a. It is noted that almost all of the geotechnical investigation data is spatially located on the TC3 land, where MBIE intended for deep geotechnical investigation, site specific assessment and design to be undertaken. This figure illustrates the predominance of CPT in the east of the city reflecting thicker deposits of soft to firm silt and loose to medium dense sand. Interbedded gravels typically occur to the west of the Central Business District (CBD). These gravels are generally unable to be penetrated by CPT and therefore borehole investigations with accompanying standard penetration testing (SPT) are the dominant investigation tool if deep investigations are required.

The CGD has been very successful in a large part due the sharing of geotechnical information between the private and public sectors. Figure 1a shows time plots of uploads and downloads of geotechnical information to the CGD with the uploaded data amounting to 35,000 data files which have subsequently been downloaded approximately 700,000 times. This means that on average data is being re-used 20 times and therefore geotechnical engineers are reviewing more data relating to surrounding ground conditions than what they normally would without a CGD, resulting in more informed specific assessment and design. Also the sharing of geotechnical data has enabled costs savings (resulting from a reduced scope of necessary investigations due to access to neighboring geotechnical data) for the Canterbury recovery of between NZ$50 and NZ$100M. This excludes contract administration, supervision and reporting related costs associated with generating new geotechnical information. More recently, the CGD has been expanded to encompass a part of the Hawkes Bay region of New Zealand and a similar database has also been set up for the Auckland region. The Hawkes Bay geotechnical database is still in its infancy and is slowly starting to populate. The locations of the available CPT data are shown in Figure 1b.

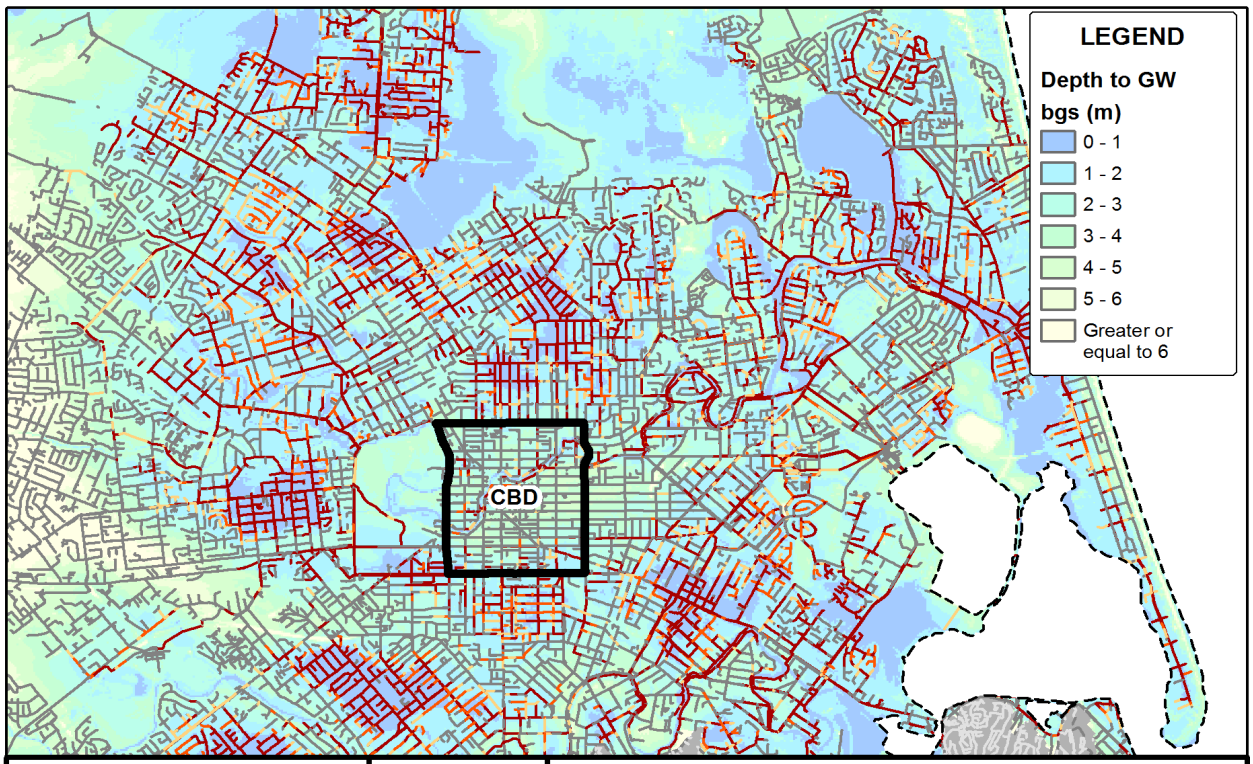
Examples of Potential Data Analyses

The high density of data enables interpolation between points which in turn allows interpretative maps to be generated such as depth to groundwater, soil behaviour type index, I_c, CPT tip resistance, q_c (which is a measure of soil density), depth to hard / dense soil layers etc. Figures 2 to 6 below present some examples of how the extensive dataset collated in the CGD can be analysed and mapped for more strategic purposes, outlined above, to inform decision makers. Without this extensive dataset these analyses would be difficult to produce.

Horizontal Infrastructure Networks (e.g., roads, electricity, waste water and fresh water)

Figure 2 shows the entire Christchurch City Council (CCC) Wastewater (WW) network overlain on the 15th (lower), 50th (mean) and 85th percentile (higher) groundwater surfaces (van Ballegooy

et al, 2014) as well was as the CPT based I_c layers over the portion of the network in the TC3 area. The red pipes (about 30% of the network) represent the part of the network which is likely to be almost always below the groundwater table (below the 15th percentile groundwater level). This part of the network when founded in sandy soils ($I_c < 2$) would probably require extensive dewatering equipment to repair or replace any pipework irrespective of the time of year the work is undertaken. The orange and yellow pipes represent the part of the network (about 6% and 7%) which is between the 15th and 50th percentile and 50th and 85th percentile groundwater surfaces respectively and is therefore for the majority of the year likely to be either below (the orange pipes) or above (the yellow pipes) the groundwater table. Lastly, the grey pipes (57% of the network) are above the 85th percentile surface and represent the part of the network which is likely to almost always above the groundwater table. This part of the network is therefore much easier to access, repair and replace as it unlikely to require dewatering gear and is less impacted by soil type. This information could be used to improve infrastructure asset management such as operational and capital expenditure budgeting. It could also be used to assess the network vulnerability to various natural hazards, such as the liquefaction hazard (when overlaid on predicted liquefaction vulnerability maps shown in Figure 3), to inform decisions about improving network resilience.



WW network pipes	Proportion of the WW network in Christchurch	Proportion of the WW network in the TC3 area of Christchurch	
		$I_c < 2$	$I_c > 2$
Above 85th Percentile Surface	57%	34%	13%
Between Median and 85th Percentile Surface	7%	7%	2%
Between 15th Percentile and Median Surface	6%	5%	2%
Below 15th Percentile Surface	30%	26%	10%
Total	100%	100%	

Figure 2. CCC WW network overlain on the 15th, 50th and 85th percentile groundwater surfaces.

Predicted Ground Behavior for Natural Hazards Planning Purposes

Figures 3a and 3b show the calculated one-dimensional post-liquefaction reconsolidation settlement (S_{V1D}) over the top 10 m of the soil profile using the Boulanger and Idriss (2014) liquefaction triggering methodology at the Serviceability Limit State (SLS) and Ultimate Limit State (ULS) design motions (representing the 25 and 500 year return period motions) respectively as specified in the MBIE (2014 & 2015) guidelines. These analyses dictate the type of foundation solutions required for residential buildings using the MBIE guidelines.

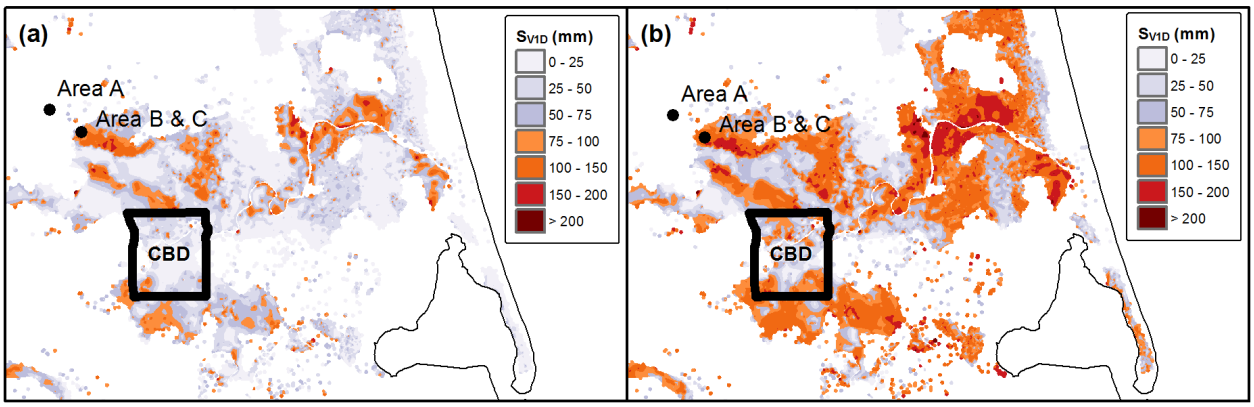


Figure 3. Calculated S_{V1D} at the (a) SLS and (b) ULS ground motions.

When comparing Figure 3 with the observed land damage maps shown in van Ballegooy et al. (2015), it can be noted that areas with values of calculated S_{V1D} > 75 mm for the SLS scenario generally experienced minor-to-moderate and moderate-to-severe liquefaction related land damage following the September 2010 event. Similarly, areas with values of calculated S_{V1D} > 75 mm for the ULS scenario generally experienced minor-to-moderate and moderate-to-severe liquefaction related land damage following the February 2011 event. Calibration of liquefaction vulnerability parameters against observed ground performance, allows liquefaction vulnerability mapping to be undertaken for a variety of earthquake scenarios in other areas in New Zealand with similar ground conditions, only if geotechnical data was available through a nationwide database. This would help identify the most vulnerable areas and this information could be used to:

- Eliminate or reduce urban intensification in such areas and therefore result in decreased community vulnerability;
- To better plan critical infrastructure to avoid these areas or to increase their resilience if avoidance is not possible;
- Better understand where affordable housing developments can be undertaken, not requiring significant investment in more expensive foundation and/or ground improvement solutions;
- Enables consenting authorities to undertake a high level review on proposed developments and focus efforts in higher risk areas;
- Enables district plan revision writers to align development rules with predicted land performance; and;
- Enables improved catastrophe loss modelling for insurance and hazard management purposes and for emergency response specialists to undertake appropriate scenario response activities.

Modelling of Appropriate Foundation Solutions

MBIE has progressively developed guidelines on foundation design since 2010. These guidelines where last updated in 2015 to include a series of integrated foundation and/or refined ground improvement solutions to address various levels of predicted ground performance. The MBIE guidelines specify a number of criteria based on specific geotechnical assessment to determine which foundation systems are appropriate to be used on various soil profiles. The criteria include calculation of the S_{V1D} parameter at the SLS and ULS scenarios (shown in Figure 3a and 3b respectively) and assessed against thresholds to differentiate between where various foundation and ground improvement solutions can and cannot be used. Models have been developed to illustrate where expected foundation solutions may be utilised geospatially in the rebuild of Christchurch based on the MBIE (2015) guideline criteria. As an example, Figure 4a shows where TC1, TC2 and TC3 structural foundation systems can generally be applied in the TC1, TC2, and TC3 areas of Christchurch. Figure 4b shows where 1.2 m thick Gravel Rafts (GR) and Soil Cement Rafts (SCR) in conjunction with TC2 foundations can generally be applied in the TC3 area, Figure 4c shows where 4 m deep Rammed Aggregate Piers (RAP) and Stone Columns (SC) in conjunction with TC2 foundations can generally be applied in the TC3 area and Figure 4d shows where 4 m deep Driven Timber Poles (DTP) in conjunction with TC2 foundations can generally be applied in the TC3 area. It is noted that RAP/SC ground improvements require construction verification testing to demonstrate that the post-improvement soils achieving the target densities specified in the MBIE guidelines. RAP/SC ground improvements are less effective in silty soils compared to sandy soils and hence a gradation of the level of confidence (or likelihood) that the RAP/SC ground improvement will achieve the target criteria is shown in Figure 4c. The dark green areas indicate a very high likelihood (close to 100%) and the light green shading indicating a very low likelihood (close to 0%) that RAP/SC ground improvement will be successful.

If a similar density of geotechnical data sets exist elsewhere, or were to be established through a nationwide database, then similar geospatial foundation system models could be prepared for other areas of New Zealand. This information could be used to:

- Allow the regulatory authorities to test criteria for reasonableness of outcome and avoid unintended outcomes of proposed design solutions;
- Guide building design and urban growth strategies of territorial authorities;
- Give planners, developers and engineers, a big picture view of what is possible and where;
- Undertake cost benefit analyses of investing in robust foundation solutions as opposed to implementing more routine but more vulnerable foundation systems and accepting the risk of significant building damage during a significant seismic event;
- Help quantity surveyors and estimators to estimate appropriate foundation rebuild costs so that property owners can specify appropriate sum insured values for their insurance policies; and;
- Enable specialist contractors to assess opportunities for investment in specialist equipment and ground improvement construction techniques.

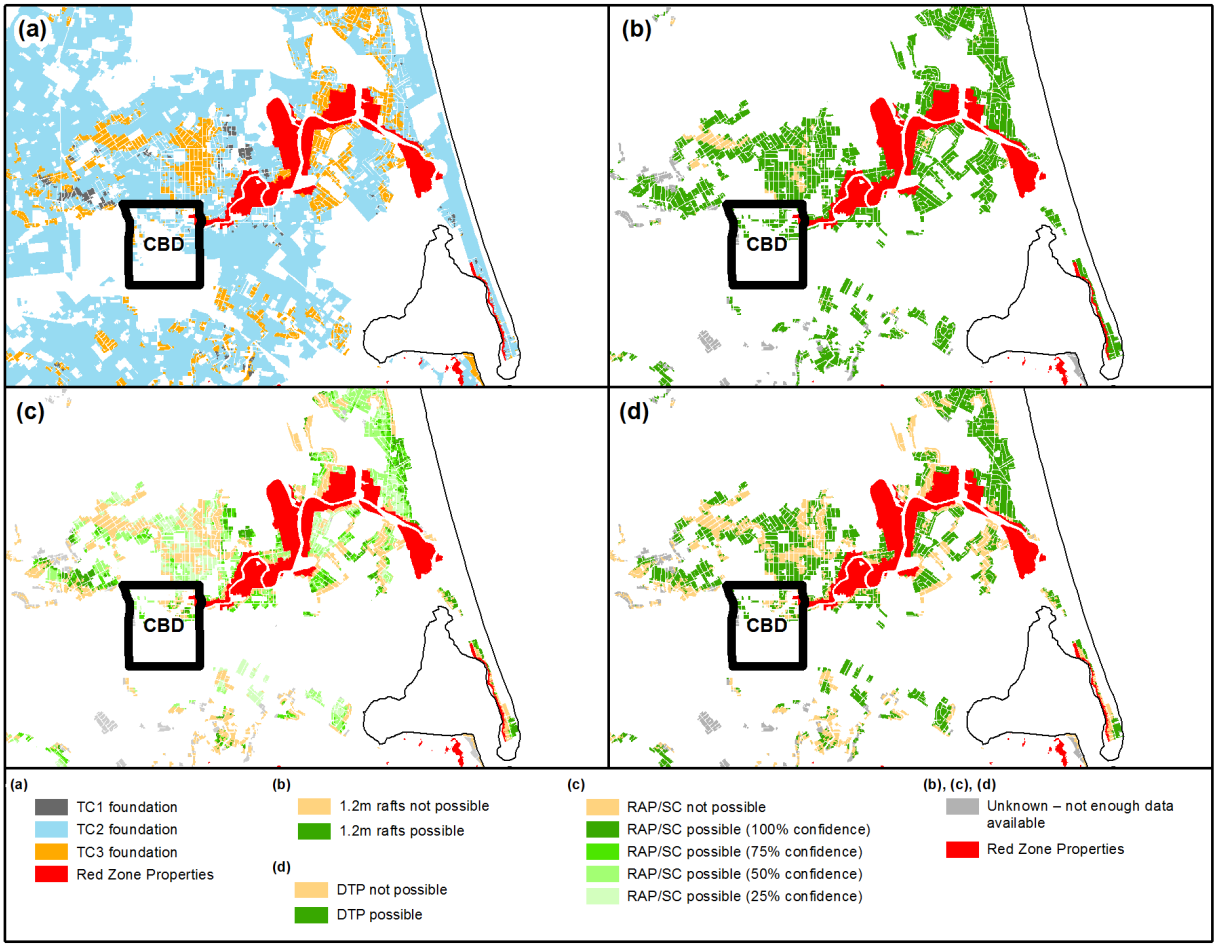


Figure 4. Foundation solutions based on the criteria in the MBIE (2015) guidelines. (a) Map of where TC1, TC2 and TC3 structural foundation systems can generally be used. (b, c and d) Maps of where 1.2 m thick GR or SCR rafts (b), 4 m deep RAP/SC (c), and 4 m deep DTP (d) in conjunction with TC2 foundations can generally be used.

Subsurface Geological Features

As discussed previously, a shared geotechnical database like the CGD enables industrywide easy access to a high density of geotechnical data. This data can be used to better inform the development of a geological model for a given area which in turn can be used to better predict future performance enabling geotechnical engineers to provide more informed design recommendations (potentially resulting in more appropriate and economic foundation solutions). To demonstrate this, three areas of Christchurch (areas A, B & C in Figure 3) which experienced similar levels of ground shaking are compared in Figure 5. Despite their close proximity to each other, these three areas performed differently as demonstrated by the land damage observations and liquefaction related ground surface subsidence as presented in Figure 5. The land damage observations show area A performed worse than areas B & C during the September 2010 event. However the land damage observations over the CES are comparable for all three areas. The liquefaction related ground surface subsidence over the CES shows that areas A & B subsided by similar amounts, typically in the order of 300 to 400 mm. Comparatively the ground surface subsidence in area C is less than areas A & B with typically recorded settlement in the order of

100 mm to 300 mm. The land damage and subsidence observations indicates that area A is vulnerable to liquefaction related land damage at lower Peak Ground Accelerations (PGA). While area B appears to subside due to liquefaction related effects it does not appear to be vulnerable to liquefaction related land damage at lower PGA. Area C has not significantly subsided and does not appear to be vulnerable to liquefaction related land damage at lower PGA.

Figure 5 also shows the calculated SLS S_{V1D}[6] at each CPT location for each of the three areas. This liquefaction vulnerability parameter predicts similar performance for areas A & B with predicted SLS S_{V1D} typically between 100 to 150 mm, whereas for area C the S_{V1D} parameter is predicting considerably better performance with predicted SLS S_{V1D} typically between 0 to 50 mm. The contrast in calculated SLS S_{V1D} values between the adjacent B & C areas indicate a sharp geological boundary change between these two areas and reveal the location of a potential historic infilled river channel. The identification of this feature would not have been possible without the density of CPT data available in the CGD. With reference to the September 2010 land damage observations, while the S_{V1D} parameter is doing a good job of predicting performance for an SLS event in areas A & C, it appears that it is not capturing an important element which is influencing liquefaction related performance in area B. Based on the MBIE (2015) guideline criteria, shallow ground improvements in conjunction with TC2 foundations cannot be used in areas A & B because the SLS S_{VID} is greater than 100 mm. However, the guidelines recommend that engineering judgement should be applied supported by detailed examination of the geotechnical data and observed land performance throughout the CES rather than strict observance to the criteria.

Figure 6 shows plots of q_c, I_c and the calculated liquefaction triggering factor of safety (FS) vs depth over the upper 10 m for the CPT traces within each of the areas indicated in Figure 5. The liquefaction triggering FS values are calculated using the Boulanger and Idriss (2014) liquefaction triggering methodology at the design SLS ground motions. Sensitivity plots of calculated S_{V1D} vs PGA over the top 10 m of the CPT traces for a magnitude 6 earthquake are also shown in Figure 6. Examination of the CPT traces in these three areas help to explain why the S_{V1D} liquefaction vulnerability parameter is over-predicting the liquefaction related damage at SLS levels of earthquake shaking when compared to the observed September 2010 performance.

For area A the CPT I_c traces indicate a highly interlayered soil profile with I_c values rapidly fluctuating between 1.8 and 3 all the way down the CPT trace. The q_c traces indicate a relatively loose soil profile typically less than 5 MPa over the full length. For area B the CPT q_c and I_c traces are similar to area A except between approximately 3 and 4 m below the ground surface (bgs) where a zone of very soft material is encountered with measured q_c typically less than 1 MPa and the I_c values typically above 2.6. Similar to areas A & B, area C also shows relatively loose material over the upper 3 to 4 m. Below this level the measured q_c values increase to 10 to 20 MPa with the majority of the CPT traces terminating by 6 m because they were unable to penetrate the denser soils. The I_c values typically range between 1 to 2 below 4 m indicating the presence of sand and gravel layers.

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[6] It is important to note that any of the existing liquefaction vulnerability parameters, such as LPI, LPI_{ISH} or LSN, (described in van Ballegooy et al., 2015) could also be used to demonstrate this point. The Sv_{1D} parameter is used in this case because it is the parameter used in the MBIE (2015) guidelines.

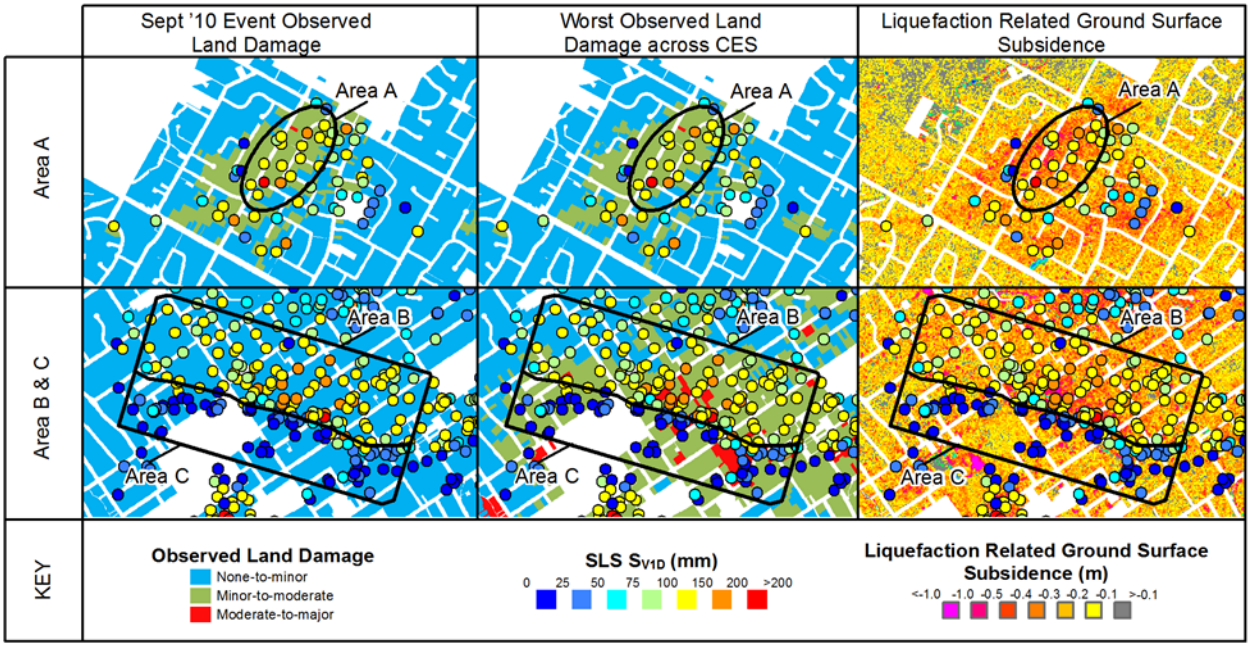

Figure 5. Land damage observations for the September 2010 event (left column) and the CES (middle column) and liquefaction related ground surface subsidence (right column) for areas A, B & C. The calculated SLS S_{V1D} values at each CPT location are shown on each of the maps.

Based on the q_c and I_c plots it is possible to infer typical ground models for each of the three areas. Area A could be described as a relatively loose silty sand to sandy silt to a depth of 10 m. Area B could be described as a relatively loose silty sand for the upper 3 m underlain by a 1 m thick soft silty clay / organic soil between 3 and 4 m bgs underlain by a relatively loose silty sand for the remainder of the ground profile. Area C could be described as relatively loose silty sand to sandy silt for the upper 3 to 4 m bgs which is underlain by dense sand and gravel layers.

The FS vs depth plots shown in Figure 6 can be used to interpret the thickness of any liquefying and non-liquefying layers within the ground profile at the SLS ground motions. The soil profile in area A is characterized by the presence of a non-liquefying crust for the upper 1.5 m underlain by a thick deposit of liquefying soil layers for the remainder of the ground profile. Similar to area A, area B is characterized by the presence of a non-liquefying crust for the upper 1.5 m which is underlain by a 1.5 m thick liquefying layer. The soft silty clay to organic soil between 3 and 4 m bgs is a non-liquefiable layer which is then underlain by liquefying material for the remainder of the ground profile. For area C the FS periodically dips below 1 in the upper 3 to 4 m indicating the presence of a few interbedded layers of liquefying soils sandwiched between non-liquefying layers. Below 4 m no liquefaction triggering is generally predicted

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Combining each of these information sources together an interpretation of the predicted liquefaction vulnerability of the three areas can be made that reconciles with the land damage observations. Area A is vulnerable to liquefaction related damaged at SLS levels of shaking. This vulnerability is demonstrated by the presence of a thin non-liquefying crust underlain by a thick liquefying layer. The high calculated S_{V1D} values appropriately capture this vulnerability. Area B is not vulnerable to liquefaction related damage at SLS levels of shaking. However this is not

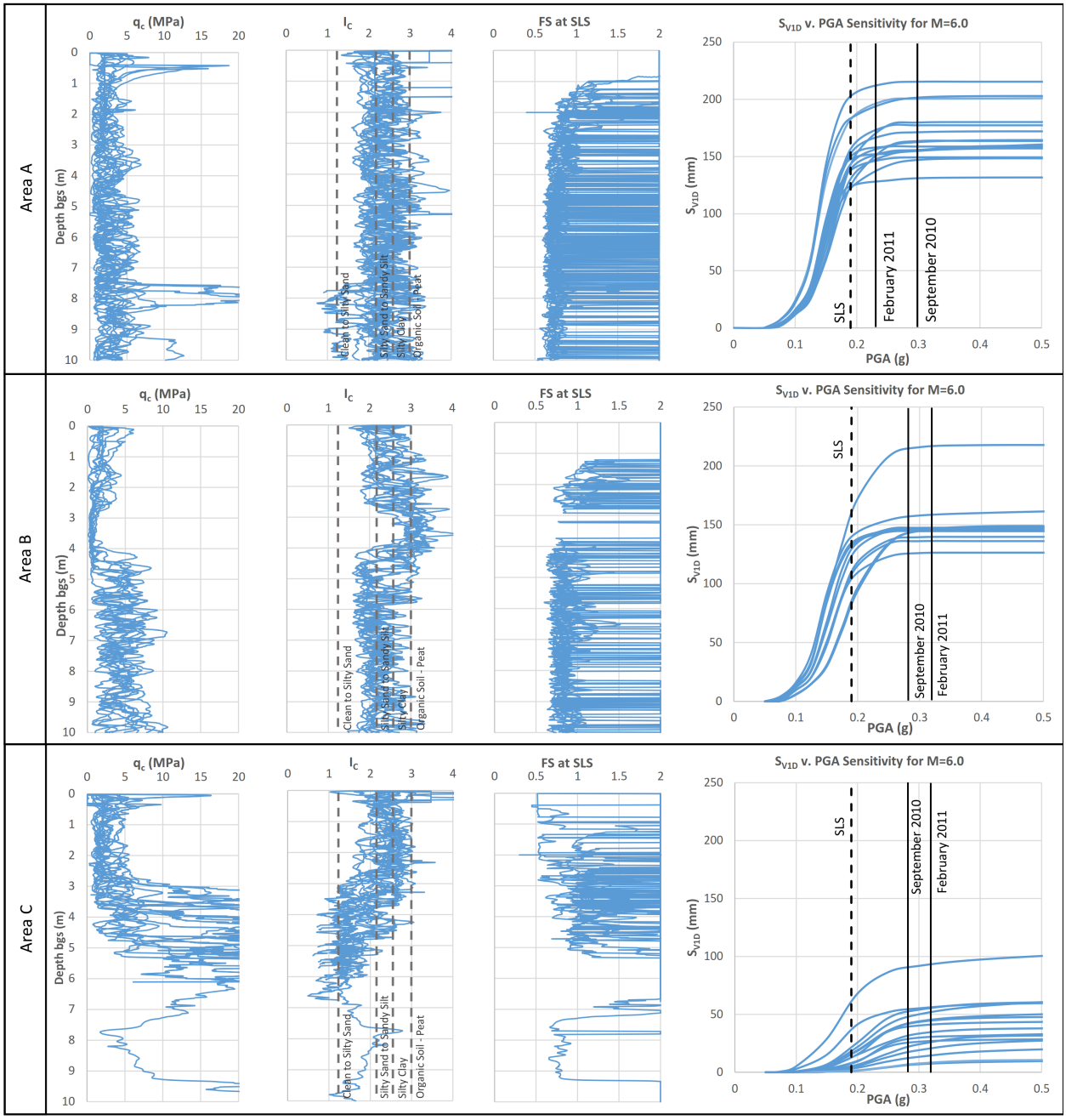

Figure 6. Plots of q_c, I_c, and SLS S_{V1D} vs depth over the upper 10 m and plots of S_{V1D} vs PGA for a magnitude 6 earthquake for the CPT within areas A, B & C as shown in Figure 5.

captured by the S_{V1D} liquefaction vulnerability parameter as it is predicting relatively high S_{V1D} values at a similar range to those calculated in area A. From the CPT traces it can be seen that the non-liquefiable silty clay to organic material soil layer at 3 to 4 m bgs is probably suppressing the damaging effects of the liquefaction occurring below this depth. The S_{V1D} parameter is not adequately able to account for the presence of this layer and hence based on observed land damage and engineering judgement shallow ground improvements (such as GR

and SCR rafts) in conjunction with TC2 foundations could likely be used in this area even though the MBIE (2015) guideline criteria of SLS S_{V1D} < 100 mm would not be achieved. Finally area C is not vulnerable to liquefaction related damage at the ground surface at SLS levels of shaking due to the presence of the dense sand and gravel layers encountered from about 4 m bgs. The S_{V1D} parameter is able to capture this vulnerability by predicting relatively low values. As shown by this example, engineering recommendation with respect to area B have been better informed and improved as a result of the collated geotechnical data available in the CGD.

The CGD geotechnical dataset in conjunction with the land damage mapping after each of the main CES earthquakes is an extremely valuable reference point for predicting ground performance in other areas of New Zealand with soil deposits susceptible to liquefaction. If soil profiles with CPT traces similar to the CPT profiles of areas A, B or C (shown in Figure 6) are obtained for another area of New Zealand with similar depth to groundwater, then at similar levels of earthquake shaking, similar land performance could be expected.

The lessons to be gleaned from this particular case study are:

- Confirmation that the fluvial geological environments are highly variable over short distances;
- Geological models should be developed and used to inform the interpretation of site specific CPT data;
- Access to area wide data more easily enables a geological model to be formulated which in turn can be used to better scope geotechnical investigations and improve prediction of future performance. This enables geotechnical engineers to provide more informed design recommendations (potentially resulting in more appropriate foundation solutions); and;
- Calculated liquefaction vulnerability parameters, such as the MBIE (2015) S_{V1D} parameter, should not be solely relied on to predict future performance. Engineering judgement is required underpinned by detailed examination of all geotechnical data not only from the specific site but also from the surrounding area.

Discussion and Conclusions

In this paper, examples are presented of how an extensive geotechnical dataset can be used to:

- Undertake a high level assessment of a specific project with regard to information from the surrounding area to better inform the geotechnical risks;
- Provide ground strength and seismic ground performance data that could be used in other locations in New Zealand as a benchmark for expected ground performance in similar geological settings, particularly areas with complex subsurface geological models;
- Enable infrastructure providers to be more informed in their asset management (such as operational and capital expenditure budgeting) and to better target more vulnerable areas for strengthening and also, post an event, to optimize the repair/replacement effort;
- Provide sub-surface data to regulatory authorities and decisions makers so as to enable them to make well informed land planning decisions and determine the appropriateness of investment strategies and solutions;
- Enable regulatory guidance to be prepared and assess the likely impact of guidelines and building codes;

- Enable specialist contractors to assess opportunities for investment in specialist equipment and ground improvement construction techniques;
- Help quantity surveyors and estimators to estimate appropriate foundation rebuild costs so that property owners can specify appropriate sum insured values for their insurance policies; and;
- Enable improved catastrophe loss modelling for insurance and hazard management purposes and for emergency response specialists to undertake appropriate scenario response activities.

In addition to the benefits outlined above, access to extensive geotechnical datasets enables the research community to undertake research projects that would normally not be possible as a result of budget constraints.

The data sharing model in Canterbury has enabled a significant dataset to be developed to the benefit of both the private and public sectors and it is a quiet success story in the recovery of greater Christchurch following the CES. As a result MBIE is in the process of facilitating the development a nationwide geotechnical database, building on the success of the CGD. This paper presents a compelling argument that a nationwide geotechnical database populated by geotechnical data together with an associated collaborative data sharing model will provide significant benefits to other areas of the country. It is hoped that the geotechnical community and their clients will support the existence of a New Zealand wide database and begin actively contributing data as a step towards improving and achieving long term resilience.

Acknowledgements

The majority of the site investigation data in the geotechnical database have been provided courtesy of the NZ Earthquake Commission (EQC) initially and the private sector and utility companies subsequently. The dataset has been accessed through the Canterbury Earthquake Recovery Authority. The wastewater infrastructure information has been provided by the Christchurch City Council. The support of all these parties is gratefully acknowledged. This extensive post disaster geotechnical dataset is understood to be a first of its kind in the world and would not have been possible without buy-in from both the public and private sectors to the collaborative data sharing model for the Canterbury recovery. The graphics in this paper have been generated by Tonkin & Taylor Ltd’s natural hazards group and their development was only possible with the support of EQC.

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